Detrimental Effects of Lateral Soil Movements on Pile Behaviour

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ABSTRACT: Deep excavation, tunnelling and river tidal fluctuations are some activities that can induce lateral soil movements, which can detrimentally impact nearby existing infrastructure. One major design concern is that the behaviour and mechanisms of complex soil-structure interaction that occur in these situations are often still not well understood. Limited design methods are currently available to evaluate these problems in practice. Therefore, the latest development and understanding of soil-structure interaction involving pile foundations subjected to lateral soil movements are presented with reference to successfully implemented projects and research outcomes based on finite element modelling, centrifuge experiments as well as field observations and interpretations. The novel concept of passive pile behaviour and limiting soil pressure due to stress relief will be evaluated and explained in detail.

KEYWORDS: Limiting soil pressure; Soil movements; Soil-structure interaction; and Riverbank.

1. INTRODUCTION

With the phenomenal increase in population in urban areas, construction works are now often carried out in close proximity to existing buildings and geotechnical structures. As such, many soil-structure interaction problems exist and examples of such problems include the effects of deep basement or tunnel excavation on foundations of adjacent buildings. Nowadays, the problems have become more complex as new structures are often constructed in grounds consisting of very soft soils within very congested sites as the choices of 'good' sites have rapidly diminished in the ever-growing large urban cities.

The soil movements due to nearby excavations may induce additional loading and movement on adjacent foundations. In general, when large soil movements are anticipated in the course of excavation, the construction process would be extensively monitored. Despite extensive instrumentation and monitoring, a good number of foundation damages caused by excavation-induced soil movements have been reported, see for example, Poulos (1997) and Ong et al. (2010). Such damages are often due to much thicker soft soil deposits or considerably lower shear strength of soft soil, which are unforeseen and unexpected at the design stage.

Besides the impact of excavation, soft riverbank slopes subject to tidal fluctuations can also be a source of soil movement. Riverine infrastructure namely jetties, wharves and bridges are commonly supported by pile foundations installed along riverbanks. Owing to lack of in-depth technical knowledge on this complex soil-structure interaction issue, increasing numbers of damages and failures on riverine infrastructure have been reported in Sarawak, Malaysia, see for example Ting (1997), Ting & Tan (1997) and Ting (2004).

Failures often cause significant human, financial and time loss and are also very difficult to remedy. One major design concern is that the behaviour and mechanism of complex soil-structure interaction problems are still not well understood. Very few design methods are currently available to evaluate these problems in practice.

The use of sophisticated finite element computer program would substantially help the design process of such complex soil-structure interaction problems as long as the validity of the soil models, parameters employed and boundary conditions are justifiable. Alternatively, physical modelling can offer an attractive method to further understand the behaviour and mechanism of complex soil-structure interaction problems. However, the results of conventional laboratory small-scale model tests cannot be extrapolated to prototype scale as the behaviour of soils is stress dependent. The use of centrifuge modelling technique would overcome this shortcoming. In his Rankine Lecture, Schofield (1980) highlighted the useful applications of centrifuge modelling technique to study geotechnical problems. In this paper, the understanding of soil-structure interaction caused by adjacent excavation are presented with references to successfully implemented projects or research work based on finite element modelling, centrifuge experiments and field monitoring, observations and interpretations.

The following soil-structure phenomena are addressed in this paper:

(i) Effects of limiting soil pressure due to excavation,
(ii) Effects of soil-structure interaction adjacent to deep excavations
(iii) Effects of soil-structure interaction adjacent to creeping riverbanks.

2. EFFECTS OF LIMITING SOIL PRESSURE DUE TO EXCAVATION

2.1 Case Study 1: Centrifuge study in soft clay

Details of a test involving a failed excavation in clay has been reported in Ong et al. (2004). Figure 1 shows the centrifuge model set-up for the study.

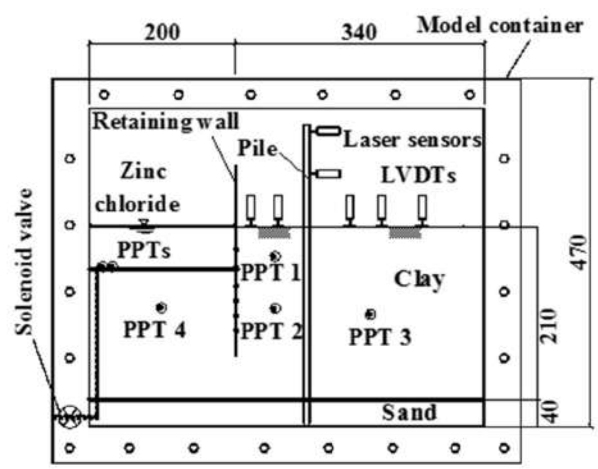

Figure 1 Centrifuge model set-up (all dimensions in mm) (Ong et al. 2003)

In this test, much of the retaining wall is embedded in the soft kaolin clay layer. The maximum excavation depth is 1.8m. The single pile is located 3m behind the wall. In-flight bar penetrometer tests were performed to quantify the undrained shear strength (c_u)

profile of the clay before the test. The shear strength profile shown in Figure 2 reveals that a 2.5m thick overconsolidated crust exists above the normally consolidated clay.

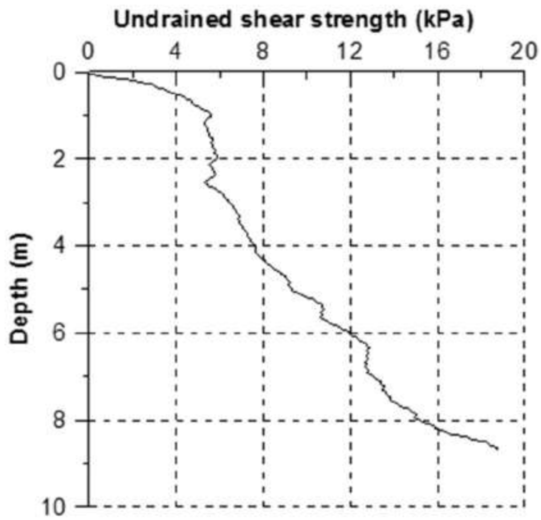

Figure 2 Undrained shear strength profile of clay (Ong et al. 2004)

Figure 3 shows the wall deflection profiles during and after excavation. The tilted wall causes the clay behind the wall to settle and the ground settlement continues to increase over time after the completion of excavation, as shown in Figure 4. The long term time dependent wall deflection and settlement troughs have been further investigated in detail by Ong et al. (2003, 2004).

