# Load testing of single piles in clay

Joyis Thomas
Coffey Geotechnics Pty Ltd, Perth, Western Australia
Martin Fahey
School of Civil and Resource Engineering, The University of Western Australia

Keywords: pile capacity, loading direction, strain softening, centrifuge tests, clay

## ABSTRACT

The paper presents the results of a series of centrifuge model pile loading tests, carried out using closed ended aluminium piles with smooth and rough surfaces, and a piezo-probe with a smooth surface. The piles were installed by jacking in slightly over-consolidated kaolin clay. The tests were conducted in displacement controlled loading in both tension and compression. The loading tests were performed to assess the effect of installation stress level, pile dimension, surface roughness, loading direction, rate of loading and repeated loading on pile capacity. The similarity between the installation and extraction pile capacities is also discussed.

## 1 INTRODUCTION

Piles supporting offshore structures can experience repeated loading of a cyclic nature with a change of loading direction due to wave and wind loading. Wharfs supported on piles can also experience slow repeated loading with a change of loading direction due to berthing and mooring loads. The repeated loading with change of loading direction can cause degradation of pile-soil stiffness in addition to reduction in capacity due to strain softening effect. This paper presents the results of a series of pile loading tests carried out in a geotechnical centrifuge to study the effect of repeated loading to failure with a change of loading direction on pile capacity and pile-soil stiffness.

### 1.1 CENTRIFUGE TESTING

### 1.2 Apparatus

The tests were performed on the geotechnical centrifuge at the University of Western Australia. The machine features a swinging platform at a radius of 1.8m, on which test packages are mounted, with a total capacity of 40 g-tonne. All the tests were performed in a strongbox with inside dimensions of 650 mm length, 390 mm width and 325 mm depth. The model piles used in this project were made from 290 mm long aluminium tubes with an outside diameter of 6.35 mm and a wall thickness of 1.1 mm. A conical tip was glued onto the end to form a full displacement pile. At 100 g, this model pile represented a prototype pile of 635 mm diameter and a self weight of 206 kN. A 9.85 mm diameter miniature piezo-probe with a 700 kPa capacity pore pressure transducer was also used as a model pile. At 100 g, the probe represented a prototype pile of 985 mm diameter with a self weight of about 410 kN. The test samples were prepared by carefully pouring kaolin slurry onto a 10 mm thick sand base drainage layer protected by a layer of filter paper. It was then consolidated under a maximum vertical pressure of 150 kPa in stages in the laboratory prior to the commencement of the centrifuge tests. The sample thickness was 200 mm including another 10 mm thick sand drainage layer at the top. The sample was subjected to self weight reconsolidation at an acceleration level of 100 g in the centrifuge prior to pile installation.

### 1.3 Shear Strength Profile

Miniature T-bar penetrometer tests were performed in flight after self weight consolidation to estimate the undrained shear strength as well as to assess the uniformity of the soil in different test packages. The test was performed at a constant rate of penetration (CRP) of 3 mm/s (model) in all the tests, and the results are shown in Figure 1. The T-bar test responses are very consistent in all the tests. In Test08, a miniature cone penetrometer test was also performed and the undrained shear strength was estimated using an $N_k$ value of 12. A regression analysis of the T-bar test data in the clay shows that the undrained shear strength profile of the clay is approximately linear, giving the following equation

$$s_u = 25 + 0.5z \tag{1}$$

where $s_u$ is in kPa and $z$ is the depth in metres. The shear strength gradient with depth is thus 0.5 kPa/m.

### 1.4 Stages in Pile Tests

In the present work, a pile loading test consisted of 5 stages: installation of the pile, dissipation of the excess pore pressure due to installation, compression loading, tension loading and pile extraction. Figure 2 shows pile load-penetration plots for these stages for one such test. After complete extraction, the pile was moved horizontally to a new position with the aid of the actuator, for the next test. The pile tests were performed in the above mentioned order, unless otherwise specified. Repeated compression and tension loading tests were performed in Test08 and Test09 to assess the impact of reversal of loading direction. In Test08, the piezo-probe was installed and, after consolidation, load tested in tension and then in compression before extracting.

