Experimental Setup for the Investigation of Ageing Effects in Pile Shaft Friction

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ABSTRACT

Although the shaft capacity of driven piles in sand has been reported to increase with time, observed trends are scattered and the mechanisms leading to the increases are poorly understood. Several attempts have been made to increase insights into the effects of time but a clear understanding has not yet emerged. This paper describes an experimental setup designed to investigate the ageing effects of pile shaft friction in the laboratory. Major challenges, considerations and limitations of the experiment are discussed. Results show that pile set-up phenomenon can be successfully modelled and that load-displacement characteristics are in keeping with those observed in centrifuge tests. Although the ageing effects observed in this laboratory-scale physical model may (or may not) be realistic when applied to field scale piles, comparative studies involving various influential parameters provide some qualitative indication of the relative influence of a variety of parameters.

Keywords: pile set-up, sample ageing, laboratory experiment, driven pile, sand

1 INTRODUCTION

The phenomenon of pile set-up (i.e. increase in pile shaft resistance with time) in sand has been reported for decades (e.g. Tavenas and Audy, 1972) but the mechanisms leading to this increase are poorly understood. The observations are usually scarce and highly scattered, and the compilation of case history data (e.g. Chow *et al.*, 1998) only yielded some general trends due to variations in site conditions, differences in local practices, etc. This limitation has prompted some laboratory-scale investigations in a controlled environment such as those reported by Axelsson (2000) and White & Zhao (2006). Nevertheless, no definitive conclusions could be drawn from these investigations and further interpretation was hampered by uncertainties such as:

a) Is stress similitude one of the critical elements in the pile set-up mechanism that must be complied with in the physical modelling?
b) The freshly deposited sand sample by itself ages following preparation and how does this ageing effect interfere with pile set-up effect?
c) Would staged or repeated load testing disturb this time-dependent behaviour as compared to load test response of 'fresh' or 'virgin' piles?

These uncertainties provided the motivation to design a laboratory-scale investigation that can provide an improved understanding of ageing effects on pile shaft friction. This paper discusses some of the challenges faced during modelling the pile set-up effects in the laboratory and how the concerns listed above were taken into consideration. Details and limitations of the experimental setup are elaborated, and followed by presentation of some typical tension load test results.

2 CHALLENGES AND CONSIDERATIONS

2.1 Real Time

The actual time itself following displacement pile installation controls creep and creep-induced microscopic changes between sand particles under constant effective stress (Chow *et al.*, 1998; Bowman and Soga, 2005). Previous studies concluded that, while physical modelling using centrifuge testing facilities have allowed significant advances to be made, the fact that the creep time scaling factor in sand remains as unity means that it has been generally uneconomic to study pile set-up effects in a centrifuge (Garnier *et al.*, 2007).

2.2 Stress Similitude

Previous studies of model piles in sand have been performed at 1-g condition (e.g. Robinsky and Morrison, 1964), using pressure chamber (e.g. Yasufuku and Hyde, 1995) and in a centrifuge (e.g. De Nicola and Randolph, 1999; Lehane and White, 2005). Although physical model tested under normal gravity condition is more efficient and most commonly employed, excessive dilation in small scale 1-g tests can distort the extrapolation made to full-scale behaviour (Mikasa and Takada, 1973; Sedran *et al.*, 2001). With the impracticability of centrifuge facility, stress similitude becomes a great challenge in the physical modelling exercise. The next best option is to employ a pressure chamber. Modelling can be performed by reconstituting the same sandy material at the same density as that at in-situ and specifying appropriate stress conditions. It is noted that stress gradient similitude is not achieved in a pressure chamber test.