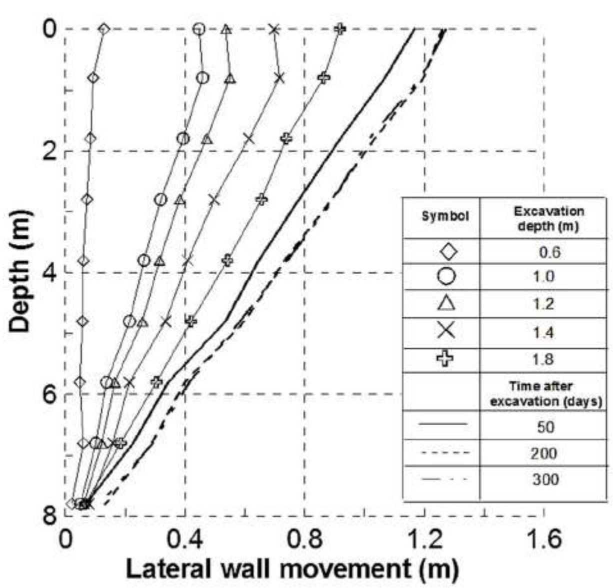

Figure 3 Wall deflection profiles during and after excavation (Ong et al. 2004)

Figure 5 shows that the maximum induced pile bending moment is located at 8.75 m below the ground level. The induced bending moment initially increases with increase in excavation depth. A maximum value of 236 kNm is recorded at an excavation depth of 1.2 m. The bending moment then decreases with increase in excavation depth. At the maximum excavation depth of 1.8 m, the bending moment reduces to 185.8 kNm. Thereafter, the bending moment profile is found to decrease further over time.

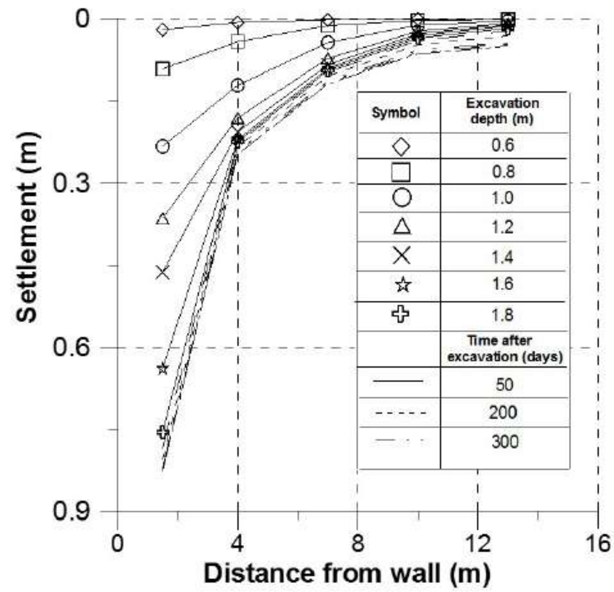

Figure 4 Ground settlement profiles during and after excavation (Ong et al. 2004)

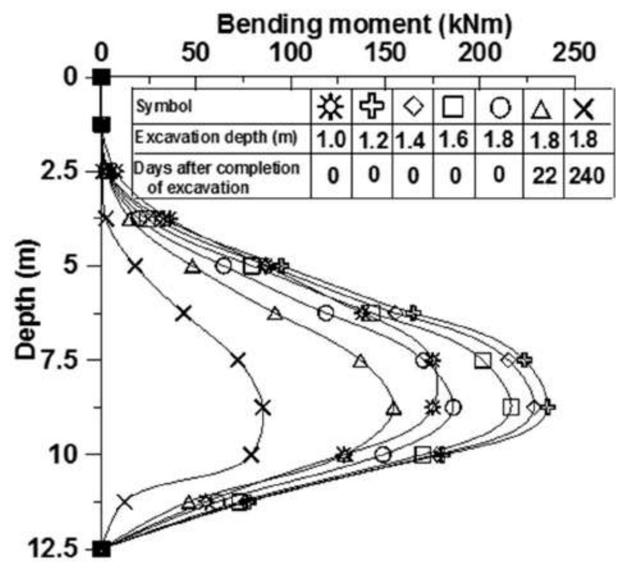

Figure 5 Development of pile bending moment profile over time (Ong et al. 2004)

High-resolution photographs were taken during various excavation stages of the test, as shown in the left-hand side photographs of Figure 6. It is evident that tension cracks have developed when the excavation depth exceeds 1.0 m.

These cracks cause the loss of contact of clay in front of the pile and may have prevented the transmission of additional soil pressures onto the pile. The vectors of soil movement shown in the right-hand side plots of Figure 6 indicate that the size of considerable soil movement zone increases as excavation progresses, but the shape does not alter. This observation is consistent with that observed by Bolton et al. (1987).

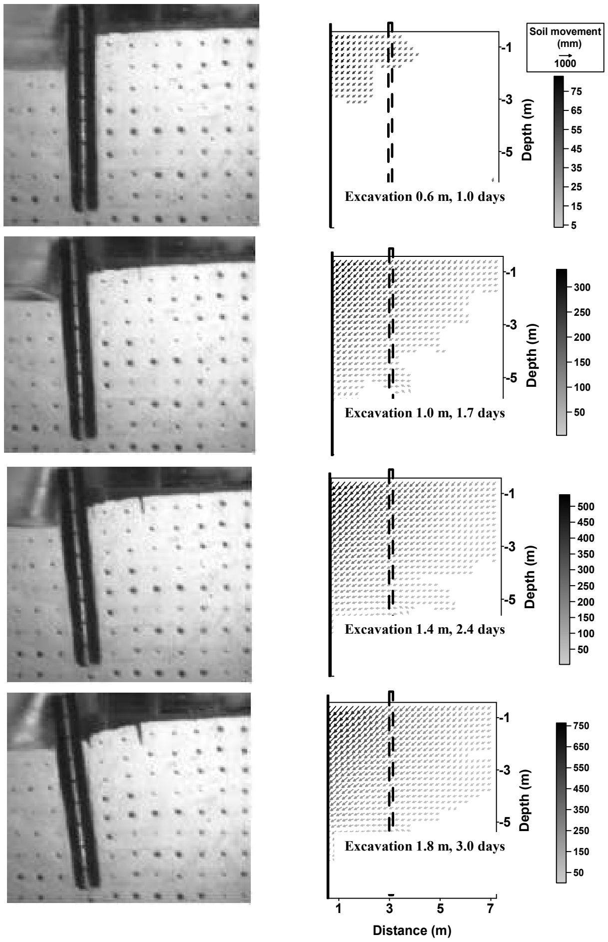

Figure 6 Pictures and vectors showing development of tension cracks and corresponding soil movements, respectively (Leung et al. 2006)

The variations of pile head deflection and free field soil movement at different depths at the pile location with time are shown in Figure 7. The free field soil movement is measured by using a commercial image processing software to track the movement of beads placed on the side surface of the clay.

It is observed that the soil starts to move ahead or "flow" past the pile at a relatively shallow excavation depth of 0.6 m. After which the difference between the soil and pile movements becomes more significant with increasing excavation depth. The movement is expected to be reasonably large during excavation due to the low undrained shear strength profile of the clay as shown in Figure 2. As expected, greater soil movement is observed to occur nearer to the ground surface.

In order to verify this finding, the soil pressure profiles are obtained by differentiating the measured pile bending moment profiles twice using a 7th order polynomial. Figure 8 shows the development of the maximum soil pressure deduced from the corresponding bending moment profiles shown in Figure 5. It is evident that the limiting maximum soil pressure values have been reached at an excavation depth of 1.2 m.