After self-weight consolidation of the soil, the instrumented model pile was installed by jacking at a constant rate of penetration (CRP) of 0.5 mm/s (model) up to a depth of about 135 mm from the surface. At this embedment length, the base of the strongbox would be more than 10 pile diameters below the pile base ($h/l$ = 1.48, where $h$ is the depth of the soil and $l$ is the pile embedment length) and hence the effect of a rigid base on the pile response may be considered negligible. The high resistance at the top is due to the 10 mm thick sand layer. Below this, the load increases almost linearly with depth. The average load at the end of installation is about 300 kN in prototype units. If the shearing at the base is assumed to occur in the undrained condition, then the base capacity is about 90 kN (using base capacity = $N_c S_u A_b$ with $N_c \approx 9$). The corresponding average unit shaft friction can then be estimated as 7.8 kPa.

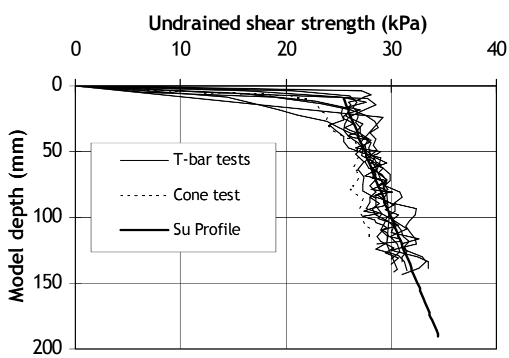


Figure 1: Undrained shear strength profile

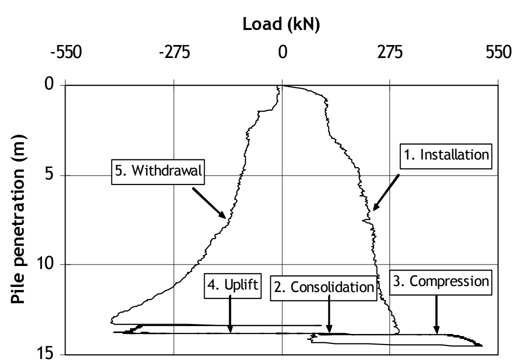


Figure 2: Stages in pile testing

The pushing force was released at the end of installation and the excess pore pressure generated was allowed to dissipate. About 90% of the excess pore pressure was dissipated in less than 21 minutes (0.4 years in prototype scale) measured from the pore pressure transducers at various radial distances. However, the next stage (compression loading) was carried out at least one hour after the pile installation. The pile was load tested in flight at a model CRP of 0.004 mm/s in general, to a displacement of about one pile diameter in compression. The tension loading test was performed at least 15 minutes after the completion of the compression test to allow for any excess pore pressure to dissipate. The pile was load tested at a CRP of 0.004 mm/s to an extraction of at least one-half of the pile diameter in tension.

In general, after the tension loading test, the pile was extracted at a CRP of 0.5 mm/s. The load-carrying capacity during pile extraction is much higher than that during pile installation. The high positive pore pressure that develops during pile installation reduces the effective stress around the pile shaft. For example, a positive excess pore pressure of about 46 kPa was observed at a radial distance of 5.4 shaft radii and at a depth of 11.8m during installation in Test04. However, a negative excess pore pressure of about 30 kPa was observed at the same point during pile extraction. This negative pore pressure increases the effective stress around the pile shaft and hence gives an increase in the pile capacity during extraction. From an average total capacity of about 400 kN and an empirically estimated base capacity of about 20 kN during pile extraction, the shaft friction can be estimated as 14 kPa, which is almost twice the value obtained during installation.

## 2 DISCUSSION OF RESULTS

### 2.1 Shaft Capacity of the Pile

The ultimate load-carrying capacity of piles driven or jacked into soil is the sum of the friction between the pile surface and the soil, and the base capacity. In cohesive soil, the shaft capacity is dominant for a ‘floating’ or friction pile and is mobilised at small displacements (about 0.5 to 2% of the pile diameter). The base capacity of a friction pile is mobilised at displacements as large as 5 to 10% of the base diameter. The ultimate load carried by the pile in Test04 at a displacement of one pile diameter is about 515 kN (prototype units), assuming that the full base capacity is mobilised at this displacement. The shaft friction of 420 kN is assumed to have fully mobilised (at the sharp change in gradient) at about 0.003 m displacement (about 0.47% of the pile diameter). This is based on the assumption that at this displacement negligible base resistance has been mobilised. Shaft friction ($\tau_s$) can be estimated in terms of the undrained shear strength of the soil, by means of an empirical factor, $\alpha$:

