EXCAVATION INDUCED RESPONSE OF PILE FOUNDATIONS

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ABSTRACT

In urban areas excavations for construction of basements, tunnels and other underground facilities inevitably influence the existing pile foundations. Due to the release of stresses during excavations, confining pressures around existing piles decrease significantly inducing additional deflections and bending moments. It is important to quantify these effects at the design stage, in order to protect the existing structures during nearby deep excavations. In this paper, the impact of excavation induced ground movements on adjacent pile groups is investigated. Numerical simulations based on the finite element method are performed on free-head and capped-head piles in different pile configurations. The problem was modelled considering the three-dimensional geometry, which facilitates to simulate the arching and shielding effect of piles within a group. The response of both interior and peripheral piles is investigated. Results show that the presence of front piles reduces the detrimental effects on the rear piles within the group. In addition the provision of a pile cap significantly reduces the deflection of pile group due to load transfer to the rear piles, which are located away from the wall supporting the excavation. Outcomes of this research will contribute towards the design and construction of resilient pile foundations.

1 INTRODUCTION

In the last few decades, significance of ground deformations on single pile response during nearby excavations was investigated extensively using both theoretical and experimental studies. Theoretical studies were carried out using the finite element method, boundary element method and finite difference method to find out the single pile response adjacent to deep excavations (e.g., Poulos and Chen 1996; Poulos and Chen 1997; Chen and Poulos 1996; Poulos 2005; Xu and Poulos 2000 and Zhang et al. 2011). Leung et al. (2000) and Ong et al. (2006a) carried out centrifuge tests to investigate the single pile behaviour during nearby excavations in sand and clay soils, respectively. However, in the literature, only few research studies have been carried out investigating the pile group behaviour due to excavation induced ground deformations. Some case histories were published in the past, where the pile groups were located very close to deep excavations (e.g., Finno et al. 1991, Goh et al. 2003), but with limited measured data and site conditions.

Chen and Poulos (1996) studied pile group response due to excavation induced lateral movements using finite element and boundary element approaches. First they computed free field ground deformations without considering the pile group using the finite element method. Then the pile group behaviour is investigated applying ground deformations to the pile, using the program PALLAS (Hull 1987), which is based on the boundary element method. They obtained group factors for maximum bending moments for three different values of limiting pile-soil pressures when the pile is located 1 m away from the excavation and the stability number, N, is seven, which is defined as:

$$N = \frac{\gamma H}{c_u} \tag{1}$$

where γ is the unit weight of soil , H is the depth of the excavation and c_u is the undrained shear strength of the soil. According to Chen and Poulos (1996), the group effect is not significant for shallow excavations, where the stability number is less than six. For deep excavations with larger soil movements, group effect has a substantial influence based on the spacing and configuration of piles within the group.

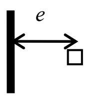
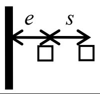
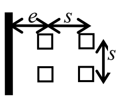
Leung et al. (2003) and Ong et al. (2006b) investigated the pile group behaviour in sandy and clayey soils, respectively, using centrifuge tests. They investigated the behaviour of free-head and capped-head pile groups made of two, four and six piles located behind unsupported excavations. According to their results, when the piles were located in a row parallel to the wall, the effect of pile-soil-pile interaction on the behaviour of individual piles is insignificant compared to the case with piles positioned perpendicular to the wall. In the latter case, the front piles located closer to the retaining wall reduces the adverse effects of excavation induced soil movements on rear piles. Furthermore, provision of a pile cap significantly affects the behaviour of individual piles in the front row. In addition, it was found that peripheral piles in a group experience higher bending moments when compared to interior piles of the same group, which are less exposed to the adverse effects of excavation induced ground movements. They mentioned that since the pile caps are transmitting the bending moments from front piles to rear piles, the design of pile caps significantly influence the behaviour of piles in a group during nearby excavations.