Thereafter, the soil pressures do not increase further with increasing excavation depth. This observation further reinforces the postulation that when the soil flows past the pile. In addition, with the presence of the tension cracks in front of the pile as described earlier, the soil could not transmit its full pressure onto the pile, resulting in a drop in induced pile bending moment as shown in Figure 5.

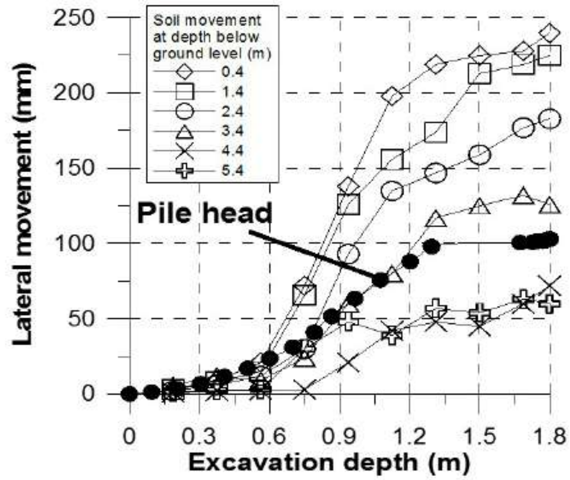

Figure 7 Variations of pile head deflection and soil movement (Ong et al. 2004)

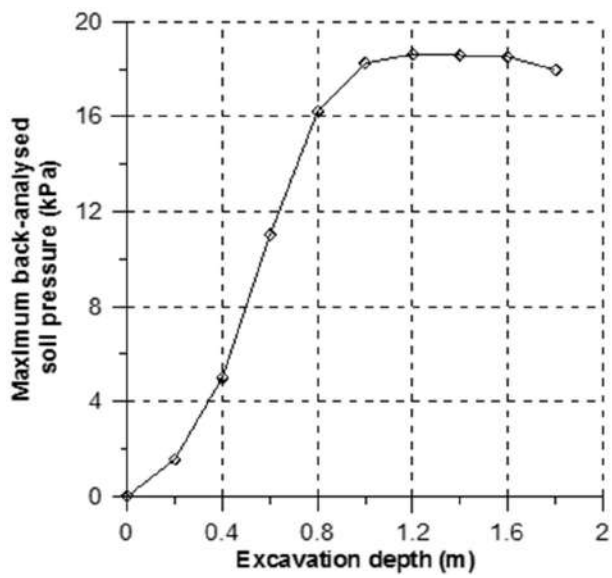

Figure 8 Variation of maximum back-analysed soil pressure with excavation depth (Ong et al. 2004)

2.2 Case study 2: Field study in soft clay

Ong et al. (2004, 2011) reported a case study where a proposed 7-storey industrial building with one-level basement car park was to be constructed at a congested site in the city. In order to construct an underground storage facility, a temporary open-cut excavation of a 1V:2.5H slope was proposed in front of the capped 4-pile group of 900-mm diameter cast-in-situ concrete bored piles. Owing to some unforeseen situations, excessive soil movement had taken place during the slope excavation and caused failure of the instrumented pile group. The post-failure pile behaviour has provided valuable field data for back-analysis.

Table 1 shows the various analysis cases performed so as to simulate the understanding level of a designer when confronted with such a case study.

Method 1 considers application of 2D FE analysis by smearing of 3-D pile properties (Ong et al. 2007, 2011), while Method 2 involves the use of an established numerical method described in detailed in Leung et al. (2006).

Table 1 Various analysis cases performed (Ong et al. 2010)

Analysis cases	**I_g of I_{cr}**	**p_y**
Case 1 (Method 1): simulates ignorance of soil flow phenomenon	I_g	Not considered
Case 2 (Method 2): simulates available knowledge on I and p_y	I_{cr}	p_y=6c_u
Case 3 (Method 2): simulates available knowledge on p_y but not on I	I_g	p_y=6c_u
Case 4 (Method 2): simulates absence of knowledge on I and p_y	I_g	p_y=K_h

Note: I_g=gross moment of inertia, I_{cr}=fully cracked moment of inertia, p_y=limiting soil pressure, c_u=undrained shear strength, K_h=soil spring stiffness

Figure 9(a) shows the outcome of the pile responses for Case 1 based on Method 1. The front pile is located nearer to the slope than the rear pile. In this case, the natural behaviour of soil deformation is negated by the presence of the 'equivalent wall' (no longer a pile) in a 2-D environment, resulting in relatively smaller magnitudes than measurements taken on site. Consequently, the predicted pile responses (bending moment and deflection) are both very much under-predicted, leading to inappropriate design of pile to resist lateral soil movement.

In Case 2, if both I_{cr} and p_y are correctly adopted, Figure 9(b) shows that the prediction of pile responses is very reasonable.

This simulates the available and appropriate level of understanding of the back-analysis carried out considering the development on site.

Case 3 simulates the situation where knowledge on limiting soil pressure is available but not on the pile moment of inertia, *I*.

In such a case, the pile bending moment tends to be over-predicted, but the deflection is under-predicted as shown in Figure 9(c).

This is due to the pile being assumed to be uncracked (much stiffer) thus attracting high bending moment and low deflection. The above does not simulate the behaviour on site where the pile has experienced lateral movement of more than 100mm. This highlights the importance of knowing the condition of the pile on site when performing back-analysis.

In Case 4, if the back-analysis is carried out without having prior knowledge of estimating limiting soil pressure and transformed pile moment of inertia, *I* on site due to cracking, the predicted pile bending moment will be grossly over-predicted as shown in Figure 9(d). However, in this case, the 'reasonable' estimation of pile deflection is merely a coincidence.

3.0 EFFECTS OF SOIL-STRUCTURE INTERACTION ADJACENT TO DEEP FOUNDATIONS

3.1 Case study 1: Pile group study in soft clay behind a stable retaining wall

Ong et al. (2009, 2011) presented a series of 8 centrifuge tests for free and capped heads of 2-, 4- and 6-pile groups adjacent to an excavation in very soft clay behind a wall that remained stable after excavation. Therefore, limiting soil pressure is not expected to be reached. In these cases, soil-pile interactions such as shadowing, reinforcing and soil arching effects are studied in detail. The centrifuge pile group test set-ups are identical to the single pile set-ups as reported by Ong et al. (2006) except that structural pile caps are introduced to tie the individual pile heads for cases with capped pile head conditions.