$$\tau_s = \alpha s_u \tag{2}$$

The average shaft friction corresponding to a shaft capacity of 420 kN is about 15.6 kPa and the value of $\alpha$ can be estimated as 0.55. The guidelines of the American Petroleum Institute (API RP2A) suggest a $\alpha$ value of about 1 for the range of undrained shear strength values in this study. The effective stress method, or $\beta$ method, for estimating shaft friction:

$$\tau_s = \beta \sigma_v' \tag{3}$$

where $\sigma'_v$ is the effective vertical overburden stress. From the average values of shaft friction and vertical effective stress, the $\beta$ value is estimated to be about 0.32 in the present study.

### 2.2 Effect of Acceleration Level

Since centrifuge tests are performed on a scaled model, inadequacies in the preparation and inaccuracies in the testing could magnify any errors in the result. Hence it is imperative to establish the same stress profile in the model soil as in the field to get the best agreement possible with the field response. Therefore all the stages of the pile testing were performed in flight. For comparison, a single test was conducted by installing the pile at 1 g after removal of 150 kPa pre-consolidation stress, and after re-consolidation in flight at 100 g, was load tested in flight in compression and tension. A time lapse of about 5 hours occurred between the removal of the pre-consolidation pressure and the installation of the pile. The load-penetration response of the piles installed in flight and at 1 g in Test04, both of which were load tested in flight, is shown in Figure 3. The responses are slightly different in compression loading: the pile installed at 1 g has a shaft capacity in compression of about 15% less than that of the pile installed in flight. However, the response in tension is almost identical. The initial pile-soil stiffness is smaller for the 1 g case in compression, but slightly higher in tension loading.

## 2.3 Scale Effect

The response obtained from two piles of different diameters – a pile with 6.35 mm diameter and a piezo-probe with a diameter of 9.85 mm - during installation and load testing in Test09 are shown in Figure 4. The installation load increases linearly, but at different rates, with increasing penetration. The piezo-probe has a base resistance ($9s_uA_b$) of about 188 kN at 5m depth and the measured total resistance was 314 kN, which gave an average shaft friction of about 8.14 kPa. Corresponding values for the single pile were 78 kN, 160 kN and 8.22 kPa. At 10m depth, the piezo-probe has a base resistance of 206 kN, total resistance of 520 kN and an average skin friction of about 10.1 kPa. The corresponding values for the single pile were 85 kN, 275 kN and 9.5 kPa. The almost identical values of shaft friction points towards the absence of any scale effect. Since the installation rate is quite low, some of the excess pore pressure dissipates during the installation process and hence an increase in average shaft friction with depth was observed.