Response of pile groups subjected to excavation induced ground movements is a three-dimensional problem due to the soil flow between adjacent piles and wall. In addition, the pile and ground deformations are coupled in the real situation. Although two step approaches discussed before decouples ground deformations and pile deformations, this assumption is not realistic. Hence in this paper, finite element models involving full three-dimensional geometry of the excavation and pile group were used to study the pile group behaviour during nearby excavations. Centrifuge tests reported by Ong et al. (2006b) were numerically simulated and the shielding effect of piles in a group during deep excavations was investigated by varying the pile group configuration and pile head conditions.

2 DESCRIPTION OF THE CENTRIFUGE TESTS

Centrifuge tests reported by Ong et al. (2006b) shown in Table 1 are used to investigate the shielding effect of piles in a group. In these tests, the behaviour of a pile group adjacent to an unbraced excavation is studied. The model container has prototype dimensions of 27 m x 10 m x 23.5 m. The Kaolin clay was filled up to a depth of 6.5 m above a Toyoura sand layer, which has a thickness of 6.0 m. A Latex bag filled with $ZnCl_2$ solution, which has a unit weight equivalent to the clay, is used to represent the soil region that needs to be excavated. The 1.2 m deep excavation was simulated by draining the $ZnCl_2$ solution in six steps over 2 days in the prototype scale. Piles were modelled using hollow square aluminium tubes with outer prototype dimension of 630 mm. Tests were carried out at a centrifugal acceleration of 50g at the geotechnical centrifuge facility, National University of Singapore.

Table 1. Pile configurations used for the analysis.

Pile configuration	Test	Dimensions	Pile head condition
	T1	*e*=3 m	Free head
	T2	*e*=5 m	Free head
	T3	*e*=3 m, *s*=2 m	Free head
	T4	*e*=3 m, *s*=2 m	Capped head
	T5	*e*=3 m, *s*=2 m	Free head
	T6	*e*=3 m, *s*=2 m	Capped head

2.2 MATERIAL MODELS AND PROPERTIES

The constitutive behaviour of the clay is modelled using the Mohr-Coulomb model. The finite element analysis simulating the centrifuge tests was carried out assuming the undrained behaviour of clay. Since the clay has a permeability of $1.18x10^{-3}$ m/day and the excavation was carried out in two days, this is a reasonable assumption. The variation of undrained shear strength of the clay with depth is shown in Figure 1. The top 2.5 m soil crust was found to be over consolidated and soil below that level was normally consolidated. The elastic modulus of the Kaolin clay was calculated using E_c/c_u = 400 (Poulos and Davis, 1980). The internal friction angle and the Poisson's ratio for the undrained Kaolin clay were assumed as zero and 0.49, respectively. Lateral earth pressure coefficient at rest, K_o, is taken as one. The unit weight of the soil is 16.5 kN/m^3 (Ong et al. 2006). The Toyura sand layer below the clay layer was also modelled using the Mohr-Coulomb model with an internal friction angle of 40° and an elastic modulus of 6z MPa, where z is the depth below the ground surface in meters (Ong et al. 2006a). The Poisson's ratio of the sand is assumed to be 0.3. Bending stiffness of square piles used for the experiments is $2.2x10^5$ kNm^2 in prototype scale, which is equivalent to the bending stiffness of a 600 mm diameter concrete pile. The pile has an embedment depth of 12.5 m in the prototype scale. A 3 mm thick aluminium plate is used as the wall in the centrifuge test, with prototype bending stiffness of $24x10^3$ kNm^2/m and an embedment depth of 8 m. Piles, pile cap and wall are modelled assuming linear elastic behaviour.

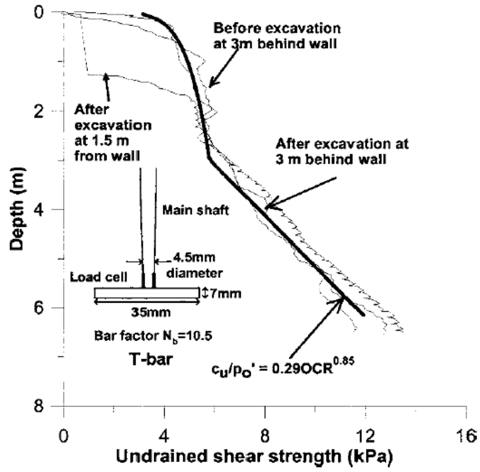


Figure 1: Variation of undrained shear strength with depth (Ong et al. 2006b).

