Behaviour of pile due to excavation-induced soil movement in clay

Comportement de foundation dû au movement du sol provoqué par l'exavation dans argile

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ABSTRACT: This paper presents the results of centrifuge model tests to investigate the responses of single pile due to excavation-induced soil movement in kaolin clay. Several tests were conducted with the pile located at various distances behind the retaining wall. In each test, in-flight excavation was carried out until the wall collapse in order to investigate the worst possible scenario that can happen on the pile. It is noted that the maximum induced pile bending moment and pile head deflection occur at the time when the wall begins to collapse. Thereafter, both pile bending moment and deflection are observed to decrease with time.

RÉSUMÉ: Cet article présente les résultats des essais sur centrifugeuse pour étudier les réponses d'une foundation seule dû au mouvement du sol provoqué par l'excavation dans argile. Les plusieurs essais ont été dirigés dans argile du kaolin, sur une foundation seule à plusieurs distances derrière le mur retenant. Dans chaque éssai, l'excavation 'in-flight' a été emportée jusqu'à la chute subite du mur pour étudier le scénario possible le plus mauvais qui peut se passer sur la foundation. Il est noté que le moment de flexion maximum et la deflection a lieu quand le mur commence à s'écrouler. Par la suite, aucune augmentation dans le moment de flexion la deflection est enregistrée.

1 INTRODUCTION

In many cases, pile foundations are used to support lateral loadings. However, problems arise for piles supporting high-rise building whereby the piles are designed to sustain heavy vertical loads without duly considering the possibility of lateral loadings due to soil movement. Some researchers have recently developed various theoretical methods to tackle this problem, see for example Poulos and Chen (1996, 1997), Chow and Yong (1996). On the other hand, experimental and field studies are scanty and the reasons for this may be two folds. Firstly, it is practically impossible to instrument existing piles while planning a deep excavation nearby. Secondly, the cost implication of a prototype field experiment is tremendous. Small-scale laboratory model test is an attractive alternative but the results are not representative of the field prototype behaviour as soil behaviour is stress dependent. As a result, it is necessary to carry out tests in a centrifuge, where the high gravitational force will reproduce the prototype stress level and the test results can then be used to interpret prototype behaviour in a rational manner. Leung et al. (2000) and Shen (1999) described the results of centrifuge tests on pile behaviour due to excavation-induced lateral soil movement in dense sand. This paper presents deals with the pile behaviour due to excavation-induced soil movement in clay.

2 TEST PROCEDURE

2.1 *Apparatus*

Figure 1 shows the centrifuge model set-up for the present study. The model container was made of stainless steel and it has an internal dimension of 540 mm long x 200 mm wide and 470 mm high. The model pile was fabricated from hollow square aluminium tube with an outer and inner dimensions of 9.53 mm and 6.35 mm, respectively. Ten pair of strain gauges were glued on the front and rear sides of the square section at a vertical interval of 25 mm for measuring the bending moment induced in the pile during excavation. The calibrated EI of these two piles is 35 Nm^2, which is equivalent to 220 x 10^3 kNm^2 at 50g. The retaining wall was simulated by using 3-mm thick aluminium alloy plate. The modulus of elasticity of aluminium is 72 x 10^3 MPa. This is equivalent to a bending stiffness of 24 x 10^3 kNm^2/m at 50g. The total embedment of the wall was 160 mm, which was equivalent to 8 m in 50g environment.

2.2 *Clay preparation*

The soil used was homogeneous normally consolidated kaolin clay. The clay was mixed with water at approximately 1.5 times its liquid limit and was poured into a de-aired chamber for 2 days to evacuate the air bubbles from the clay slurry. This was to ensure the clay sample to be fully saturated with water. Subsequently, the clay was shifted into the container and a 17-kg weight would be placed on top of the slurry to stiffen the top layer of the slurry to enable the installation of LVDT at the clay surface. Self-weight consolidation process was carried out to form a normally consolidated clay model. This process was done by mounting the container to the centrifuge swing platform and allowed it to consolidate at 50g for approximately 6 hours. The process is monitored by PPTs which were installed into the model ground before consolidation.