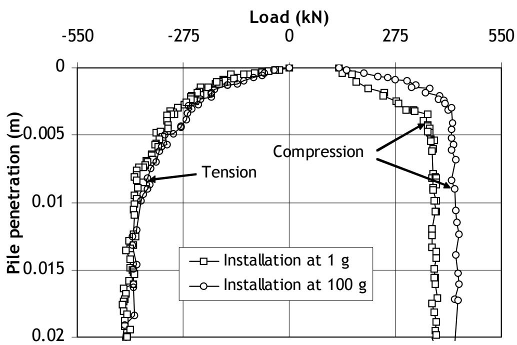


Figure 3: Effect of acceleration level

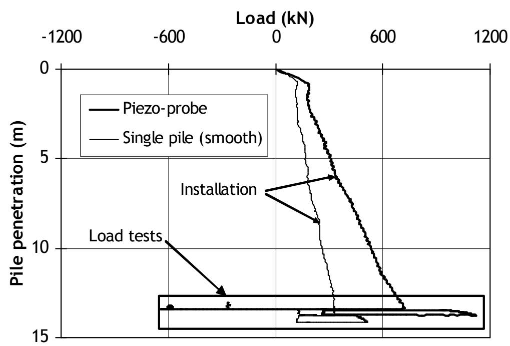


Figure 4: Scale effect

## 2.4 Effect of Surface Roughness

The responses obtained from two identical piles from Test09, one with a smooth surface and the other with a surface roughened by sand blasting, are shown in Figure 5. The installation load increases linearly with depth, but at different rates. For the rough pile, the base resistance ($9s_uA_b$) is about 90 kN and the measured total resistance is about 415 kN at 10m depth, which gives an average shaft friction of about 17 kPa. The average shaft friction estimated for the smooth pile at the same depth is about 9.5 kPa. During installation, the pile may not be undergoing purely undrained behaviour, but may be a partially drained one. The large difference in shaft friction could be due to the slow and thus partially drained installation process. The interface friction angle ($\delta$) between kaolin clay and smooth surfaced aluminium was measured as 14.7° and rough surfaced aluminium as 23° in a direct shear box test. In Test09, the shaft capacity is about 730 kN for the rough pile and about 450 kN for the smooth pile in compression loading. This corresponds to an average shaft friction ($\tau_s$) of about 26.5 kPa for the rough pile giving an increase of about 62% over that of the smooth pile ($\tau_s$ = 16.4 kPa). This increase in $\tau_s$ is proportional to the increase in ratio of $\delta$ (tan 23°/tan 14.7° = 1.62).

## 2.5 Effect of Loading Direction

The response of the piezo-probe at two testing locations in Test08 with different first loading direction is shown in Figure 6. At the first location, the piezo-probe was load tested first in compression and then in tension (C-T test). The piezo-probe was then installed at another location and after consolidation, was load tested first in tension and then in compression (T-C test).

The piezo-probe in the C-T test has a shaft capacity of about 920 kN in compression and about 930 kN at the peak in tension. In the T-C test, the piezo-probe was subjected to a reversal in loading direction for the first loading. The piezo-probe in the T-C test has a tension capacity of about 940 kN and 600 kN in compression at the peak which later dropped to 565 kN before experiencing an increase in capacity. A change in loading direction has therefore caused about 35% reduction in shaft capacity in the compression mode of loading. A further 3.5% drop in capacity was observed when the soil strain-softened. However, in the tension mode of loading a shaft capacity degradation of only 9% was observed. The first compression loading in C-T test does not seem to influence the tension capacity. In fact, both tests underwent a reversal in loading direction while loading in tension. It is clear from these tests that the first loading direction has a marked influence on the capacity in the compression mode of loading. Kraft et al. (1981) reported a fall in shaft capacity of more than 26% in compression despite a set up time of 320 days after a reversal in loading direction.

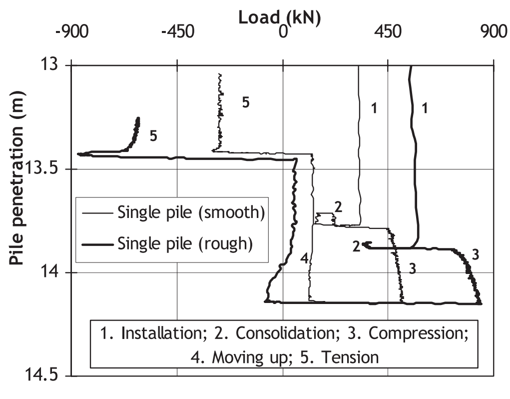


Figure 5: Effect of surface roughness

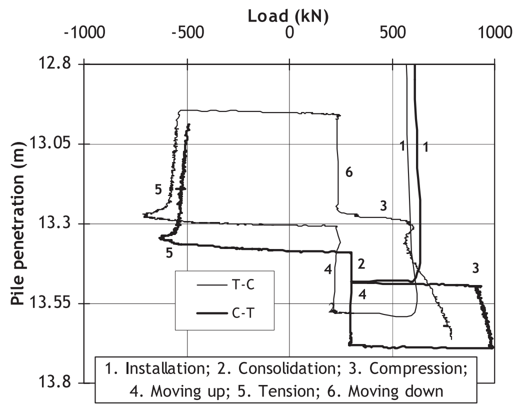


Figure 6: Effect of loading direction

## 2.6 Effect of Repeated Loading

A test sequence of repeated pile loading tests C-T-C-T on the rough pile and C-T-C-T-C on the piezo-probe was carried out and the results are shown in Figure 7. There was a 20% reduction in capacity in compression for the piezo-probe after the first reversal of loading direction. A further reduction of 6% was observed during the strain softening after the peak. It is noteworthy that the compression capacity prior to strain softening after the reversal of loading direction is very similar to the pile capacity during installation. This tendency is independent of pile diameter, surface roughness and first loading direction after consolidation. The reduction in compression capacity was observed only after a reversal in loading direction was effected. A reversal in loading direction could have caused re-alignment of soil particles. Particle re-alignment may give rise to a reduction in the radial effective stress. Randolph and Wroth (1982) suggested that the increase in the radial effective stresses due to pile installation and the subsequent consolidation may be lost when the soil particles re-align.

