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Three-dimensional centrifuge modelling of the effects of tunnelling on vertical behaviour of pile group

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Abstract. In densely built urban cities, tunnels are likely to be constructed adjacent to existing deep foundations. Tunnelling activity inevitably induces soil stress changes and soil movements in the ground, which may affect nearby existing pile foundation. Although a number of studies have been carried out by various researchers to investigate tunnelling effects on piles, the three-dimensional effects of tunnelling on pile group have not been well-investigated and understood. In this paper, a three-dimensional centrifuge test was carried out to investigate the effects of tunnel construction on a pile group in dry sand. The diameter of each pile was designed as 0.8m (in prototype) and the diameter (D) of the simulated tunnel is 6.08m. Tunnel advancement was modelled in-flight by controlling a volume loss of 1.3%. The tunnelling process was divided into four stages to model the three-dimensional tunnelling effects. The closest distance between the tunnel and the pile group was 1.5m at the pile toe. Measured settlement of pile group, as well as axial force distribution of piles in the pile group are reported and discussed.

1. Introduction

Tunnels are often preferred for the solution of congested traffic in big cities. Due to the limited underground space in densely populated areas, tunnels are often constructed near piled foundations. Tunnel construction inevitably causes change in soil stress and hence induces ground movements. Uncontrolled ground movement can induce cracking in building and additional loads on piles under nearby structures. Estimation of tunnelling effects on pile foundations is a major challenge for designers and engineers.

The effects of tunnelling on piles have been investigated numerically or analytically by a number of researchers (Chen et al., 1999; Mroueh and Shahrouh, 2002; Lee, 2004; Kitiyodom et al., 2005; Lee and Ng, 2005; Lee, 2012). In addition, some centrifuge model tests have also been carried out to investigate tunnelling effects on piles. Bezuijen and van der Schrier (1994) pointed out that pile settlement could be quite significant if the distance between pile and tunnel is less than the tunnel diameter. Loganathan et al. (2000) carried out three centrifuge tests to investigate the effects of tunnelling on the settlement, lateral movement and bending moment of piles in clay. Working load was applied to the piles. Tunnelling-induced bending moments and axial forces on the piles were presented. Jacobsz et al. (2004) investigated the adverse effects of tunnelling beneath a pile in dry sand.



An influence zone was identified above and around the tunnel in which the pile could undergo significant settlement. Lee and Chiang (2007) found that the depth of the tunnel relative to the pile had a significant influence on the distribution of bending moment along the pile in dry sand. Marshall and Mair (2011) carried out centrifuge model tests to investigate the effects of tunnelling beneath driven or jacked end bearing piles in sand. They found that the volume-loss-induced pile failure, as reflected in a significant increase in the rate of pile settlement, is closely related to the distance between the tunnel and the pile toe.

Although the effects of tunnelling on piles have been investigated by a lot of researchers, most of the studies were carried out under plane strain condition. Three-dimensional tunnelling effects on piles have not been systematically investigated. In this study, centrifuge modelling was carried out to assess the responses of a pile group due to tunnelling effects. A three-dimensional tunnel construction including four advancing stages was simulated. A volume loss of 1.3% was controlled for each subsection. The settlements of the pile group were recorded during the four construction stages. Heavily instrumented model piles were used to capture axial forces distribution induced by tunnelling at different advancing stages. Three-dimensional influence of tunnelling effects on the vertical behavior of pile group at different advancing stages were reported and analysed.

2. Centrifuge modelling

2.1. Experimental setup

The centrifuge model test was carried out at the Geotechnical Centrifuge Facility of the Hong Kong University of Science and Technology (Ng et al. 2001; Ng et al. 2002). The test was conducted at an acceleration of 40 g.

Figures 1(a) and 1(b) show the schematic elevation and plan view of the centrifuge model, respectively. A 2×2 pile group was located at center of the model container. Each pile had a diameter (D_p) of 20mm and a length of 600mm. The embedded depth of each pile was 490mm. In prototype, the corresponding pile diameter and embedded depth was 0.8m and 19.6m, respectively. The model tunnel was located at the same elevation as the pile toe. The tunnel diameter (D) was 152mm, which was corresponding to a diameter of 6.08m in prototype. The distance from the center-lines of the tunnel to each pile was $0.75D$, and the closest distance between the tunnel and pile group was only 1.5m in prototype.