2.3 Sample Ageing vs. Pile Set-up

An increase in sand stiffness and strength over time at constant effective stress following disturbance (i.e. ageing) has been observed in ground reclamation and densification works and is revealed by increases in cone penetration end resistance (q_c) and shear wave velocity (V_s) (e.g. Mitchell and Solymar, 1984; Ng *et al.*, 1998). Pile set-up in sand is another form of the sand ageing phenomenon due to the soil disturbance caused by pile installation. Experimental studies of displacement piles in sand require a model sand bed to be prepared prior to any installation and testing works. The freshly deposited and pressurised sample by itself would age (e.g. Joshi *et al.*, 1995) which may interfere with the interpreted pile set-up effect. Therefore, it is important to characterise the sample ageing effect specifically for the experimental setup employed and identify an appropriate testing schedule so that this undesired side effect can be mitigated.

2.4 Consistency and Efficiency

Pressure chambers (often known as calibration chambers) have been widely employed to establish interpretation methods and engineering correlations for cone penetrometers and other in-situ testing tools. With all due measures taken to keep the sample consistent (so that the experiments are repeatable and thus comparable), the variation among chambers can sometimes be high, complicating interpretation. The conventional setup of accommodating one inclusion for tool insertion (at the centre of chamber) at a time can be very time consuming and expensive. One way to increase efficiency and sample consistency is by having more than one testing point in a chamber (i.e. as one would have in centrifuge testing).

2.5 Boundary and Interaction Effects

There are four boundary condition options available (BC1 to BC4) for pressure chambers of which BC1 and BC3 are most commonly employed (Jamiolkowski *et al.*, 2001). BC3 is preferred because the setup is simpler and the zero horizontal strain resembles the in situ condition. Bolton *et al.* (1999) show that there is no significant boundary effects with rigid wall chambers if the closest distance from the penetration point to the nearest wall boundary is 10 times the cone diameter (10D). The interaction effect between points is expected to be less severe than the wall boundary constraint and therefore adopting a minimum spacing between test piles of 10D is considered acceptable.

2.6 Particle Size Effects

By using a modelling-of-model approach (varying pile diameters and particle sizes) in the centrifuge, Foray *et al.* (1998) concluded that scale effects are insignificant if the diameter of a model pile is not less than 200 times the mean particle size (d_{50}). In contrast, Bolton *et al.* (1999) demonstrated in the centrifuge cone penetration tests that the normalised q_c values were identical for cases in which the cone diameter was larger than about $20d_{50}$. Despite different findings obtained, research has generally shown that scale effects, mainly due to particle size, can be significant in physical model in sands.

One feasible way to minimise this adverse effect is by employing very fine sand or silica flour in the model (e.g. De Nicola and Randolph, 1999). Smaller particles are, however, more vulnerable to crushing (Bolton and Lau, 1988) and have different grain shape and roughness that may affect the shear band thickness (Desrues and Viggiani, 2004). It is noteworthy also that natural sand deposits are seldom as uniform as commercial clean sands and usually comprise some small amount of fines that may influence their engineering behaviour.

2.7 Pile Installation Methods

In driven pile research, installation is usually performed by pushing the model piles monotonically into the soil sample to mimic the pile driving procedure (e.g. Robinsky and Morrison, 1964; Klotz and Coop, 2001). Recent investigations conducted by Lehane & White (2005) showed that different installation procedures employed (to mimic monotonic, jacked and pseudo-dynamic installations) have a significant influence on the radial effective stress developed on the pile shaft and thus its frictional resistance. The results of tension load tests revealed that the pile installed by monotonic jacking has much lower capacity and reaches its peak stress at a relatively small displacement compared to others. The pressure cells on the pile shaft indicated that the monotonically jacked pile had higher stationary radial stresses following installation (due to a lower degree of friction fatigue) but the changes in radial effective stress during shearing (due to constrained dilation) were smaller.

2.8 Pile Load Tests

Chow *et al.* (1998) collated a number of well documented case histories regarding the gain in pile shaft resistance with time for displacement piles in sand. The very high degree of scatter was mainly attributed to the differences in the definition of initial capacity (mostly derived from end of initial driving (EOID) measurement) and to the absence of a distinction made between static and dynamic load test results. In order to avoid short-term effects due to pore pressure dissipation following pile installation, load tests should be conducted at least 12 hours after driving as suggested by Tavenas and Audy (1972) and others. Given differences between first time test and retest capacities (e.g. Axelsson, 2000), it is also desirable that any study of set-up effects should only consider ‘first time’ or ‘virgin’ capacities.