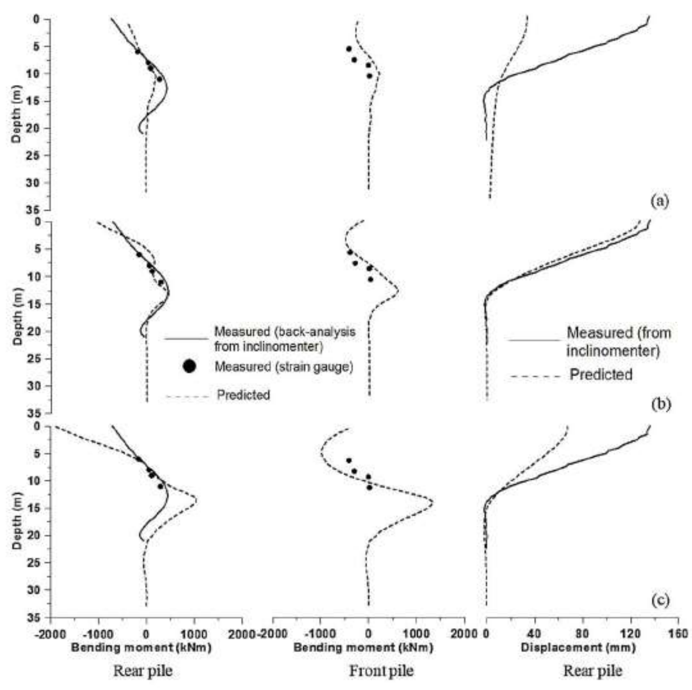

Figure 9 Profiles of measured and predicted rear and front pile bending moment and rear pile deflection for Cases (a) 1, (b) 2 and (c) 3 (Ong et al. 2010)

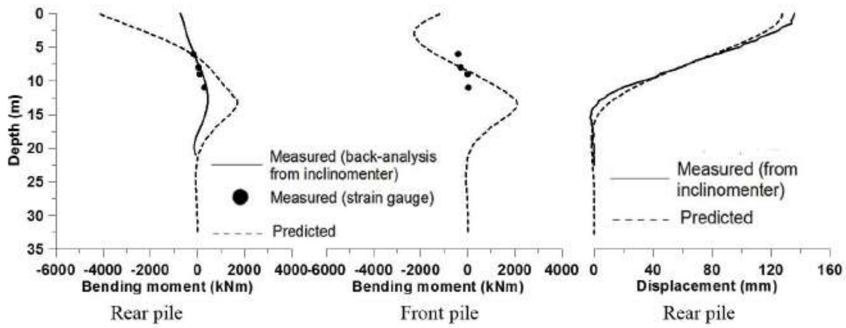

Figure 9(d) Profiles of measured and predicted rear and front pile bending moment and rear pile deflection for Case 4

It is found that the induced maximum bending moment is always smaller than that of a corresponding single pile (Ong et al., 2006) at an identical location. If the free-head piles are located at the same distance, the measured bending moment is higher for the front pile as opposed to the rear pile of the pile group. In a pile group, each individual front pile (3m behind the wall) will provide shadowing and reinforcing effects to the other the rear piles (5m behind the wall), thus reducing the magnitudes of pile deflection and bending moment. This is evident from Figures 10 and 11.

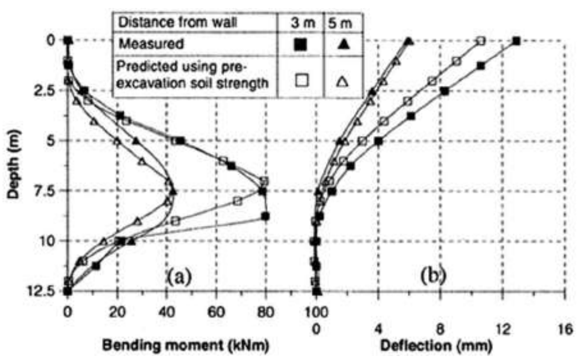

Figure 10 Predicted and measured pile (a) bending moment and (b) deflection profiles for free-head 4-pile group (Test 12) (Ong et al. 2009, 2011)

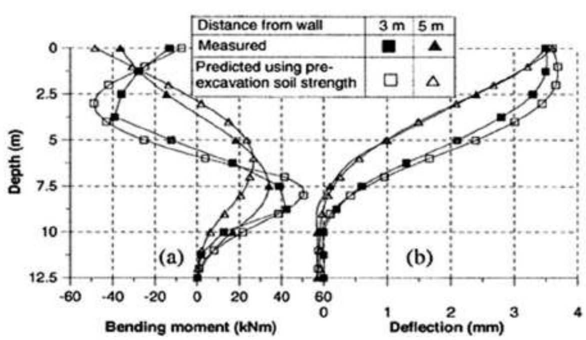

Figure 11 Predicted and measured pile (a) bending moment and (b) deflection profiles for capped-head 4-pile group (Test 13) (Ong et al. 2009, 2011)

The degree of shadowing experienced by each individual pile depends on its relative position with its surrounding piles. It is observed that the induced bending moment for the front peripheral pile is greater than that of the front centre pile at the same distance behind the wall. Similarly, the bending moment developed at the rear peripheral piles is also greater than that of the rear centre pile at the same distance behind the wall. As the number of piles in a group becomes larger, the shadowing and reinforcing effects become more prominent.

The immediate effect of pile shadowing and reinforcing effect is to reduce the detrimental effects of excavation-induced soil movement on the pile group. By capping a pile group, the individual pile heads are forced to act in unison when subject to different magnitudes of soil movement, depending on the distance of the piles from the wall.

The induced bending moment of the front pile, which experiences a greater soil movement, is moderated by the rear pile through the pile cap. The interaction between the front and rear piles induces negative bending moment at the pile head, but reduces the magnitude of bending moment developed along the pile and the pile group deflection as observed in Figure 11.

Soil arching and "separation" of soil as shown in Figure 12, have been observed to occur between the front piles of a pile group when the soil moves upon excavation, in the 4-pile group. The arch is formed between the rows of piles when the yielded soil gets detached from its surrounding. The detached soil is then forced to squeeze between the row of piles but without significantly increasing the pressure acting on the piles.

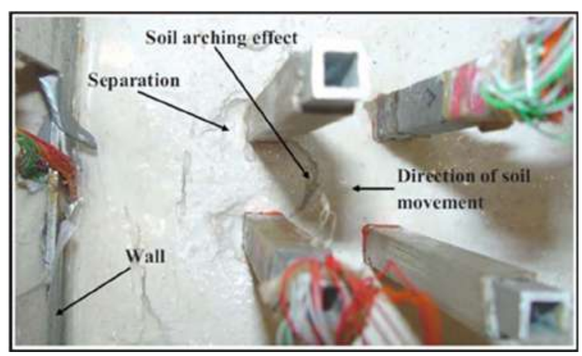

Figure 12 Soil arching and separation observed in Test 12 (Leung et al. 2006)

Generally, the observed long term maximum positive bending moment would increase after excavation until about 50 days later and subsequently reduce with time. It is believed that progressive wall and soil deformations are the reasons for such observed time dependent pile behaviour. On the contrary, the maximum negative bending moment generally reduces slightly over time after excavation. This behaviour could be the result of pile-pile cap interaction as the maximum negative pile bending moment is located nearer to the pile cap.

To account for the pile group shadowing and reinforcing effects, an empirical soil moderation factor, k_s was introduced to "correct" the measured free-field soil movements. By back-analysis, the magnitude of soil moderation factor, k_s is established to be 0.8 for a 2-pile group, 0.7 for a 4-pile group and 0.5 for a 6-pile group in clay as compared to 0.9 for a 2-pile group, 0.8 for a 4-pile group and 0.6 for a 6-pile group in sand as reported by Leung et al. (2003).