Fig. 1(b) shows the plan view of the model. The longitudinal length of the tunnel was 304 mm, which was equal to $2D$. In order to model the three-dimensional tunnelling process, the tunnel excavation was divided into four subsections, each of which had an advancing distance of $0.5D$. The three-dimensional influence of each advancing stage on the pile group was investigated.

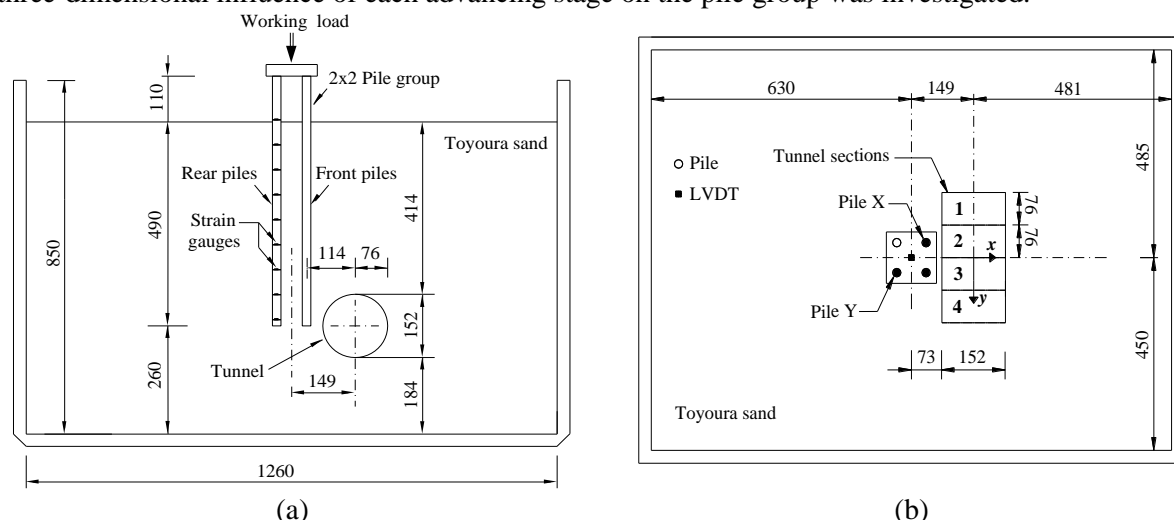


Figure 1 Schematic diagram of centrifuge model: (a) Elevation view; (b) Plan view

2.2. Model preparation

The model package is shown in Figure 2. Dry Toyoura sand ($G_s = 2.65$, $e_{\max} = 0.977$, $e_{\min} = 0.597$, $\phi'_{cv} = 31^\circ$) (Ishihara, 1993) was used in the test. The centrifuge model was prepared by pluvial deposition method. Sand was rained from a hopper which was kept 500mm above the soil surface. The measured average soil density was 1506 kg/m^3 , which was corresponding to a relative density of 57%.