3 FINITE ELEMENT MODEL

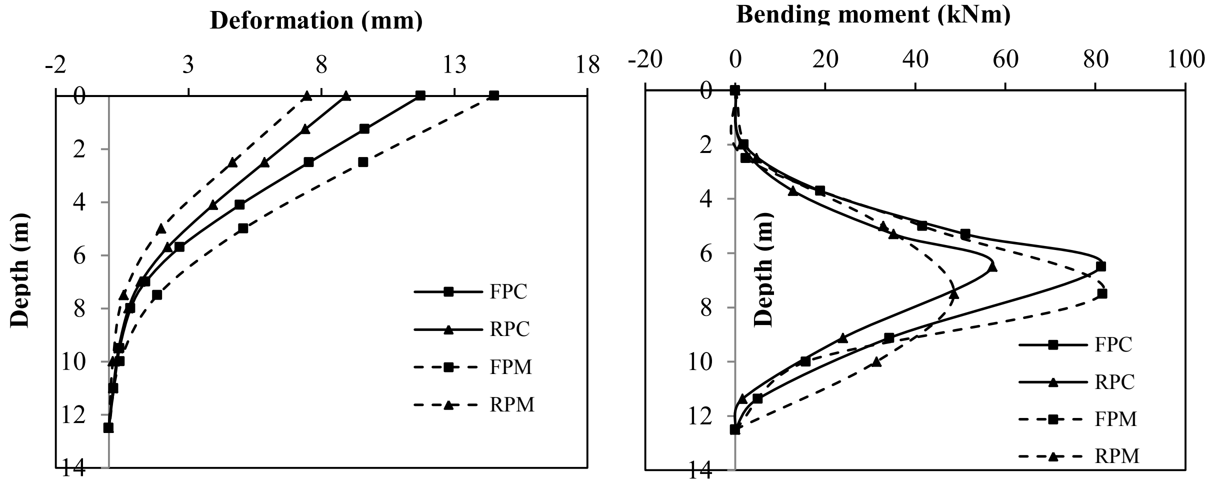
A three-dimensional finite element model based on prototype dimensions is used to simulate the centrifuge tests. ABAQUS/Standard finite element program is used to investigate the problem. Due to symmetry of the problem, only half of the geometry is considered for the numerical model.

Soil, pile, pile cap and wall were modelled using twenty-node quadrilateral brick elements with reduced integration formulation. At the bottom of the finite element mesh, horizontal and vertical movements are restrained ($u_x = u_y = u_z = 0$). Nodes over the vertical side faces are free to move in the vertical and horizontal directions along the surfaces of the container and restrained only in the directions perpendicular to the side faces. In the centrifuge model, tips of the pile group extend up to the bottom of the container and the movement of pile tips is restrained only by the surrounding soil and base of the container. If the same restraining conditions are applied to pile tips as for the base of the container, large bending moments will be developed at pile tips, which is not in agreement with the observed bending moments at the pile tip. Hence only the centre of the bottom of each pile is restrained from movements in all directions.

The soil-pile interaction in tangential direction is modelled using the Coulomb friction model, which is governed by a friction coefficient and a limiting displacement for elastic slip. Here a value of 0.3 was selected as the friction coefficient. Based on the typical values reported by Broms (1979), a limiting displacement of 5 mm was selected for the elastic slip to mobilise the full skin friction at the pile-soil interface based on the typical values reported by Broms (1979). The pile-soil interaction in normal direction is modelled using a “hard” contact while allowing separation at the pile-soil interface. Another advantage of allowing slippage and separation at the pile-soil interface is that it will avoid the overestimation of the deflection and bending moment of the pile as shown by Miao et al. (2006).