3.2 Case study 2: Pile group damage during excavation

During a rain storm, a slope failure resulted in excessive soil movement near the excavation for the construction of an underground basement in Singapore. The instrumented 900mm bored pile group constructed nearby the excavation failed due to excessive soil movements. The failure of the instrumented pile group was defined by relatively large pile deflections due to the lateral soil movements occurred from the slope failure.

3.2.1 Soil Profile

Figure 13 shows an interpretation of the subsurface soil profile and geotechnical properties. Standard penetration tests (SPTs) and situ vane shear tests were conducted to study the underlying soft marine clay. Using a standpipe piezometer, the groundwater level was found to be 1m below the ground level before excavation started. Below the 1m thick fill, a greenish grey marine clay of 9m thick followed by 2.5m thick Old Alluvium which consisted of loose clayey sand and medium-dense sand. Furthermore, a 9.5m thick dense clayey sand was found after which the borehole was terminated in very dense clayey sand.

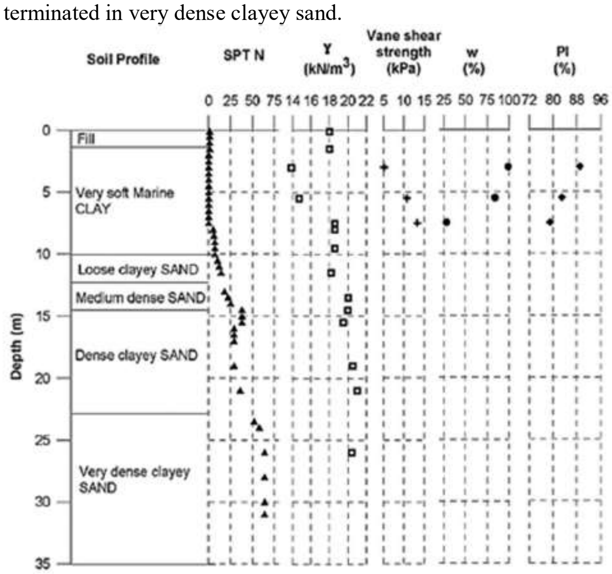

Figure 13 Interpreted subsurface soil profile at site (Ong et al. 2015)

3.2.2 Instrumentation Program and Layout

Expecting the excavation in front of the piles, a pile group consisting of four 900mm cast-in-place concrete bored piles were instrumented to measure the bending moment along the pile during and after excavation. Two instrumented piles, one near to the excavation and the second one further away from excavation, were instrumented. Figure 14 shows the elevation and plan views of the instrumented piles with respect to the slope excavation.

A pile with minimum reinforcement of 0.5% steel can be categorised as lightly reinforced pile. For lightly reinforced concrete structures, the contribution of reinforcement can be neglected for the gross moment of inertia, I_g, for the section (Branson, 1977). Kong and Evans (1987) developed a reliable method to determine the cracked moment of inertia, I_{cr}, for a rectangular beam section and hence, it is assumed that I_{cr} for a circular bored pile can be similarly represented based on the principle of conservation of cross-sectional area. The stresses in steel and concrete are assumed to be proportional to strain for a fully cracked section.

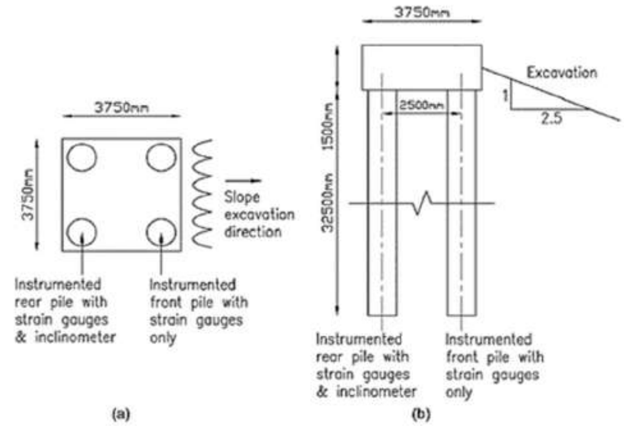

Figure 14 (a) Plan and (b) elevation views of instrumented pile group with respect to the slope excavation (Ong et al. 2015)

3.2.3 Moment of Inertia

Bending moment occurs when a pile is subjected to lateral soil movement. The depth and width of a crack may vary if the bending moment exceeds the cracking moment, based on the final bending moment distribution along the pile. Hence, the moment of inertia for such particular cracked section is between I_g and I_{cr} values. Therefore, an effective moment of inertia, I_e, is required to analyse the particular cracked section.

Figure 15 shows several computed profiles of I_e values with respect to the rear pile length when subjected to an increase of lateral soil movements at various construction stages.

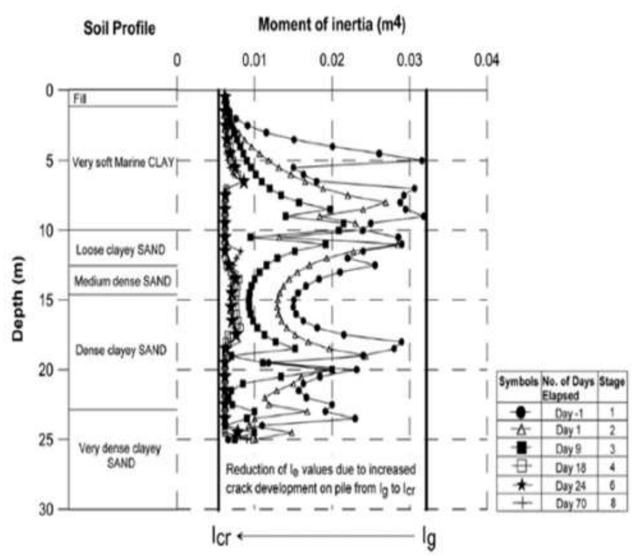

Figure 15 Computed profiles of effective moment of inertia, I_e, along the instrumented rear pile over the excavation period (Ong et al. 2015)

On Day 0, the 3.5m deep excavated slope failed in front of the pile group, causing an increase in pile lateral deflections up to a depth of approximately 13m. This depth matches with the soft marine clay and loose clayey sand strata. Dense sand was found below 13m and the lateral soil movements recorded were reduced.

The largest rate of reduction of I_e values with respect to depth was about 6m deep, which matches with the final excavation depth in front of the instrumented pile.

The development of pile cracks within this depth increased with increasing excavation depth due to increasing lateral soft clay. Negative deflection was observed for the rear piles within the dense and very dense sand strata. The negative deflection of the pile has an average magnitude of 2.6mm with a maximum value of 3.8mm. The cracks seemed to have just been initiated along the pile, probably due to mobilization of the passive resistances to resist the increase in lateral soil movements experienced by the upper part of the pile.