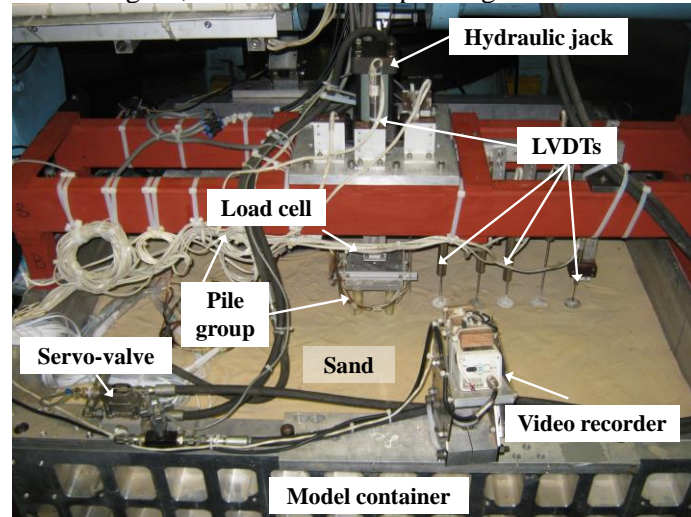


Figure 2 Centrifuge model setup

2.3. Model piles and instrumentation

In the 2×2 pile group, each model pile was made from an aluminum tube. The outer diameter of each pile was 20mm, which was corresponding to a pile diameter of 0.8 m in prototype scale. Each pile had an axial rigidity $E_m A_m$ of 3517N and a bending rigidity $E_m I_m$ of 113 Nm^2 . The corresponding $E_p A_p$ and $E_p I_p$ were 5627kN and 289 MNm^2 in prototype scale, respectively. The 2×2 pile group was also shown in Figure 1(a). The spacing between piles was 70 mm, which is equal to $3.5D$. Among the four piles in the pile group, piles X and Y were instrumented with strain gauges. The cap of pile group was made from aluminum with a thickness of 28 mm.

Load was applied in-flight to each pile by a closed-loop loading system, which was in a load-control mode. The magnitude of load to each pile was controlled by a load cell. The overall settlement of the pile group was measured by a linear variable differential transformer (LVDT 1) located at the center of the pile cap. The location of the LVDT is shown in Figure 1(b).

2.4. Modelling of tunnel excavation

The model tunnel was consisted of four cylindrical rubber bags, which were filled with de-aired water. Each tunnel excavation stage was modelled in-flight by draining out a controlled amount of water from the rubber bag. The amount of water drained away was controlled to induce a volume loss of 1.3% for the tunnel advancement.

2.5. Test procedure

After model preparation, the acceleration of the centrifuge was increased to 40g. The pile group was loaded in-flight step by step. An incremental vertical load of 500N (800kN in prototype scale) was applied in each step. Each load increment was maintained for 3 minutes, which was sufficient to obtain a steady state in this test. When the load increased to a designed working load of 14000 N (22400 kN in prototype scale), the tunnel advancement with designed volume loss was carried out. Four advancing stages were simulated by draining away water from each of the four rubber bags respectively, as shown in Figure 1(b). During the test, the changes of axial forces along each instrumented pile, as well as the settlement of pile group were recorded.

3. Test results

All the test results are presented in prototype scale unless stated otherwise.

3.1. Settlement of pile group

The settlement of the pile group at the center of pile cap was measured by LVDT 1 located at the center of the pile cap (shown in Figure 1(b)). The settlement of pile group was normalized with the diameter of pile (i.e., 800mm). The results are shown in Figure 3. The first tunnel excavation stage induced a settlement of 4.7 mm (i.e., 0.6%D) to the pile group. The second excavation, however, caused a much larger settlement of 28.8 mm (i.e., 3.6%D) to the pile group. This was the largest among all tunnel advancing stages. The third excavation stage induced only 6.9 mm overall settlement to the pile group, although it was located at the same distance to the pile group as the second excavation (since they were symmetrical). It appears that the influence of the second advancing stage was more significant to the pile group. The settlement induced by the fourth excavation was 3.0 mm. After the four stages excavation were completed, the total overall settlement was about 43.5 mm, which was corresponding to 5.4% of the pile diameter.

The settlements measured by LVDT 2, LVDT 3, LVDT 4 were also included in Figure 3. It can be seen that settlement measured by LVDT 4 was smaller than that measured by LVDT 1 and LVDT 3. This indicated the pile cap suffered from a tilting towards the tunnel. Based on the difference between the settlement value measured by the LVDT 3 and LVDT 4, as well as the spacing between the two piles (3.5D), the transverse tilting was calculated as 0.15%. Similarly, the longitudinal tilting can be deduced with the difference of measurement between LVDT 2 and LVDT 3. The deduced longitudinal tilting was 0.03%, which was substantially smaller than the transverse tilting.