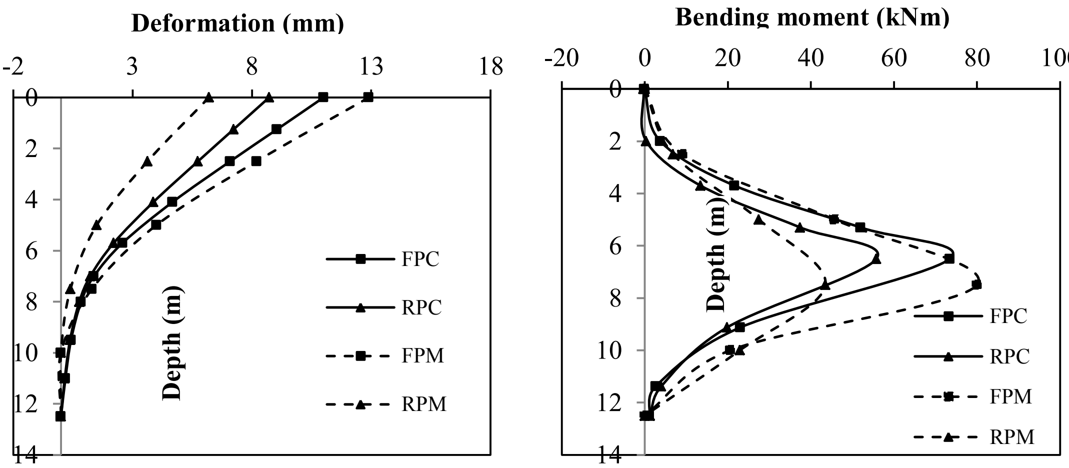
4 COMPARISON OF FINITE ELEMENT MODEL RESULTS WITH CENTRIFUGE TEST DATA

In this section results from the finite element model are compared with measured data from centrifuge tests to validate the numerical model. Figure 2 shows the computed and measured pile response for the test T3 at the end of 1.2 m depth of excavation. Two free-head piles are used in this test. The front pile is located 3 m away from the excavation and the rear pile is located 5 m away from the excavation. Since this is an unbraced excavation, pile deformations shown in Figure 2(a) are in cantilever shape. Deflection of front pile, obtained from the finite element analysis, is slightly under predicted and the deflection of rear pile is slightly over predicted compared to the centrifuge test results. This may be due to the inhomogeneous soil shear strength parameters in the lateral direction of the soil domain used for the centrifuge test as illustrated in Figure 1. Measured undrained shear strengths show different profiles at distances of 1.5 m and 3 m from the excavation after completion of the excavation. The predicted bending moment profile for the front pile shown in Figure 2(b) agrees well with the measured values during the centrifuge test. However, for the rear pile, computed bending moment slightly over predicted the measured bending moment. In both piles, the maximum bending moment occurs near the mid-height of the pile. For both pile deflections and bending moments developed in the rear pile are less than those developed in the front pile. The maximum bending moment developed in the rear pile is 30% less than that in the front pile and the maximum deflection developed in the rear pile is 24% less than that in the front pile. These results confirm the shielding effect of front piles on the rear piles.



Note: FPC- Computed value for front pile, RPC- Computed value for rear pile, FPM- Measured value for front pile, RPM- Measured value for rear pile.

Figure 2: (a) Pile deflection and (b) Pile bending moment for Test 3.