3.2.4 Observation of Different Degrees of Cracking along Pile Length

Figure 16 shows the different degrees of cracking along the pile length.

The ratio of back-analysed bending moment, *M*, and the cracking moment, M_{cr}, is plotted versus the ratio of measured deflection, *D*, and the initial deflection at the onset of cracking, D_i, during different stages for the back-analysed pile responses between Day 0 and Day 37.

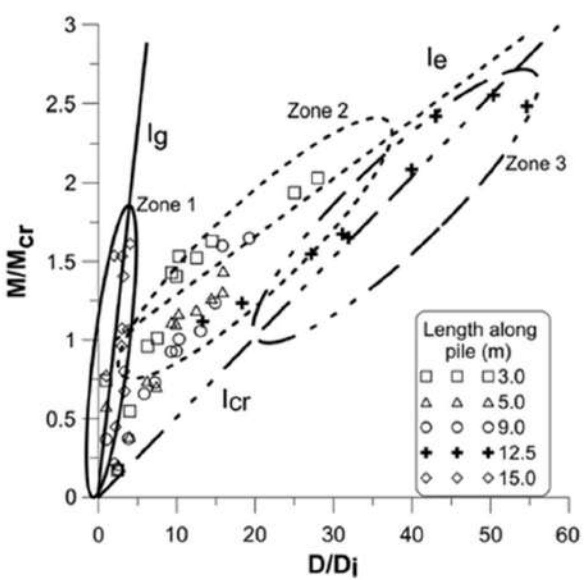

Figure 16 Interpreted bilinear moment-deflection curve (Ong et al. 2015)

Three distinct zones can be clearly identified from Figure 16. Zone 1 can be defined as the zone where the pile experiences minimal cracking. The measured data points at 15 m depth with dense clayey sand layer produced N value ranging between 29 and 50. The pile is prevented from deflecting excessively due to the higher resistance obtained by the sand and the reduced lateral soil movements.

Zone 2 shows the data points at depths of 3, 5 and 9 m which are found between the two extreme Zones 1 and 3. These depths consist of soft marine clay with little soil resistance as opposed to those of sand in Zone 1. For this zone, it is found that cracking is ongoing.

Zone 3 shows the measured data points for the pile responses at depth of 12.5 m. This depth matches with the soil interface defined as the maximum pile bending moment developed. Hence, as opposed to Zone 1 and 2, the ratio of M/M_{cr} and D/D_i are greatest in Zone 3.

Similar trends of the pile behaviour at different degrees of cracking as shown in Figure 16 were reported by Branson (1977). The bilinear moment-deflection curve recommends that the pile moment capacity reduces with an increase in load levels and the pile deflection increases together with increasing degree of cracking of the pipe material. This will change the relevant section of the pile from an initial uncracked pile (Zone 1) with minimal cracking to an intermediately cracked segment (Zone 2) and lastly to a fully cracked pile section (Zone 3) where substantial cracking has occurred.

4.0 EFFECTS OF SOIL-STRUCTURE INTERACTION ADJACENT TO CREEPING RIVERBANKS

Ting (1997) and Ting & Tan (1997) have documented increasing numbers of damages and failures on riverine infrastructure in Sarawak, Malaysia. The failures have translated the unnecessary risks onto the public besides requiring high costs for any eventual remedial works. Therefore, greater understanding on pile-soil behaviour caused by tidal fluctuations should be developed in order to achieve a more sustainable design methodology for riverbank infrastructure. In this paper, the behaviour of individual pile embedded at different locations in the riverbank slope has been investigated and shall be discussed in detail.

Goh et al. (1997) highlighted that the induced lateral soil movements due to surcharge from a nearby embankment damaged adjacent piles. They developed a numerical model based on the finite element method for analyzing the response of single piles subjected to lateral soil movement and verified the results against full scale tests and centrifuge model tests. Stewart et al. (1994) also stated that piles supporting bridge abutments on soft clay would be loaded laterally due to the lateral soil movements derived from the construction of the approaching embankments.

Ong et al. (2006) investigated the behaviour of single pile subjected to excavation induced soil movements behind a stable retaining wall by using centrifuge model tests. They reported that soil movements induced further bending moment and deflection on the adjacent pile after the excavation.

As discussed earlier, much research was conducted on the pile-soil interaction without considering the effect of repeated loading that can induce soil movements. The behaviour of individual piles subjected to soil movement as a result of tidal fluctuation (repeated loading), will be studied using the geotechnical centrifuge.

4.1 Model pile and soil characteristics

The centrifuge model setup is shown in Figure 17 and all tests were conducted at 50-g on the National University of Singapore (NUS) geotechnical centrifuge. The instrumented model pile was fabricated from a hollow square aluminium tube with an outer dimension of 10mm and a wall thickness of 1.00mm. Ten pairs of strain gauges in bending configuration were glued at opposite faces of the model pile at vertical intervals 25 mm. The final width of the model pile is 13.0mm (650mm in prototype scale; unless otherwise stated the number in the bracket will be referring to the prototype scale). The total length of the pile is 350mm (17.5m) with an embedment depth of 250mm (12.5m) and 300mm (15.0m) for the pile in mid-slope and pile in crest, respectively. The prototype bending rigidity, *EI*, of the model pile is approximately 2.2×10^5 kNm^2. This is equivalent to a 600mm diameter grade 35 concrete bored pile. Malaysian kaolin with properties shown in Table 2 was used in the model tests.

Table 2 Properties of Malaysian kaolin KM25

Sample	LL (%)	PL (%)	PI (%)	Soil Classification
KM25	59	48	11	MH

From the centrifuge scaling law, the model time scale is $1/N^2$ (N is 50) of the prototype scale. The duration of a cycle of 5 m water fluctuation is 4 min (6.94 days). The results from the two tests will be discussed in terms of soil lateral movements quantified based on pile bending moments, pile head movement and pore water pressure changes.

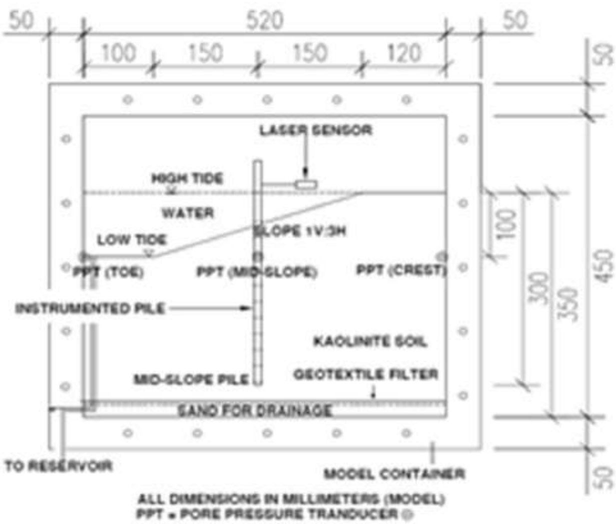

Figure 17 Centrifuge model setup for mid-slope and crest piles (Wong et al. 2016)

4.2 Lateral soil movement based on particle image velocimetry (PIV)

As shown in Figure 18, the typical vector plot was generated using GeoPIV software developed by White and Take (2002).