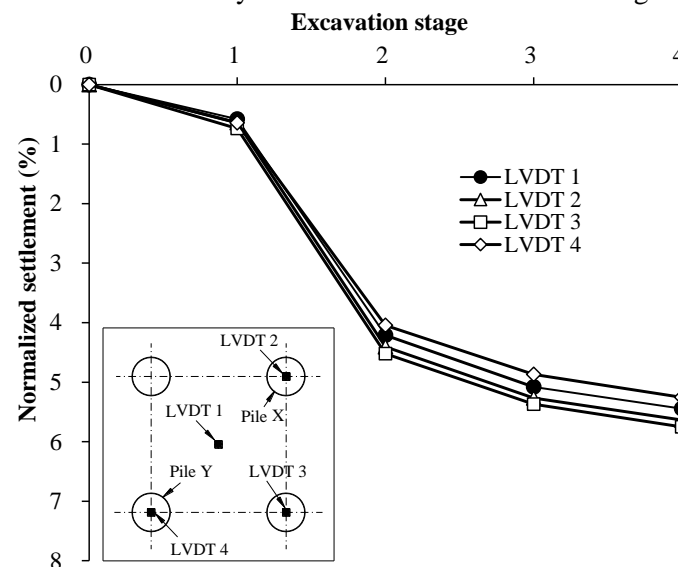


Figure 3 Settlements of the pile group at various tunnelling excavation stages

3.2. Axial force of pile

Fig. 4(a) shows the tunnelling-induced axial forces along pile X. In general, as tunnel advancing was conducted, the axial force along the upper portion of the pile shaft decreased. This indicates that load carried by pile X gradually decreased due to tunnel excavation. The reductions during advancing stages were -25.4, 205.9, 90.0 and 25.4 kN, respectively. It is clear that the second excavation stage caused the largest reduction in the pile load. At the end of the excavation, the load carried by pile X decreased by about 295.9 kN, which was corresponding to 5.3% of the applied working load for each pile (5600 kN).

The toe resistance was increased as the tunnel advancing was carried out. The increments in toe resistance due to the four advancing stages were 40.0, 90.2, 90.2 and 60.2 kN, respectively. The fourth

excavation still caused considerable increase of the toe resistance. The final increase in toe resistance was 280.6 kN at the end of the tunnel advancement. This was equal to 5.0% of the working load of each pile.

These results show that the load carried by pile X decreased due to the tunnel excavation, while at the same time the toe resistance increased. It is indicated that the shaft resistance of pile X was reduced by the tunnel construction. The reduction in shaft resistance may cause a load transferred to its toe resistance (at an expense of pile settlement), or to other piles in the rear pile row.

Fig. 4(b) shows the tunnelling-induced axial force along pile Y. As the four advancing stages were conducted, the axial force increased along the entire pile shaft. This was different from the behaviour of pile X. The load carried by pile Y gradually increased due to tunnel advancing. The increments during advancing stages were -15.8, 157.6, 94.7 and 47.3 kN, respectively. The second excavation caused the largest increase in the pile load. This was consistent with the measured settlement of pile group shown in Figure 3. At the end of the excavation, the load carried by pile Y increased by about 283.8 kN, which was corresponding to 5.1% of the applied working load to each pile (5600 kN). It should be noted that the magnitude was almost the same as the reduction in the load carried by pile X.

The toe resistance of pile Y also increased as the tunnel advancing was carried out. The increments due to the four advancing stages were 0, 168.3, 168.3 and 134.3 kN, respectively. The fourth excavation still caused considerable increase of the toe resistance of pile Y. This was similar to the behavior of pile X. The final increase in toe resistance was 470.9 kN at the end of the tunnel advancement. This was equal to 8.4% of the working load.

The results show that the load carried by pile Y increased when the tunnel advancing was carried out. As a result, both the shaft resistance and the toe resistance were increased. This was different from the behavior of pile X. The results may indicate the change of the load distribution between the piles in a group. A certain amount of load originally carried by the front piles transferred to the rear piles. In this study, the magnitude of the transferred load was 5.1 to 5.3 % of the applied working load to each pile.