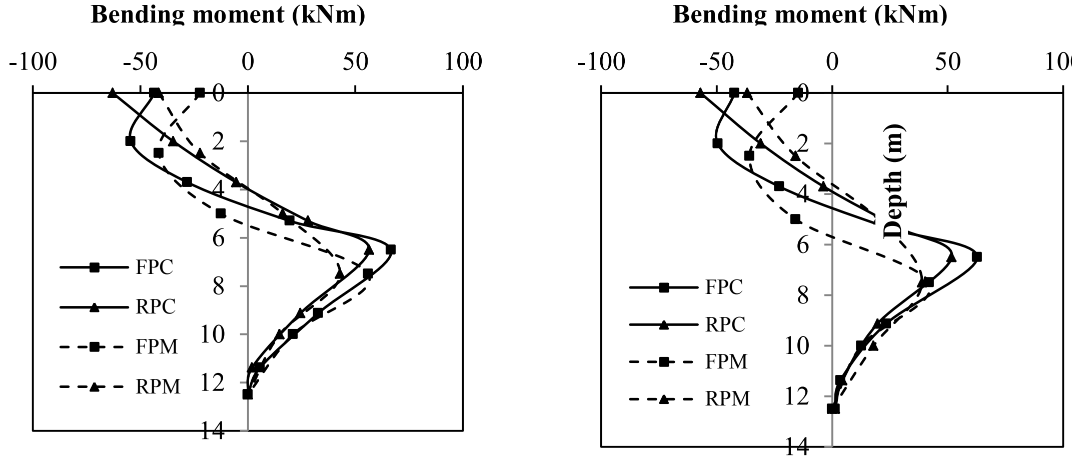
Note: FPC- Computed value for front pile, RPC- Computed value for rear pile, FPM- Measured value for front pile, RPM- Measured value for rear pile.

Figure 3: (a) Pile deflection and (b) bending moment for Test 5.

Figure 3 shows the measured and predicted response for a group of 2 x 2 free head piles (T5) and they are similar to the results given by test T3. In test T5, two piles are located in a line perpendicular to the wall as shown in Table 1. The predicted location of the maximum bending moment slightly differs from the measured one. Due to the arching effect between piles in a row parallel to the wall, the pile deflection and bending moment values are slightly less for test T5 when compared to centrifuge test T3. This difference becomes significant when the ratio of pile spacing over pile diameter decreases. Overall, finite element predictions agree well with the measured pile deflections and bending moments.

Centrifuge tests T3 and T5 consist of free head pile groups. When there is a capped pile group, a concrete pile cap with planar dimensions of 3 m x 1.25 m and a thickness of 1.55 m is used for test T4 and 3 m x 3 m cap with thickness of

1.55 m is used for test T6. Figure 4 shows the bending moment distributions along the pile predicted from the finite element model and the bending moment measured during the centrifuge test. Bending moment predictions from the finite element model are slightly higher than the centrifuge test results. One reason for this difference may be the method adopted to connect the concrete pile cap to the pile group. In the centrifuge test, pile cap is welded to the pile group and in the finite element model, a tie constraint is used to connect the cap to the pile, which represents a fixed boundary condition with zero rotational and translational movements at the pile head. The difference between the degrees of restraint in two cases may have contributed to the difference observed between finite element and centrifuge results. However, the overall bending moment distribution has the same shape.

