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Analysis of pile group behaviour due to excavtion induced ground movements

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ABSTRACT: This paper presents the impact of excavation induced ground movements on adjacent pile groups. Numerical simulations are performed on free-head and capped piles with different pile configurations. The piles, pile cap and retaining wall are assumed to have linear elastic behaviour. Constitutive behaviour of the soil is modelled using the Mohr Coulomb model. Finite element method was used to model a series of centrifuge tests reported in the literature. The predicted and measured values of deflection and bending moment are compared. The problem was modelled in three dimensions to simulate the arching and shielding effect of piles within the pile group. The response of interior piles and peripheral piles are also investigated. It was found that presence of front piles reduce the detrimental effects on the rear piles within the group. Analysis shows the provision of pile gap significantly reduces the deflection of pile group due to load transfer to the rear piles, which are located away from the excavation wall.

1 INTRODUCTION

Excavation adjacent to pile foundations, for the construction of basement, mass rapid transit station and other underground facilities in congested urban areas, are inevitable. During the excavation, due to the stress release in the adjacent soil, the confine pressure around the pile foundations tends to reduce drastically and finally induce additional deflections and bending moments in piles. These detrimental effects should be quantified at the design stage, in order to make sure the stability of the structure against ground deformations due to nearby excavations.

Some case histories (Finno et al. 1991, Goh et al. 2003) were reported in the past, where the pile groups were located adjacent to deep excavations. Numerous studies (both numerical and experimental) have been carried out in the last few decades to study on single pile response adjacent to excavations. Theoretical studies were carried out (Poulos and Chen 1997; Zhang et al. 2011) using the finite element method, boundary element method and finite difference method to find out the pile response due to influence factors such as depth of excavation, support system, soil properties, loading conditions, pile head conditions and pile properties. Similar findings were established from centrifuge tests carried out in sand (Leung et al. 2000) and clayey soils (Ong et al. 2006a).

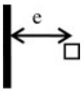
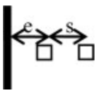
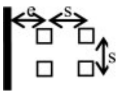
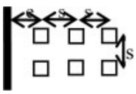
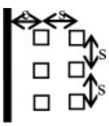
Only few studies have been carried out on the pile group response due to excavation induced movements. Leung et al. (2003) investigated the pile group behavior in sandy soil, during excavation induced ground movements using centrifuge tests, where free-head and capped-head pile groups made of two, four and six piles were considered. Poulos and Chen (1997) studied pile group response due to excavation induced lateral movements by introducing a group factor, which depends on the pile horizontal spacing.

In this paper, finite element method was used to study the pile group behavior during excavations. Centrifuge tests reported in the Ong et al. (2006b) were modeled using three dimensional finite element models. The predicted pile response is compared with measured response and the pile group behaviour is discussed during the excavation.

2 NUMERICAL ANALYSIS

The finite element model used in this analysis is verified against the Centrifuge test results reported in Ong et al. (2006b) which used to investigate the behavior of a pile group in clay, near to an unsupported excavation behind a stable wall. Test was carried out at 50 g at the National University of Singapore geotechnical centrifuge. The model container has prototype dimensions of 27 m × 10 m × 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. Figure 1 shows the variation of undrained shear strength of the clay with depth, obtained using T-bar penetrometer test. The distribution shows that top 2.5 m soil crust was over consolidated and soil below that level was normally consolidated. The soil region that needs to be excavated was replaced by Latex bag filled with $ZnCl_2$ solution which has a unit weight equivalent to the clay. Piles are modelled using hollow square aluminum tubes with outer prototype dimension of 630 mm including an

Table 1. Pile configurations used in the analysis.

