Proceedings of the 20th International Conference on Soil Mechanics and Geotechnical Engineering– Rahman and Jaksa (Eds)

Pile ageing in clay to support life extension of offshore platforms

Vieillissement des pieux dans l'argile pour soutenir la prolongation de la durée de vie des plates-formes offshore

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ABSTRACT: Extending the life of existing offshore platforms may require assessment of long-term changes in the shaft capacity of driven pipe piles, as re-lifeing is often associated with increased platform loads. Motivated by one such extension proposed in south-east Asia, this paper presents results from a field investigation into pile ageing in soft clay in Bayswater (Perth, Australia) that was conducted over a period of one year. Both first time tension load tests and retests were conducted on 5 m long, jacked steel piles at various times after installation. A normalised peak shaft friction variation with time is established and compared with that measured in field pile tests in Onsoy clay, which has a similar plasticity index. It is shown that the ageing characteristic of shaft friction for the Onsoy and Bayswater clays is comparable when effects of consolidation are accounted for. The observed increases in capacity with time are of significance and should be considered in foundation re-lifeing (or reuse) projects.

KEYWORDS: Pile ageing; clay; shaft friction.

1 INTRODUCTION

It is well known that the shaft capacity of driven piles in clay increases with time post installation (e.g. Wendel, 1900). A large component of this increase arises as the excess pore pressures generated during driving dissipate and radial effective stresses (σ'_r) acting on the pile shaft increase. However, shaft frictions may continue to increase with time in a process referred to as ageing – with an increasing number of cases studies documenting overall pile capacity increases after the excess pore pressure has dissipated (Pestana et al., 2002; Bullock et al., 2005; Doherty & Gavin, 2013; NGI, 2013).

At present, the mechanisms giving rise to ageing in clay are not well understood. They are believed to arise due to a combination of some or all of the following factors:

(i) Creep and relaxation processes in the clay adjacent to the pile shaft after excess pore pressures have dissipated, leading to further increases in σ'_r, with this effect believed to be more significant for plugged piles (Karlsson et al., 2019);

(ii) Geochemical reactions caused by cation exchange between the steel pile and the surrounding clay, possibly enhancing cementation or bonding forces, and leading to a more dilative clay response (NGI, 2013);

(iii) Movement of the shear zone away from the pile shaft due to welding of a clay crust to the pile shaft (Karlsrud et al., 1990);

(iv) Enhanced bonding forces between clay platelets following remoulding during installation, potentially linked to thixotropy (Andersen et al., 2002).

Piling ageing experiments have shown that, after equalization of installation generated pore pressure, shaft friction increases in approximate proportion with the logarithm of time, in accordance with the following equation proposed by Karlsrud et al. (2005):

$$\tau_f(t) = \tau_f(t_0)\cdot[1+\Delta_{10}\cdot\log(t/t_0)] \tag{1}$$

where $\tau_f(t)$ and $\tau_f(t_0)$ are the shaft frictions at times t and t_0 respectively, where t_0 is a reference time usually taken at the time when full consolidation is completed.

Doherty & Gavin (2013) compiled a database of shaft capacities measured at various times after pile installation over a period of up to a decade. For these cases, it was found that there was, on average, a shaft capacity gain of 25% per log cycle of time i.e. $\Delta_{10} = 0.25$, although it should be noted that many of the tests considered were re-tests of the same piles. NGI (2013) found that the rate factor, Δ_{10}, for first time loaded piles varies significantly between clay type and propose a relationship between Δ_{10} and the clay plasticity index (I_p) and overconsolidation (*OCR*). This relationship implies Δ_{10} reduces with increasing I_p and *OCR*. However, there is insufficient consistency in the current dataset to enable general application of this equation.

Load history can complicate the assessment of ageing effects, with reporting case studies documenting that re-tested piles can display negligible to significantly enhanced ageing effects. Similarly, sustained loading (such as is present for self-weight dominated offshore platforms) can increase long term shaft resistance, although this is not consistent across the literature.