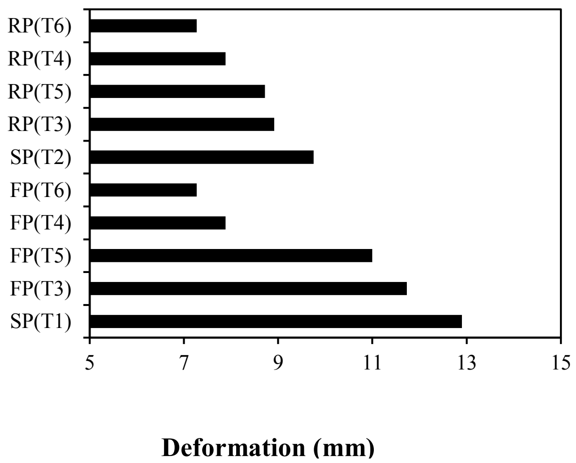


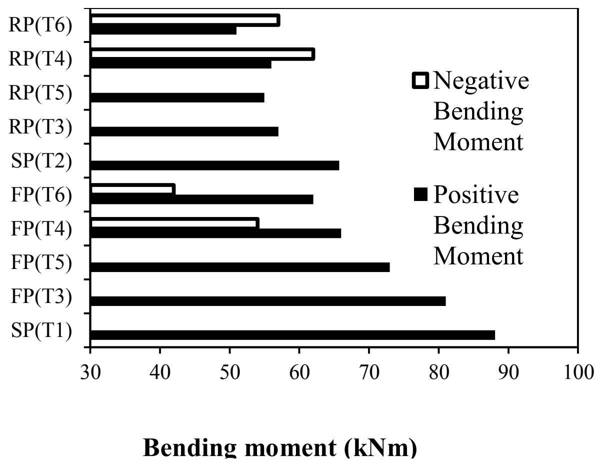
Note: FPC- Computed value for front pile, RPC- Computed value for rear pile, FPM- Measured value for front pile, RPM- Measured value for rear pile.

Figure 4: Pile bending moment for (a) Test 4 and (a) Test 6.

5 SHIELDING EFFECT

Figures 5 and 6 show the comparison of pile head deflection and maximum bending moment, respectively, for rear and front piles in tests T3 and T5 with free head and T4 and T6 with capped head. In these two figures results are also presented for single piles at the corresponding locations for front and rear piles, which are located at 3 m (T1) and 5 m (T2) from the excavation. For tests T4 and T6 with pile caps, bending moment developed at the pile head is negative. Hence both maximum positive and negative moments are given.





Note: SP- Single pile, FP- Front pile, RP- Rear pile.

Figure 5: Comparison of pile head deflection.

Figure 6: Comparison of maximum bending moment.

According to Figure 5, minimum pile deflections are observed, when there is a pile cap. This is due to the high stiffness of the pile cap. The maximum pile head deflection is the same for both front and rear piles. In this case pile tip is pinned. Therefore both piles show the same deformed shape and do not show any shielding effect from the front pile on the rear pile. For tests T3 and T5 with free head piles, front and rear piles deflect less than single piles at 3 m and 5 m away from the wall, respectively. The reduction in deflection in front pile compared to the single pile at the same location is due to contribution from the rear pile in carrying extra loads imposed on the pile group by excavation induced ground deformations. In both tests T3 and T4, rear piles deform less due to the shielding effect from the front piles. When the number of piles in the group increases from 2 to 4, pile deflection further decreased. This decrease is due to arching between piles along the wall. These observations confirm that the presence of more piles tend to increase the shielding and arching effects on the pile group. In this section only 2 m spacing between piles is considered. However, the ratio between pile spacing and diameter may have a significant influence on the shielding and arching effects of the pile group.

Maximum bending moments for the same set of tests also show a response similar to deflections. When the number of piles increased from 2 to 4, maximum bending moment reduces due to arching between piles parallel to the wall in addition to the shielding effect of the pile group for both free head and capped head pile groups. In capped pile groups, maximum negative bending moment is less than maximum positive bending moment for front piles but in rear piles, maximum negative bending moment is higher than maximum positive bending moment. Overall, maximum bending moments developed in front and rear piles in capped and free head piles are less than those developed in single piles at the corresponding locations confirming the shielding and arching provided by the piles in a group.

6 CONCLUSIONS AND RECOMMENDATIONS

In this study, three-dimensional numerical modelling is used to simulate a series of centrifuge tests carried out at the centrifuge facility at the National University of Singapore. The results obtained from the finite element analysis showed good agreement with centrifuge test results for the free-head piles. In the case of capped piles, computed values are very much higher than the measured values. This is due to the higher pile head fixity developed in the finite element model, compared to the welded pile-cap connection formed in the centrifuge test. The presence of front piles tends to reduce the excavation induced moments in rear piles due to shielding effect. In addition when there are piles in a group parallel to the wall supporting the excavation, arching between piles will further reduce the bending moments developed in rear and front piles. In capped–head piles, the maximum lateral movement is less than that of a rear pile with a free-head. The provision of pile cap helps to moderate the excavation induced lateral movements and maximum bending moments in pile groups.

7 ACKNOWLEDGEMENTS

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