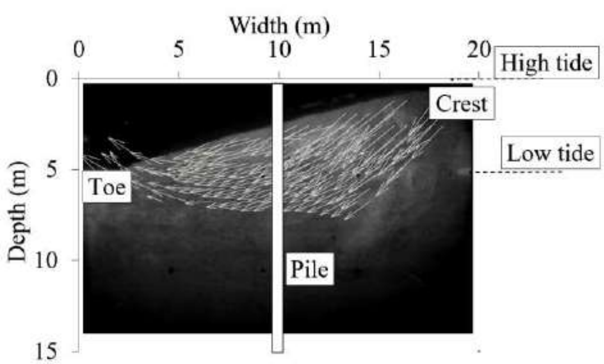

Figure 18 GeoPIV vector plot at 50th cycle of 5 m water fluctuation (Wong et al. 2016)

It can be seen that larger magnitude of soil movements occurred at the mid-slope during fluctuation. Figure 19 shows the lateral soil movement profiles at mid-slope over the tidal cycles.

In addition, Figure 20 illustrates the rates of lateral soil movements with increasing tidal cycles. The rate of mid-slope lateral soil movement (458.7 mm/cycle) is the largest during the first cycle of low tide at 0.51 m below the ground surface. The rate of the mid-slope lateral soil movements decrease with increasing number of cycles as shown in Figure 20 (from 458.7 mm/cycle to 10.2 mm/cycle at 0.51m below mid-slope ground surface). Interestingly, the rate of the lateral soil movement does not seem to approach zero as illustrated in Figure 20. This indicates that the lateral soil movement may be continuing 'indefinitely'.

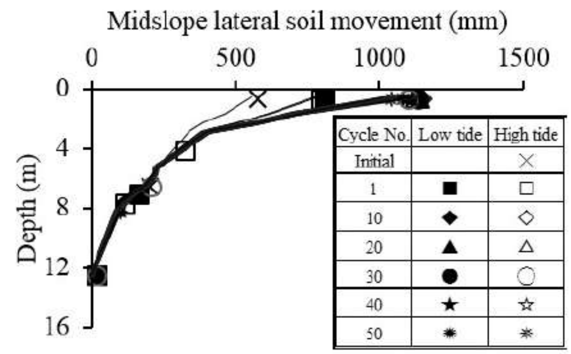

Figure 19 Mid-slope lateral soil movement (Wong et al. 2016)

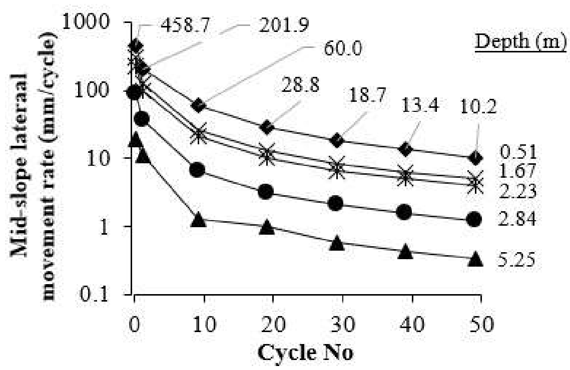

Figure 20 Rate of mid-slope lateral soil movement at different soil depths (Wong et al. 2016)

4.3 Pile bending moments

The mid-slope bending moment profiles over 50 cycles of 5 m water fluctuations are shown in Figure 21. The development of pile bending moment profiles against the number of cycle of tidal fluctuations is plotted in Figure 22. The maximum pile bending moments occurs at depth of 4.25m. As shown in Figure 7, the pile bending moment built up significantly at the low tide of first tidal cycle. The recorded maximum pile bending moment is 432.7 kNm. Subsequently, the pile bending moment profiles (see Figure 21) and maximum pile bending moment (see Figure 22) reduce as the number of cycles increase.

The maximum pile bending moment during the low tide of the first tidal cycle was due to the model soil being in undrained condition during the first tidal cycle. In other words, the model pile was subjected to higher total soil lateral stress during the first low tide. The pile bending moment decreased at all depths along the pile as the number of cycles increased. The reduction of the pile bending moment in the subsequent cycles after the occurrence of the maximum pile bending moment was due to the model soil behaviour approaching the drained condition. This observation will be discussed in detail later with reference to the development of excess pore water pressure generated. As such, the model pile was subjected to the effective lateral soil pressure, which was lower compared to the total undrained soil pressure.

Another noteworthy observation is that the magnitudes of pile bending moment and lateral soil movement at low tides are smaller than the magnitudes at high tide. This is most likely due to the riverbank being partially supported by the water body during high tide. Hence, the magnitudes of lateral soil movement and pile bending moment reduce during high tide.

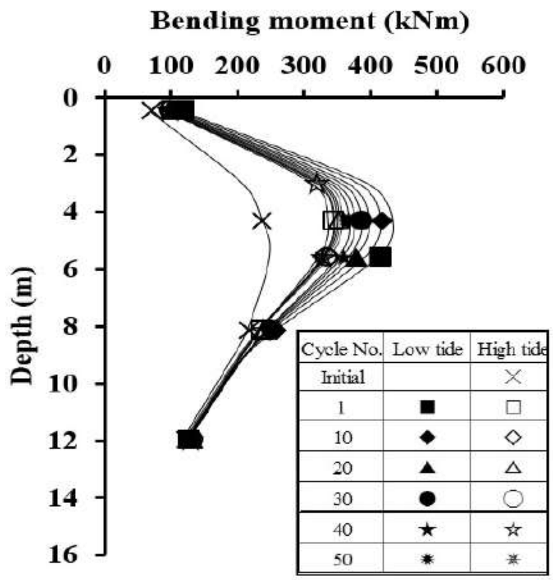

Figure 21 Mid-slope pile bending moment profiles over 50 tidal cycles (Wong et al. 2016)

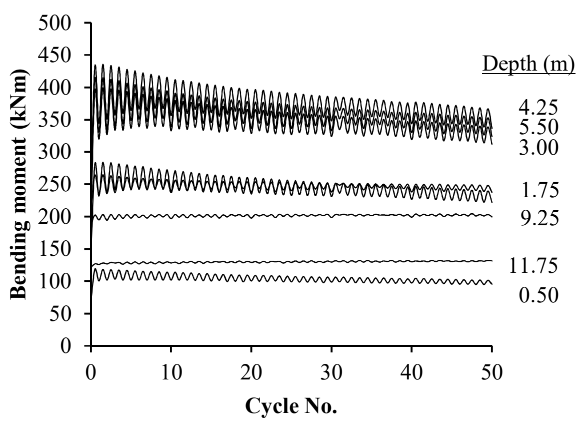

Figure 22 Changes of mid-slope pile bending moment at different depths over 50 tidal cycles (Wong et al. 2016)

4.4 Pile head movements

The pile head movement was recorded using the non-contact laser transducer (Wenglor-CP24MHT80) with working range between 40mm and 160mm (prototype). The outcome is plotted in Figure 23. The pile shows increasing pile head movements with respect to the increasing number of tidal cycles. The largest pile head movement of 1,677mm was recorded during the low tide of the 50th cycle of water fluctuation. Figure 24 shows the rates of lateral pile head movement of the mid-slope single pile. It can be seen that the rate is the largest during the initial cycle (3301.1 mm/cycle), which subsequently decreases with increasing number of tidal cycles. It is worthy to highlight that the rates of pile head lateral movements do not seem to approach zero as the number of tidal cycle increase. The last recorded rate was 0.48 mm/cycle at the 50th tidal cycle. The progressive increase in pile head movements due to the 'creeping' rate of lateral soil movement (see Figure 24), indicates that the piles may experience serviceability issues in the long-term.