The installation method can also play a major role in pile ageing. Powell & Skinner (2006) observed $\Delta_{10} = 0.17$ measured over a period of 4 years in first time load tests on bored piles in London clay. As these piles were not driven, it is unlikely that the increase is related to thixotropy or ongoing creep strains, and must therefore arise due to geochemical interactions between the clay, water and concrete and the progressive strengthening of the shear zone (principal displacement shear) lying within the clay away from the interface.

Based on the foregoing discussion, it is evident that ageing characteristics depend significantly on the clay type as well as the pile type, pile material and loading history. More research is required to establish general trends, and in the meantime the most reliable means of assessing the likely degree of ageing of piles in any particular soil is to conduct soil-specific investigations.

2 MOTIVATION AND BACKGROUND

There are a large number of offshore platforms in south-east Asia, many of which have been in operation for decades. Some of these platforms are being considered for life extension, which may include increases in axial load carried by the piles. Initial studies showed that modern design methods could not be relied upon to justify an increase in predicted capacity, and attention turned to the assessment of pile ageing.

As part of a multi-phase research programme, this paper presents the findings from field testing in a medium-plasticity, lightly overconsolidated clay beside the Swan River in Perth (referred to as Bayswater clay). This site was selected because the clay plasticity and overconsolidation ratio is comparable to the clay found at the subject platform sites in south-east Asia, as well as with clay at Onsoy where pile ageing tests have also been performed (NGI, 2013).

Field tests were performed over a one-year period, and the results are used to advance studies into the ageing of pile shaft friction.

3 DESCRIPTION OF FIELD TESTS

While field-testing of full scale, large diameter piles is expected to provide the clearest demonstration of pile shaft friction ageing for a specific location, such testing is time consuming and expensive. Alternatively, tests can be performed on somewhat smaller diameter piles and in more accessible sites where broadly similar soil is found. The field tests described in this paper comprised tension tests on jacked 165 mm diameter, 6.5 m long open-ended steel piles, with a wall thickness of 5 mm.

3.1 *Site description*

The current test site is located in Bayswater, roughly 10 km from the Perth city centre and adjacent to the Swan River (Fig. 1). The soil stratigraphy at the site comprises a relatively thin layer of sand, overlying a deep layer of very soft clay. This clay is associated with a paleochannel and is part of the stratum referred to more generally as Swan River Alluvium (SRA) often found within 1 km of the river in the Perth metropolitan area.

Figure 1. Pile test site location

The plan location of individual piles in the test area is shown in Fig. 2, along with the position of various CPTs used to characterise the soil. The minimum distance between the 165 mm diameter piles was 2 m (roughly 12 diameters) to minimise interaction effects.

The CPTs closest to the test area are CPT-5, CPT-6 and CPT-7. The corrected cone resistance (q_t), sleeve friction (f_s) and pore pressure (u_2) are plotted in Fig. 3 and show the upper 1 m crust underlain by soft clay, with strength gradually increasing with depth. The results are similar to Low et al. (2011) from tests in the Burswood area of Perth, also adjacent to the Swan River.