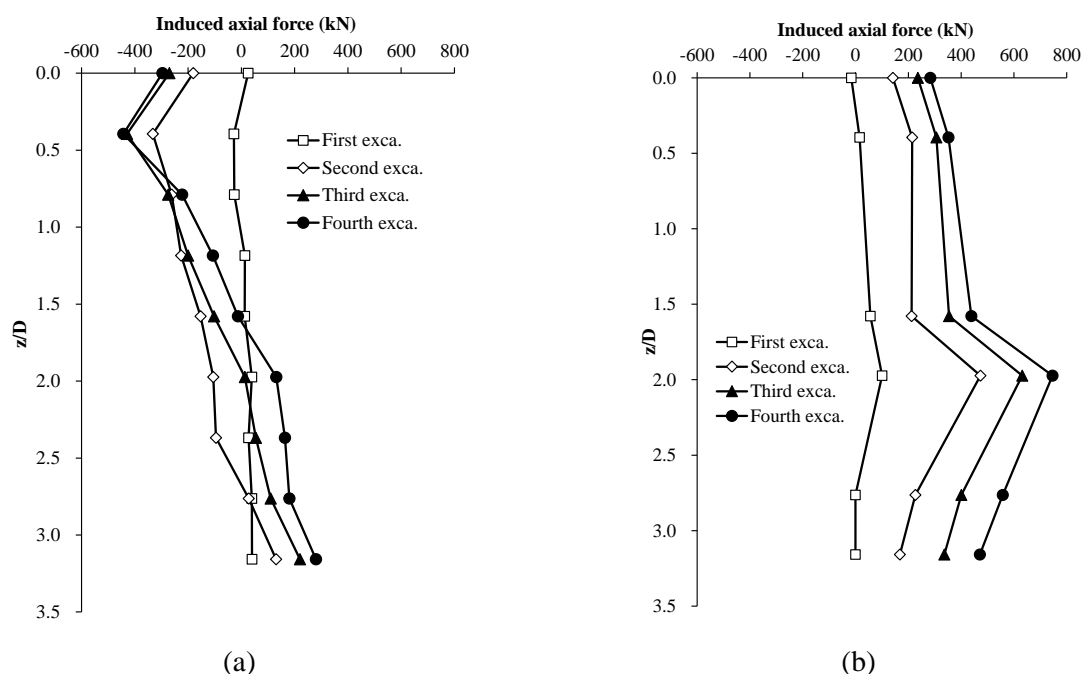


Figure 4 Measured induced axial force in piles due to tunnelling in: (a) Pile X (b) Pile Y

4. Summary and Conclusions

In this paper, the effects of tunnelling on the vertical behavior of pile group were investigated by carrying out three-dimensional centrifuge model test. The three-dimensional tunnel construction was

simulated in-flight by controlling a volume loss of 1.3% in four advancing stages. Based on the test results, the following conclusions can be drawn:

(1) The tunnel excavation caused a load transfer from the front pile to the rear pile in the pile group. The magnitude of the transferred load was 5.1 to 5.3 % of the applied working load to each pile. This was because the shaft resistance of the front pile was reduced due to the stress relief as well the soil movement induced by tunnel excavation. Meanwhile, the tunnel excavation caused an increment of toe resistance for both front and rear piles.

(2) The construction of tunnel caused a 5.1% reduction in the load taken by front pile. In the four excavation stages, the second excavation induced the largest reduction in the pile load. This was consistent with the measured pile settlement. The toe resistance of front pile increased by 5.0% after the tunnel excavation.

(3) In contrast to the decrease of load taken by the front pile, the load taken by the rear pile increased by 5.1% of the working load. The shaft resistance increased after the four stages excavation of tunnels. The increase of shaft resistance was also different with that for front piles. Similar to the behavior of front pile, the final increase in toe resistance was 470.9 kN (i.e., 8.4% of the working load) at the end of the tunnel advancement.

(4) Under a volume loss of 1.3%, the settlement of pile group induced by the tunnel excavation was about 5.4% of each pile diameter. In the four excavation stages, the largest settlement of the pile group was induced by the excavation of the second subsection. The excavations of the other three tunnel sections induced much less settlements to the pile group.

Acknowledgments

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