After the clay consolidated, the pile would be installed under 1g environment. A guide which was made of perplex was used

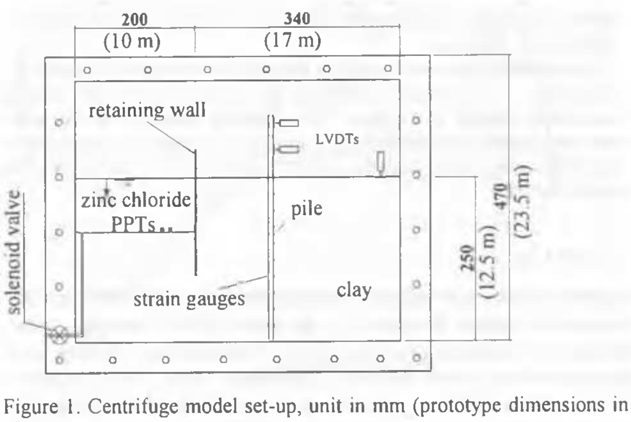

Figure 1. Centrifuge model set-up, unit in mm (prototype dimensions in parentheses)

to guide the pile while it was pushed into the clay. This was essential to ensure the pile to be installed vertically. After this, removal of clay was carried out and the clay was replaced by zinc chloride solution in a latex bag. Subsequently, the container was again mounted on the centrifuge platform and spun up to 50g. The solution would be drained out in-flight to simulate excavation. The pile responses due to the induced soil movement will be captured by the strain gauges mounted on the pile shaft.

2.3 *Outline of Tests*

Three experiments were conducted and are labelled as Clay1, Clay2 and Clay3. In model test Clay1, the pile (hereinafter referred to as pile1) was located 2 m away from the retaining wall; and in test Clay2, the pile (hereinafter referred to as pile2) was located at 4 m away from the wall. In both tests, the excavation was carried out until 4.5 m depth which was identical to the excavation depth in the dense sand tests reported by Leung et al. (2000). However, in soft clay, the excavation depth of 4.5 m was enough to cause the retaining wall to collapse. From tests Clay1 and Clay2, it was noted that the retaining wall collapsed at an excavation depth of approximately 2.5 m. For test Clay3, the pile was at 3 m from the wall and the maximum excavation depth was 3 m.

3 PILE RESPONSES

The induced bending moments along the pile are the primary data obtained from the centrifuge model tests. Other responses of piles such as deflection and rotation profiles are derived from the bending moment data using the spline method, see Shen (1999).

3.1 *Test Clay1*

Figure 2 shows the measured ground settlement at 7 m behind the retaining wall. The datum for settlement is taken at the point when excavation starts. The depth of excavation is also plotted in Figure 2 to study the relation of settlement with excavation depth. It can be seen that the surface settlement of clay continues to increase even after the excavation has stopped. Figure 3 shows the development of bending moments with excavation depth. As expected, the bending moment increases with excavation depth. The increment of bending moment is not linear and the moment reaches the peak value at an excavation depth of approximately 2.5 m. After the peak value, the moment starts decreasing although the excavation is continued until 4.5 m. From video pictures, the wall has failed at an excavation depth of approximately 2.5 m. When the wall fails, the soil moves excessively which causes the soil to reach a plastic state and yielding starts to take place. As a result, the soil will just 'flow' around the pile without exerting any extra load on it. At the same time, due to the increase in pore pressure, the stiffness of the clay decreases and hence the load on the pile reduces. This leads to a decrease in the pile bending moments. The changes in the bending moments are better captured in the plot against time shown in Figure 4. The bending moment reduces with time and the rate of reduction decreases with time.