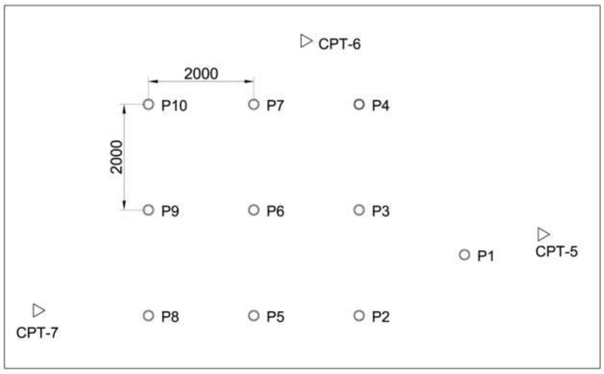

Figure 2. Schematic layout of field tests; Piles P1 to P10 and CPTs 5-7

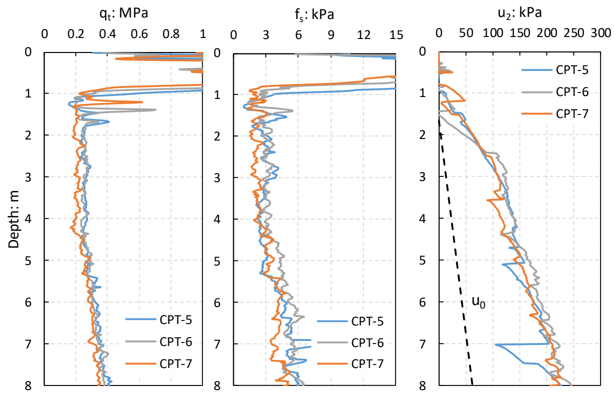

Figure 3. Cone penetration test results

3.2 *Soil properties*

The properties of the Bayswater clay are listed in Table 1. Bayswater clay has medium-plasticity, organic content approaching 10% and is lightly overconsolidated. Piezocone testing was used to determine horizontal coefficient of consolidation (c_h) using the procedure of Houlsby and Teh (1991), and assuming a soil rigidity index (I_r) of 100. The undrained strength is estimated from CPT results using a cone factor N_{kt} = 12 and is broadly consistent with the strength determined in simple shear tests on reconstituted clay (not shown).

Table 1. Properties of Bayswater clay over the embedded pile lengths

Liquid limit	57 %
Plasticity index	26 %
Clay content	33 %
Organic content	9.5 %
OCR	1 to 1.3
Horizontal coefficient of consolidation, c_h	13 m^2/yr
Undrained strength, s_u	17[1] kPa

[1] Average over the depth of interest of the piles.

3.3 *Pile installation*

A surface excavation of ~ 0.5 m was first made in the general area of the pile tests, roughly 1 month before pile installation. To eliminate the effect of the upper crust on the tension capacity of each pile, a cased borehole (190 mm in diameter) was installed to a depth of ~ 1 m below the original ground level (Fig. 4b). The soil inside the pile was augered out before an excavator was used to push the piles into the clay to the target depth of 6.5 m (Fig. 4a). The embedded length of each pile in the clay was therefore ~ 5 m, extending between 1.5 m and 6.5 m below original ground level (Fig. 5). Soil plug length ratios of between 74 % and 95 % were measured post installation.

Figure 4. Photos of (a) pile installation; and (b) installed pile.

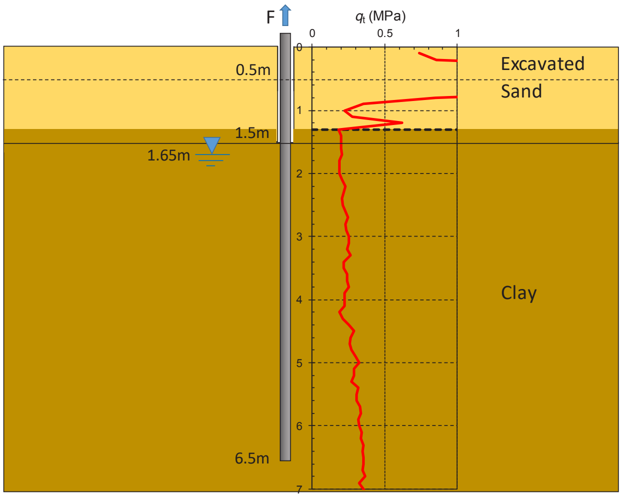

Figure 5. Schematic elevation of field test.

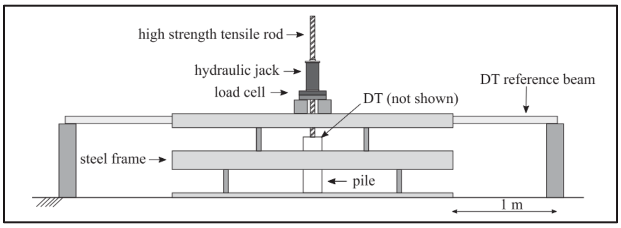

Figure 6. Pile testing setup.