3 EXPERIMENTAL DESIGN AND PROCEDURES

3.1 Soil Properties

The soil used in many of the pressure chamber tests referred to below was collected from The University of Western Australia (UWA) test bed site at Shenton Park, located about 5 km from the city centre. The sand is predominantly siliceous with mild traces of carbonates which could provide very weak cementation and bonding between particle grains. The sand is sub-angular to sub-rounded, uniformly graded with a mean particle size (D_{50}) of 0.42 mm and coefficient of uniformity of about 2. Maximum and minimum void ratios were recorded as 0.79 and 0.44, respectively.

3.2 Equipment

A cylindrical steel chamber with an inner diameter of 395 mm, 400 mm height and 5 mm thickness was used to house the sand samples. The base plate was watertight sealed with a small opening left to allow saturation. The top plate was 40 mm thick with circular openings that allowed insertion of the model piles. The sample was subjected to vertical stresses using a reaction frame (comprising rectangular hollow sections bolted to the ground) and a hydraulic jack placed at the centre of top plate that jacked against the cross beam of the frame. A load cell was placed in line with the jack to ensure the required pressure was achieved and maintained at all times. This system, involving a near-rigid lateral wall with constant vertical stress is classified as a BC3 chamber test.

An actuator, which is normally employed in beam centrifuge experiments, was used to perform jacked pile installation and tension load tests. A miniature load cell was attached to the model pile head to record forces required for both compression and tension loads. Cone penetration tests (CPT) were performed using a 7 mm diameter cone penetrometer to characterise the soil profile as well as to investigate the effects of sample ageing. The 8 mm diameter model piles were made from mild steel, sandblasted to a centre line average roughness of 2.5 μm and had an embedded length of 300 mm.

3.3 Sample Preparation

The sand was first oven-dried, sieved to remove impurities and particles larger than 1.18 mm and deposited by air-pluviation using an automated sand rainer. Dense samples were achieved by setting the shutter opening of the hopper at 2 mm while keeping the fall height constant at approximately 120 mm. The areal uniformity was monitored by setting four control points at the edge of each quadrant during the pluviation process. The samples created were 360 mm in height with an average relative density of 78% and corresponding dry unit weight of 17 kN/m^3. The sample was then saturated by allowing water to permeate through the opening at the base plate. The water pressure head driving the flow was kept low (below 5 kPa) and a layer of geofabric laid at the bottom ensured flow was induced over the full base area. Following saturation, vertical stress was applied gradually in 10 increments with each increment maintained for about 5 minutes until a final pressure of 200 kPa was attained. The sample was then left to age under this constant effective stress for 7 days (before model piles were installed), and maintained for the remainder of the experiment.

3.4 Pile Installation and Testing

The model piles were mostly installed by impact driving method, which was achieved by controlled tapping action with a hand-held hammer with a specified number of impacts (average 400 blows for a total penetration of 300mm); this method mimicked the driven pile installation process. Some piles were installed by monotonic jacking using an actuator which involved a continuous push at a rate of 1 mm/s. A total of 6 model piles were inserted into each sample with a minimum distance between piles and between a pile and the wall boundary equal to 10D. The model piles were then load tested at a constant rate of 0.01 mm/s to their ultimate tension capacity or to a maximum displacement of 1 times the pile diameter (1D). The 1-day capacity was adopted as the reference benchmark capacity. All tests were performed on 'fresh' model piles to avoid unnecessary complications in interpretation.