The bending moment profiles for pile1 are shown in Figure 5. The shape of the induced bending moment profiles for various excavation depths is similar. The bending moment at the pile head and pile tip are almost zero and for all cases, the maximum moment occurs almost at the same location of 7.5 m below the ground surface.

3.2 *Test Clay 2*

Figure 6 shows the induced bending moment profiles at various excavation depths for pile2. It is noticed that there are some similarities between pile1 and pile2. The induced bending moments for both piles increase significantly until the excavation depth of 2.5 m. This again indicates the wall has failed at an excavation depth of 2.5 m and the soil is likely to have reached a

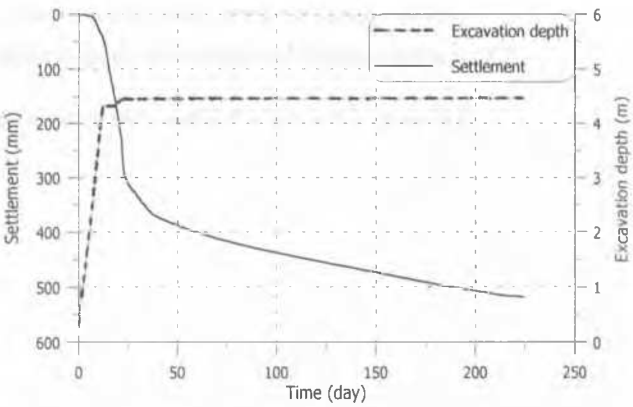

Figure 2. Variations of excavation depth and ground settlement (test Clay1) with time

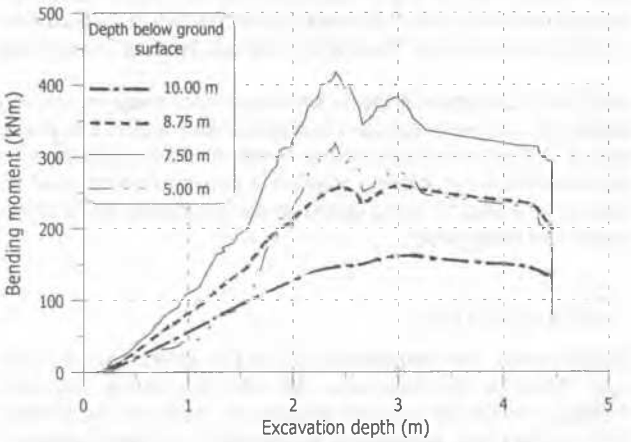

Figure 3. Variations of induced pile bending moments with excavation depth (test Clay1)

plastic state and yielding may have occurred. As pile2 and pile1 are respectively located at 4 m and 2 m behind the wall, pile1 is thus subjected to more severe soil movement. As expected, bending moment of pile1 is higher than that of pile2.

3.3 *Test Clay3*

In test Clay3, pile3 was located at 3 m behind the wall. In this test, the maximum excavation depth of 3 m is smaller than that of previous two tests. The induced bending moment profiles of pile3 are presented in Figure 7. Again, the shape of the profiles is similar to that of pile1 and pile2. The moment increases significantly up to an excavation depth of 2 m. When the excavation depth increases from 2 m to 2.5 m, the moment only in-

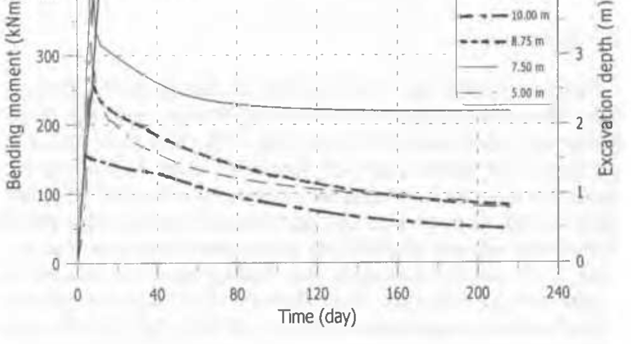

Figure 4. Variations of bending moments with time (Test Clay1)