Table 2. Ageing test programme in Bayswater clay

Pile	Load	Time for test (days)	*PLR* (%)	Material
P1	1st time test	0.04	91%	Mild Steel
P3	1st time test	1	76%	Mild Steel
P4	1st time test	3	79%	Mild Steel
P4	Retest	249	79%	Mild Steel
P4	Retest	370	79%	Mild Steel
P6	1st time test	3	95%	Galvanized
P6	Retest	370	95%	Galvanized
P5	1st time test	28	95%	Mild Steel
P5	Retest	370	95%	Mild Steel
P8	1st time test	217	74%	Galvanized
P10	1st time test	217	81%	Mild Steel
P10	Retest	370	81%	Mild steel
P2	1st time test	370	89%	Mild Steel
P7	Post-cycling	263	75%	Mild Steel
P9	Post-cycling	370	79%	Mild Steel

3.4 *Pile testing procedure*

Static tension tests were performed using the arrangement shown in Fig. 6. Aluminium beams were arranged in pyramid formation around the test pile, providing the reaction force to extract the pile from the ground. The ground beams were located a clear distance of around 9 pile diameters from the test pile to minimise interaction effects. The load was applied to the piles using a hydraulic jack reacting against a nut placed at the end of a high tensile strength threaded rod. This rod was connected to the pile top and passed through the loading frame, hollow load cell and the jack. Two displacement transducers were supported on a reference beam to (independently) measure pile head displacement.

In each test, the uplift force was applied in increments of 5 kN initially, reducing to 2.5 kN as the piles approached failure. For increments up to 20 kN. each load increment was held for 5 minutes before the following load increment was applied. Beyond 20 kN, each increment was held for 10 minutes.

3.5 *Test programme*

Field testing comprised 10 first times static tension tests performed at different times following installation, as outlined in Table 2. Eight of the 165 mm diameter pipe piles were fabricated from mild steel, with a further two piles fabricated from galvanized steel. All piles were sand blasted to a centreline average roughness of 5 to 7 µm. Two of the piles included re-testing to explore the effect of adding a second load stage while two other piles were tested after cyclic loading (not discussed).

4 TEST RESULTS

4.1 *Load testing*

The variation of average shear stress with pile head displacement measured in select first time tension tests and retests on the Bayswater piles is shown in Fig. 7. The variation with time since installation of the peak average shaft friction (τ_p) inferred for each test is plotted in Fig. 8. Note that these τ_p values can be assumed to correspond with actual peak values – as the piles are relatively rigid according to the stiffness factor derived using the criteria in Randolph & Murphy (1985).

The general trends observed are as follows:

(i) The time factors given by Teh & Houlsby (1991) combined with the piezocone c_h values indicate that the degree of consolidation following pile installation is likely at 93% after 9 days (and hence essentially complete). Following consolidation, peak average frictions show an irregular but clear trend to increase with time. The peak friction 217 days after installation is 16% higher than that 28 days after installation.

(ii) As the tests are load controlled, an accurate inference of the degree of brittleness cannot be made. It is clear, however, that a brittle response is observed. Peak friction (τ_p) is recorded at displacements between 5 mm and 10 mm, while ultimate friction (τ_{ult}) requires displacement of more than 30 mm to fully develop.

(iii) Ultimate shaft friction appears to show a relatively minor dependence on time, suggesting that ageing is related to the development of a stronger bonded layer around the pile – with the high peak strengths subsequently lost by large shearing distortions induced post-peak. Displacement control testing would be required to confirm this observation.

(iv) Although there is some scatter in the data, there is no clear trend for the pile axial stiffness to increase with time.

(v) The peak shear stresses for the re-tested piles (P4 and P6) are similar to those seen in corresponding first time tests. This observation contrasts with tests reported in NGI (2013) in low or medium plasticity clays, which showed consistently higher capacities on reloading.

(vi) The peak shaft frictions of the galvanized piles are only slightly lower than those of the mild steel piles, suggesting that the galvanization process did not inhibit the creation of a bonded crust around the piles over the long term. Testing with stainless steel piles would provide a better examination of the influence of a more inert steel type.