4 SAMPLE AGEING

An investigation of sample ageing was performed using a 7 mm diameter cone penetrometer installed at 1 minute, 1 day, 7 days and 28 days after the final pressure has been applied to the top of the sample. The corresponding CPT net q_c data normalised by effective vertical stress, σ'_v ($Q = q_c - \sigma_v / \sigma'_v$) plotted against depth normalised by the cone diameter (D_c) are presented in Figure 1a. The measured Q profiles are typical of those measured in a sand of constant relative density in a pressure chamber (Lunne *et al.*, 1997). The cone resistance gradually developed during initial penetration up to approximately 8 to 10D_c, at which penetration a plateau was achieved and maintained until the bottom boundary was approached.

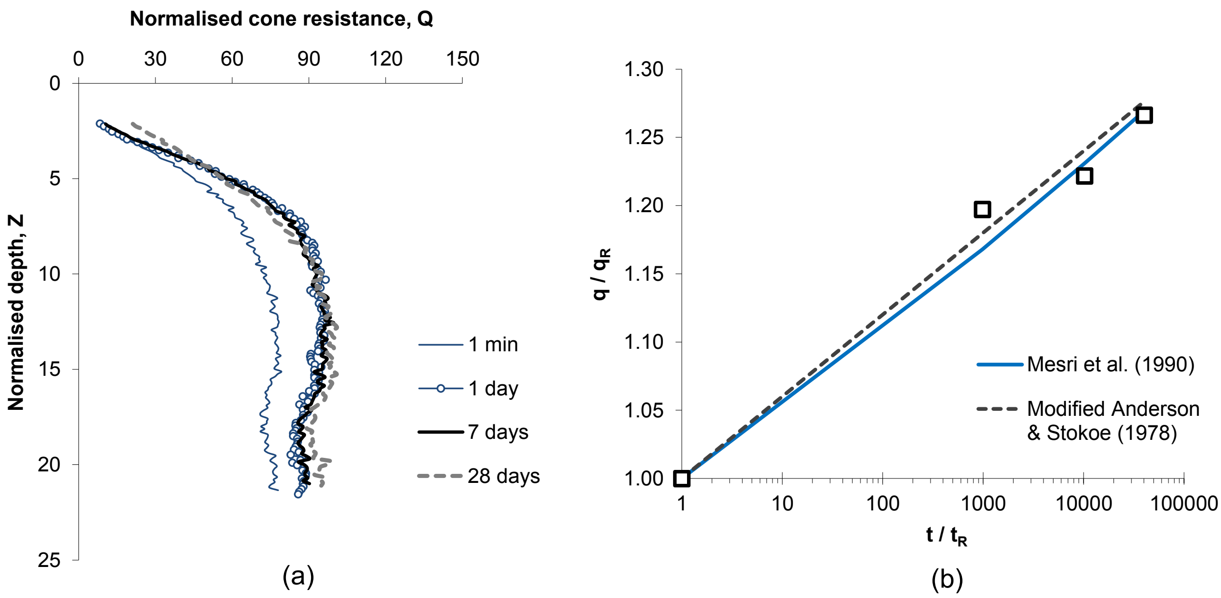

Figure 1.Cone resistance change with time

Mesri *et al.* (1990) proposed an empirical expression for the estimation of cone resistance increase with time as follows:

$$\frac{q_c}{(q_c)_R} = \left(\frac{t}{t_R}\right)^{C_D C_\alpha / C_C} \tag{1}$$

where $(q_c)_R$ is a reference cone resistance at a reference time t_R (at the end of primary consolidation); q_c is cone resistance at any time $t > t_R$; C_D reflects any densification by such disturbance as vibration and blasting; C_α is the secondary compression index; and C_C is the compression index. Figure 1b shows the normalised average cone resistances (below $10D_c$) plotted against normalised time using a logarithmic scale (both axes were normalised by their respective initial readings at 1 minute). By adopting C_D of 1.0 and C_α/C_C of 0.0225 (average for a wide range of clean granular soils which varied from 0.015 to 0.03; see Mesri *et al.* (1990)), Equation (1) can be seen on Figure 1b to fit the observed CPT data very well.