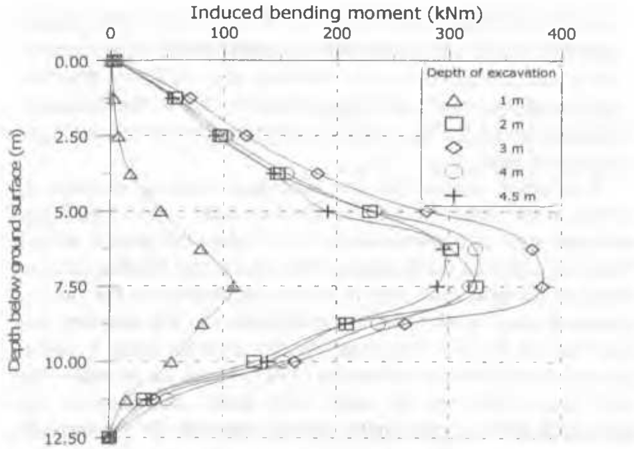

Figure 5. Induced bending moment profiles for pile1 (2 m away from wall)

creases marginally. After the maximum excavation depth of 3 m, the bending moment decreases.

From the three tests, it is noted that as the distance between the pile and wall increases the bending moment on the pile reduces. pile1 has the highest maximum induced bending moment as it was located nearest to the wall as compared to pile2 and pile3.

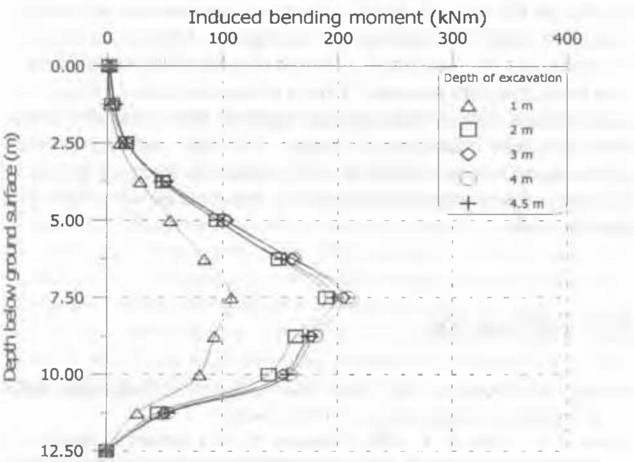

Figure 6. Induced bending moment profiles for pile2 (4 m away from wall)

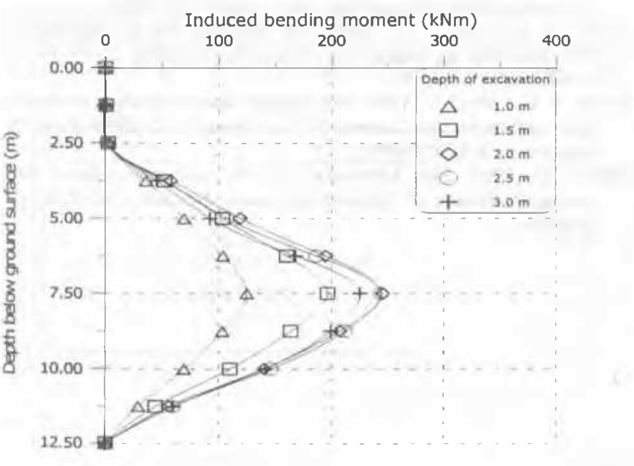

Figure 7. Induced bending moment profiles for pile3 (3 m away from wall)