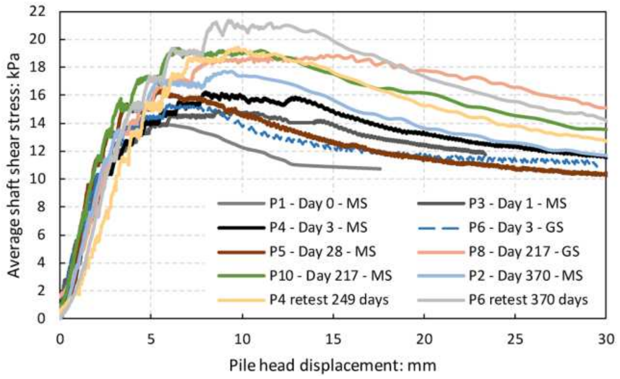

Figure 7. Measured shaft shear stress vs pile head displacement

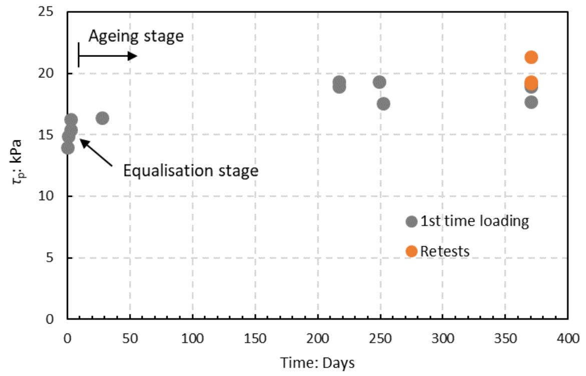

Figure 8. Variation of peak average shaft friction with time.

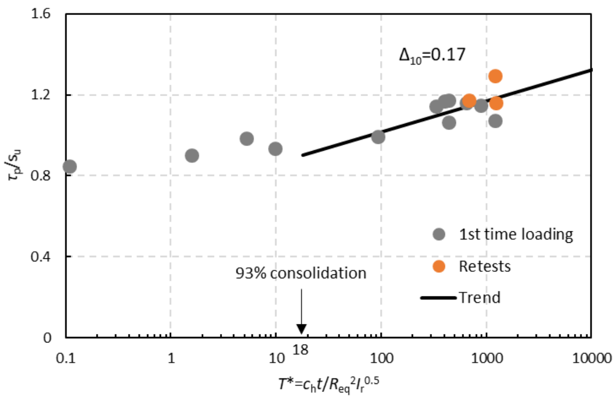

Figure 9. Variation of normalised peak average shaft friction with normalised time.

4.2 *Normalisation of time and shaft resistance*

Randolph (2003) suggested that the dissipation curves for piles of different wall thickness fall within a narrow band, when expressed in terms of time factor T_{eq}, based on the equivalent pile diameter, rather than the true diameter. In the analysis of dissipation around a piezocone, Teh & Houlsby (1991) proposed a 'generalised' time factor T^*, given by $T^* = c_h t/R^2 I_r^{0.5}$. Based on this, the time factor (T^*) is adopted to normalise ageing time in order to get a clearer indication for field pile tests of different diameters – where $T^* = c_h t/R_{eq}^2 I_r^{0.5}$ and $R_{eq}^2 = (R^2 - PLR \cdot R_i^2)$, R and R_i are the outer and inner pile radius respectively, and PLR is the measured plug length ratio after installation. As positive excess pore pressures dissipate, pore water flow radially away from the pile.

An estimate for the rigidity index (I_r) in these equations is not required provided the same I_r value used to determine c_h from piezocone dissipation tests is used to represent consolidation around the pile. For this case $T^* = T^*_{50} [R_{CPT}/R_{eq}]^2 [t/t_{50}]$. For convenience, $I_r = 100$ was adopted in the calculation of c_h for the Bayswater field tests.

The data in Fig. 8 are re-plotted against time factor T^* for field tests in Fig. 9, and in which the measured average peak shaft resistance is also normalized by the average undrained strength. The trend of increasing peak friction with the logarithm of time is clearly evident, with a Δ_{10} value of around 0.17 inferred for the Bayswater clay.