The gain in cone end resistance with time can also be expressed using a simple logarithmic function which was modified from Anderson and Stokoe (1978) who quantified the ageing effects based on small strain shear modulus (G_o):

$$\frac{q_c}{(q_c)_R} = 1 + N_{qc} \log\left(\frac{t}{t_R}\right) \tag{2}$$

where N_{qc} is the normalised change in q_c per log cycle of time. The least squares regression line can be approximated very well by assuming N_{qc} of 0.06 using Equation (2) as included in Figure 1b. This 6% (for every log cycle of time) of sample ageing effects was the basis used to determine the working schedule for this experimental study. Considering various constraints, the authors decided to install the model piles after 7 days (~10000 minutes) of sample ageing, which meant that the gain in q_c over the selected investigation period of 1 month for pile set-up would be less than 5%.

5 TENSION LOAD TESTS

Figure 2a presents the typical results for tension load tests performed on model piles installed by driven and monotonic jacking methods. Consistency in the installation resistances confirmed the repeatability of the experimental procedures and also indicated good sample uniformity and an absence of any significant boundary or interaction effects. A few driven model piles were retested to examine differences with the behaviour of 'fresh' piles, as indicated in Figure 2b. The key experimental observations include:

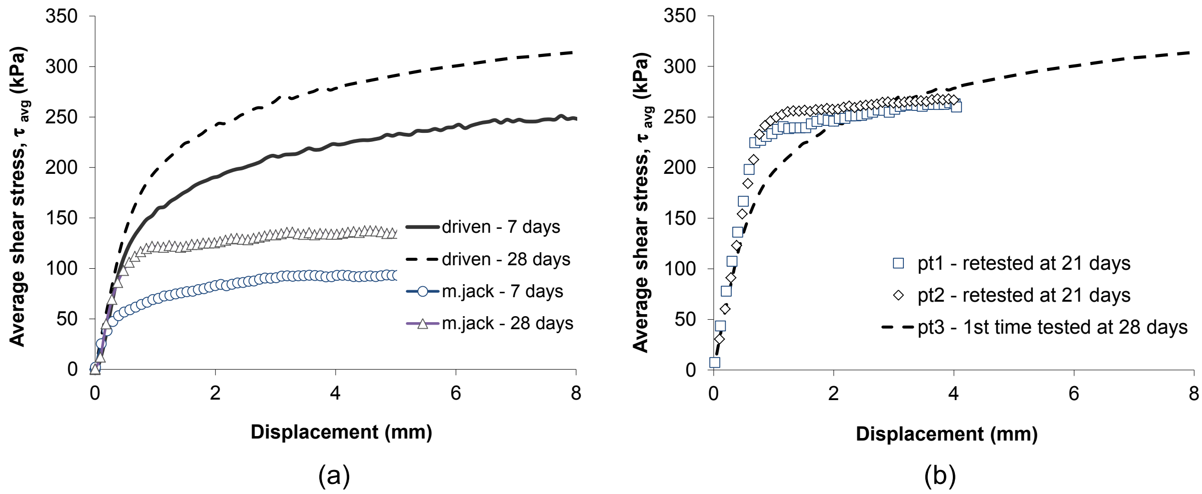

Figure 2. Typical tension test results

a) Increases in pile shaft friction with time are clearly observed for model piles installed by both installation methods.
b) The uplift capacities for model driven piles are much larger than those installed by monotonic jacking, in line with previous observations reported by Lehane & White (2005).
c) Larger displacements are required to mobilise the ultimate shear stress following more severe cyclic disturbance caused by impact driving.
d) The ultimate capacities of 'retested' piles are smaller but stiffer and reach ultimate conditions at a smaller displacement than 'fresh' piles.

6 CONCLUSION

The set-up of driven pile shaft friction was successfully simulated in the laboratory under a controlled environment. Various experimental details can be modified (such as the installation methods and static loading conditions given as an example here) to examine the influence on the load-displacement characteristics and set-up behaviours. It is also shown how effects of sample ageing can be allowed for when examining pile set-up.

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