4 TIME EFFECTS ON PILE BEHAVIOUR

Figure 8 shows the relation between the maximum bending moment which occurs at 7.5 m below ground level with the depth of excavation. It is evident that the induced bending moments increase significantly until the excavation depth reaches around 2.4 m when the retaining wall is at the onset of collapse. Although the readings of pile1 were slightly jumpy and the plot shows some instability, the results obtained from the test is still reasonably reliable and provide a good source to study the pile behaviour. The change of the maximum bending moment with excavation depth suggests that the soil has become plastic and flowed around the piles without exerting any extra loads on the piles. For pile1, the maximum bending moment was 420 kNm at about 2.4 m depth of excavation. After the wall failure, the bending moment decreased and the phenomenon will be examined later. On the other hand, the bending moment for pile2 at 2.4 m depth of excavation was 190 kNm, and this continued to increase after the failure took place. However the rate of increment was insignificant. After the maximum excavation depth of 4.5 m, the moment for both piles decreased. This similar trend is also demonstrated in pile3, where the maximum excavation depth is only 3 m. In this test, the maximum moment occurs before 2.4 m and it starts reducing until the excavation depth reaches 3 m. It is worth noting that, in these three piles, the moments change even after the excavation has reached the final depth. This suggests that the pile behaviour in clay is time-dependent.

The time-dependent behaviour of the piles is clearly shown in Figure 9. The maximum moment for the three piles occurs on the 7th day of the excavation. In the centrifuge tests, the excavation of 4.5 m of clay was accomplished in about 12 days in prototype time scale. For Test Clay3, the excavation of 3 m was completed in approximately 10 days. It can be seen from Figure 9 that after the excavation, the moments of the piles decreased until they became stable and the change taper off at about 150 days. The shape of the post-peak moment curves is similar to a hyperbola. This suggests that, at the time when the excavation is completed, due to the short time frame (around 10 to 12 days), the clay could be assumed still under an almost undrained condition. When the zinc chloride is drained off to simulate the removal of soil in excavation, negative excess pore water pressure will be generated. This negative pore pressure will dissipate with time. As the pore pressure increases, the effective stress of the soil decreases. The stiffness of the soil is highly correlated to the effective stress of the soil, and as a result the soil becomes weaker. This further leads to the alleviation of loadings on the piles, and hence the maximum bending moment decreases with time. At approximately 150 days, according to Terzaghi theory on the