5 COMPARISON OF BAYSWATER AND ONSOY FIELD TESTS

This section provides a brief comparison with pile ageing tests conducted in the lightly consolidated at Onsoy (Norway) and reported by NGI (2013). The pile tests were performed using steel pipe piles with outer diameter of 508 mm, wall thickness of 6.3 mm and embedded length of 17.7 m. First-time tension tests were conducted on piles at ageing period of 78, 162, 237, 369 and 729 days.

Properties of the Onsoy clay over the embedded pile lengths are listed in Table 3. Compared with the Bayswater clay, the liquid limit, plasticity index, clay content and overconsolidation ratio (OCR) of the Onsoy clay are higher than those of Bayswater clay, while the organic content is slightly lower. In this case, the undrained soil strength was determined from simple shear testing.

Table 3. Properties of Onsoy clay over the embedded pile lengths (Gundersen et al., 2019; NGI, 2013 & 2019)

Liquid limit	48~75 %
Plasticity index	24~40 %
Clay content	40~72 %
Organic content	3~4 %
OCR	< 3
Horizontal coefficient of consolidation, c_h	15 m^2/year
Undrained strength, $s_{u,ss}$	22[1] kPa

[1] Average over the depth of interest of the piles.

Like the piles at Bayswater, the piles used at Onsoy are relatively rigid, according to the criterion proposed by Randolph & Murphy (1985). The load-displacement response of piles in both clays is also similar, with both showing clear strain softening – whereby shaft resistance reduces to about 70% of the peak value after pile head displacements of up to 40 mm.

According to dissipation curves given by Teh & Houlsby (1991), the Onsoy piles require roughly 100 days to reach 93% consolidation. This is comparable to the reference time (t_0) of 100 days often adopted to indicate a representative time for complete equalisation of full-scale displacement piles (Doherty & Gavin, 2013; NGI, 2013). However, a more rigorous approach is to consider the completion of consolidation by using T^*=18 as the reference time, as shown in Figure 9.

Normalised data reported in NGI (2013) are replotted in Fig. 10 alongside the Bayswater test results. They are shown to have a consistent trend with Δ_{10} = 0.17, which broadly aligns with the value of 0.16 reported in NGI (2013).

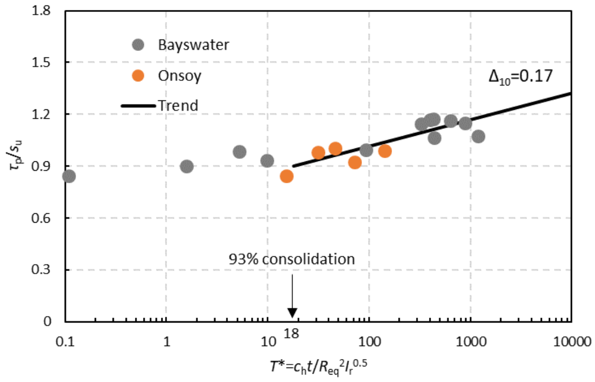

Figure 10. Variation of normalised peak average shaft friction with normalised time (first time loading only).

6 CONCLUSIONS

The drivers behind pile ageing in clay remain unknown, although are believed to be strongly controlled by the clay type and the environment within which the clay, water and steel interact. Consolidation related gains in shaft friction are clearly an important source of strength enhancement due to increases in radial effective stress, while ageing due to geochemical reactions occurs both during and after this initial consolidation phase.

The field test reported in this paper reveal that ageing effects for shaft friction in Bayswater clay are significant for peak friction, with modest increases in ultimate friction over time. Repeat testing yielded comparable increases in strength, while ageing appears to have little effect on pile stiffness.

7 ACKNOWLEDGEMENTS

The work presented in this paper is part of an ongoing research project supported in part by Shell Brunei, which is gratefully acknowledged. The first author acknowledges the support of the Australian Postgraduate Award at The University of Western Australia. The second author gratefully acknowledges the financial support by Chinese Scholarship Council (201906275010) and Natural Science Foundation of Hubei Province (2019CFB488). The fourth author leads the Shell Chair in Offshore Engineering research team at the University of Western Australia, which is supported by Shell Australia.

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