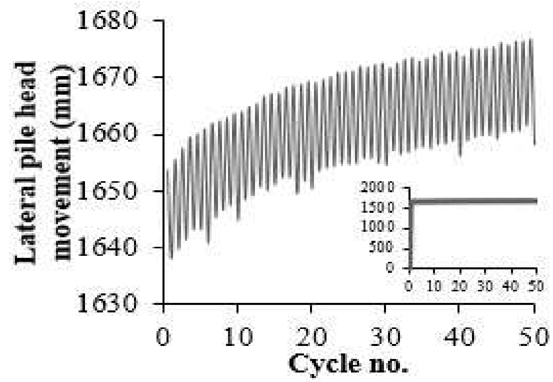

Figure 23 Mid-slope pile head movement over 50 tidal cycles (Wong et al. 2016)

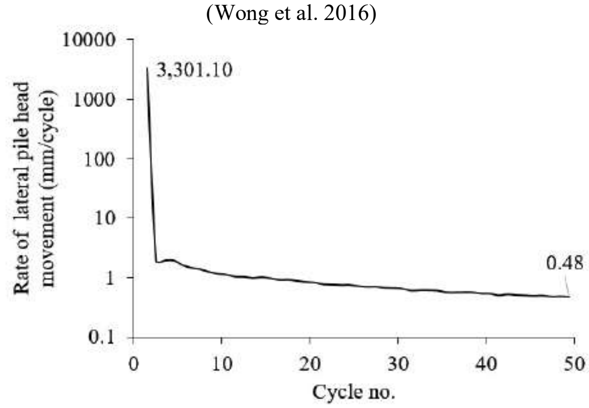

Figure 24 Rates of lateral pile head movements over 50 tidal cycles (Wong et al. 2016)

This observation indicates that such single free-head pile behaviour displays time dependent characteristics as similarly observed by Ong et al. (2006).

4.5 Pore pressure responses

The layout of the PPTs is shown in Figure 17. The excess pore water pressure variations are shown in Figure 25. For the pore pressure transducer (PPT) located at mid-slope, its measured excess pore water pressures decreased with increasing cycles of tidal fluctuation. During the tidal fluctuation, the crest PPT shows only slight decrease in the excess pore water pressures when compared to the mid-slope PPT. This implies that the slope crest is mainly in undrained condition.

At the mid-slope, the PPT shows a continuous decrease in excess pore water pressure over time. This implies that the soil effective stress increases over time. The increase in soil effective stress and thus its shear strength, is believed to have caused the magnitudes of the pile bending moment to reduce over time, which is evidently shown in Figures 21 and 22.

However, as discussed previously, it is worthy to highlight that the rates of pile head lateral movements do not seem to approach zero as the number of tidal cycle increases due to the 'creeping' rate of lateral soil movement (see Figure 24). This observation suggests that the piles may experience serviceability issues in the long-term.

Geotechnical centrifuge model test has been conducted to investigate the behaviour of individual free-head single pile subjected to repetitive soil movements due to 5 m cycles of tidal fluctuations.

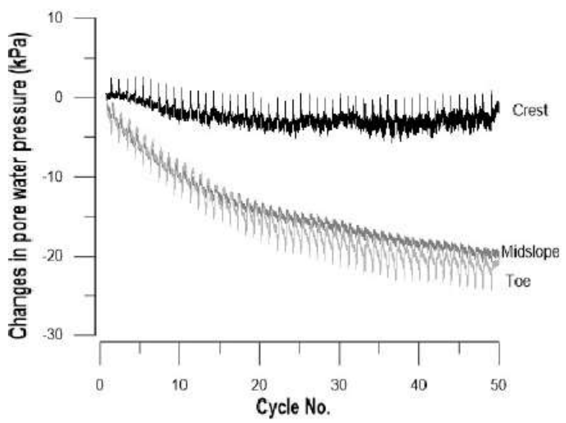

Figure 25 Excess pore water pressure variations over time (Wong et al. 2016)

The pile bending moments increased significantly at the first low tide as the soil body was in an undrained condition and thus, the lateral soil pressures acting on pile was the largest. Then, the magnitudes of the pile bending moment reduce over time due to the excess pore water pressures being dissipated over time and caused the increase in the effective soil strength. In short, the pile bending moment is more critical in the short-term undrained condition.

4.6 Summary

The cumulative pile head movement is observed to increase with the number of tidal fluctuations, even though the rates of pile head movement reduce over time. The rates of the pile head movement do not approach zero and this indicates that the free-head single pile may experience long-term serviceability problem during drained condition.

5.0 CONCLUSIONS

In this paper, the latest development and understanding of soil-structure interaction caused by adjacent excavation are presented with references to successfully implemented projects or research work based on finite element modelling, centrifuge experiments and field monitoring, observations and interpretations.

It is important that the concept of limiting soil pressure due to excavation stress relief is well-understood as it provides a fundamental understanding that has been observed in excavation works performed in soft clay or loose sand. If a pile is positioned within the active wedge failure zone of an adjacent excavation, it will experience a maximum pressure when the soil movements are sufficiently large to cause limiting soil pressures to manifest the behaviour of the surrounding soils. However, as further soil movements take place beyond this threshold situation, no additional soil pressures could further act on the pile, which in turn would not result in no further increase in pile bending moment and deflection.

The effects of soil-structure interaction adjacent to buildings supported on deep foundations could perhaps take more 'punishment' from the detrimental effects associated to excavation works. This is so due to the ability of the installed piles to transfer building loads to deeper and thus more competent soil layers, besides benefitting from the much larger soil confining pressures to mobilise greater soil strength. However, designers have to be aware that lateral soil movements and stress relief due to excavation will cause detrimental pile responses such as additional bending moment and deflection (Ong et al., 2004, 2006, 2009, 2010, 2011 and Leung et al., 2006).

This research has also successfully shown that pile foundations supporting riverine infrastructure on soft riverbank experiencing repeated tidal fluctuations should be designed for strength at short-term undrained condition and for serviceability at long-term drained condition. The development of excess pore water pressures implies that the model soil at the crest is relatively stronger (or more stable) than the section of soil at mid-slope. In summary, pile foundations supporting riverine infrastructure on riverbanks should be designed for both ultimate limit state (designed for strength and stability) as well as serviceability limit state (performance-based).

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