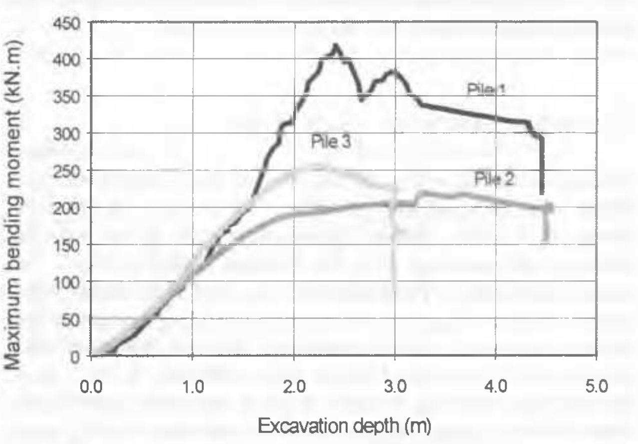

Figure 8. Change of maximum bending moment with depth of excavation

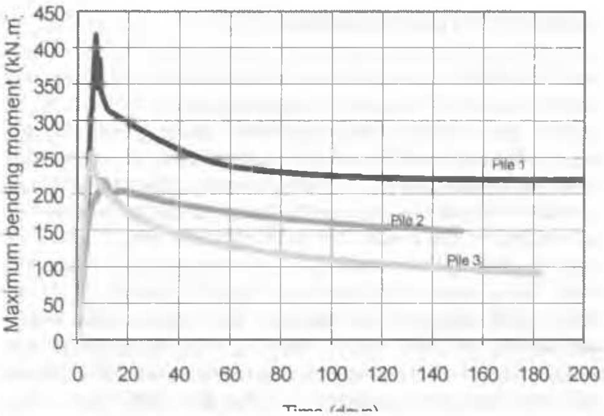

Figure 9. Change of maximum bending moment with time

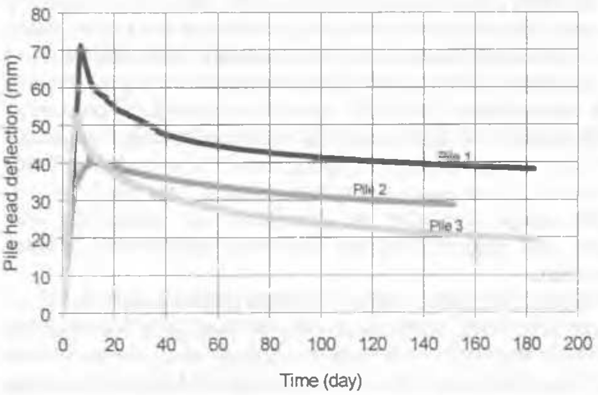

Figure 10. Pile head deflection for three clay tests

rate of one-dimensional consolidation, the average degree of consolidation for the clay is close to 90%. At this stage, the dissipation of pore pressure has almost completed, drained condition applied and the equilibrium of stress will be achieved. Hence, the bending moments of the piles stabilize.

Figure 10 shows the change of pile head deflection with time in all the three tests. A similar trend can be noticed for the tests. The deflections of the pile head increase initially and start to decrease after approximately 10 to 12 days. Comparing this with Figure 9, it clearly shows that the changes of maximum bending moment and pile head deflection with time are of similar pattern. This suggests that when the soil near the surface reaches a plastic state, the force acting on the pile will reduce. However, the soil near the pile tip is restraining the pile from moving forward, following the flow of the top soil. Therefore, the measured pile head deflections indicate a decrease with time, implying that the pile has bounced back when the soil starts to yield.

5 COMPARISON WITH SAND TESTS

When the retaining wall collapses, the soil flows around the piles without any extra detrimental effect. This can be seen from the change of bending moment during excavation. In the tests involving both sand and clay, the bending moments of pile increase with depth of excavation until the wall fails. Shen (1999) showed that the magnitude of maximum bending moment of the pile in sand would remain unchanged after the failure of wall. However the behaviour of pile in clay is different. In test Clay1, the maximum bending moment of pile1 decreased significantly after the wall collapse. In test Clay2, the moment of pile2 stops increasing after the 10^{th} day, and begins to decrease after approximately 20 days, see Figure 9. Similar trend of the change of maximum moment is demonstrated in test Clay3. The findings suggest that after the excavation ceased, the bending moment would still change with time which is very different from that observed in the sand tests. This is mainly due to the dissipation of negative pore pressure in clay which has developed during the excavation process.

It is worth noting that the maximum bending moment occurred at the same location for the free head piles in both sand and clay tests, at approximately 7.5 m below the ground surface. This suggests that the location of the maximum bending moment along at the pile shaft will be at the same level as the pivot of retaining wall. In all tests, the embedment of the retaining wall was kept at 8 m. A limit equilibrium analysis using a method proposed by Bolton and Powrie (1987) based on an admissible soil stress field on the wall, with active and passive zone switching about a pivot point was carried out. In the tests, the wall was unpropped, and by satisfying the moment and force equilibrium it was found that the pivot was located at around 0.3 m from the bottom of the wall (for detail of the retaining wall calculation, refer to Shen, 1999). This means that the pivot of the wall by this analysis method was 7.7 m below ground surface. This was very close to the elevation of the maximum bending moment.

6 CONCLUSION

Several centrifuge model tests were carried out to investigate the behaviour of pile due to excavation induced soil movement in clay. The piles are located between 2 to 4 m from the wall. It is found that the induced bending moments increase as the excavation starts until the retaining wall collapses. After the excavation is completed, the maximum moment decreases with the dissipation of excess pore pressure. This is a time dependent effect. The soil becomes almost fully drained when all the excess pore pressure dissipated after approximately 150 days and the induced pile moments then stabilized. The maximum induced pile moment decreases with the increase of distance between the pile and the wall.

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