The shaft capacity in compression of a “wished into place” piezo-probe is about 325 kN estimated by using the in-situ effective stress state prior to installation. The shaft capacity in compression after reversal of loading direction is about twice that value and interestingly the capacity in compression prior to reversal of loading direction is about three times as much. This shows the effect of installation on shaft capacity of a full displacement pile.

## 2.7 Effect of Rate of Loading

In CRP pile load testing, the penetration rate is an important parameter, which can affect the pile capacity (Nunez and Randolph, 1984). As explained previously, a rate of 0.004 mm/s was adopted for the standard pile loading tests in this study, but in Test04 and Test05, tests were performed at different rates to assess the effect of loading rate on pile capacity. Figure 8 shows the pile load-penetration response of 4 piles installed at a CRP of 0.5 mm/s. The piles were load tested at CRP rates of 0.0025, 0.004, 0.01 and 0.05 mm/s in both compression and tension. The capacities obtained from the slow and fast loading rates were very similar.

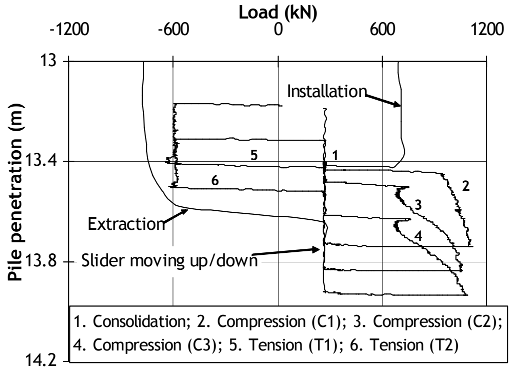


Figure 7: Effect of repeated loading

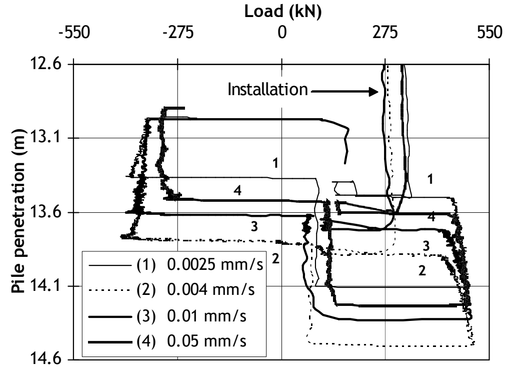


Figure 8: Effect of rate of loading

## 3 CONCLUSIONS

Conclusions that may be drawn from the centrifuge pile load tests reported herein are as follows:

- Post peak strain softening was observed only when there was a reversal of loading direction and is independent of the direction of loading.
- The capacity is greatly influenced by the surface roughness of the pile.
- In the tension mode of loading, the pile capacity reached a steady state after the strain softening. However, in compression loading, the capacity started increasing with further displacement after strain softening and in many cases almost becomes equal to the pre-reversal capacity at large pile displacements.
- The different rates of CRP loading adopted had only minor influence on the capacity.
- The pile capacity in initial loading is independent of the direction of loading. However, after a reversal of loading direction, the compression capacity can be much smaller than the tension capacity especially at small pile displacements.

## 4 ACKNOWLEDGEMENTS

The work described in this paper was carried out while the first author was a PhD student in the School of Civil Resources Engineering at UWA, supported by an APA from the Commonwealth Government of Australia, and a studentship from the Geomechanics Group at UWA.

## REFERENCES

Kraft, L. M., Cox, W. R. and Verner, E. A. (1981). "Pile load tests: cyclic loads and varying load rates." Journal of the Soil Mechanics and Foundations Division, ASCE, 107, GT1, 1-19.

Nunez, I. L. and Randolph, M. F. (1984). "Tension pile behaviour in clay - Centrifuge modelling technique, Proceedings, Symposium on the Application of Centrifuge Modelling to Geotechnical Design, Manchester, 87-102.

Randolph, M. F. and Wroth, C. P. (1982). "Recent developments in understanding the axial capacity of piles in clay." Ground Engineering, pp. 17-32.