Pile Group Configuration	Test No	Values	Pile Head Condition
	T5 T6 T7	e = 3 m e = 5 m e = 7 m	Free head Free head Free head
	T8 T9 T10 T11	e = 1 m, s = 2 m e = 1 m, s = 2 m e = 3 m, s = 2 m e = 3 m, s = 2 m	Free head Capped head Free head Capped head
	T12 T13	e = 3 m, s = 2 m e = 3 m, s = 2 m	Free head Capped head
	T14	e = 3 m, s = 2 m	Capped head
	T15	e = 3 m, s = 2 m	Capped head

epoxy coating, which was applied to avoid the disturbance of the strain gauges instrumented along a pile. The pile is 12.5 m in length in prototype scale and has a bending rigidity of 2.2×10^5 kNm^2 (prototype scale). The wall is made of an Aluminum plate, which has a thickness of 150 mm and a length of 8 m in prototype scale. The wall has a prototype bending stiffness of 2.4×10^4 kNm^2/m. The pile cap was modelled using an aluminium sheet. Here the 1.2 m deep excavation was carried out by draining the $ZnCl_2$ solution at 50 g in six steps over 2 days (proto type scale). Table 1 shows the various centrifuge tests reported by Ong et al. (2006b).

A total stress analysis was carried out assuming undrained behaviour for the clay. The stress- strain behavior of clay was simulated using Mohr-Coulomb criteria. Undrained shear strength of clay can be estimated using the following equation (Ong et al. 2006b), where undrained shear strength values vary linearly with depth.

$$\frac{C_u}{\sigma'_{vo}} = 0.29 OCR^{0.85} \tag{1}$$

where c_u is undrained shear strength of the clay σ'_{vo} is vertical effective stress and OCR is overconsolidation ratio of the clay. The Young's modulus of the kaolin was calculated using $E_c/c_u = 400$. This is a reasonable value for clay under lateral loading as mentioned by Poulos and Davis (1980). The internal friction angle and the Poisson's ratio for the clay were assumed as zero and 0.49, respectively under undrained conditions. Lateral earth pressure coefficient at rest, K_o, was taken as one. The unit weight of the soil is $16.5\ kN/m^3$, which is given by Ong et al. (2006b). The Toyura sand layer below clay was modelled using the

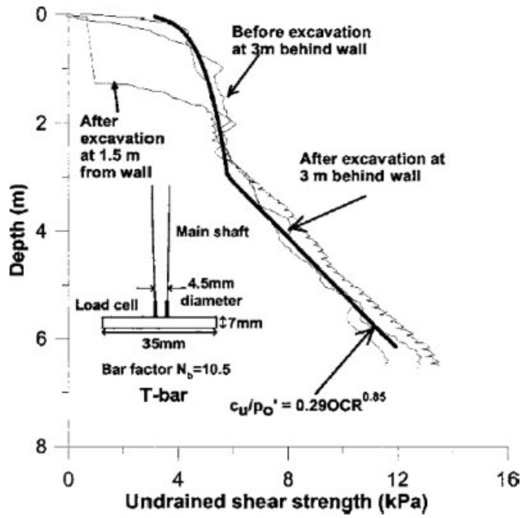


Figure 1. Variation of Undrained shear strength with depth (Ong et al. 2006b).

Mohr-Coulomb model with an internal fricton angle of 40° and a Young's modulus of 6z MPa, where z is the depth below surface (Ong et al. 2006). The Poisson's ratio of the sand is assumed to be 0.3. The pile and wall behaviour were modelled assuming linear elastic behaviour.

The centrifuge tests for different pile configurations were modelled using three-dimensional finite element models according to the prototype scale. ABAQUS/Standard finite element code was used to investigate the problem. Pre-processing and post-processing was carried out using ABAQUS/CAE. Only half of the problem was modelled due to the symmetry of the loading and geometry. The structured meshing technique in ABAQUS was used to mesh the wall, pile and the soil. Swept meshing was used to model the soil region near the pile.

The bottom soil nodes were restrained from movement in all directions ($u_x = u_y = u_z = 0$). Since the grease was applied along all four vertical sides of the container, nodes over these side faces are free to move in the vertical and horizontal directions along the faces of the container and restrained in directions perpendicular to the planes of side faces.

A pinned boundary condition was used at the bottom of the pile. Since solid elements were used for the pile, restraining the movement in all directions at the pile toe will create a fixed boundary condition and finally resulted a high bending moment at the toe of the pile. To avoid this problem, only the bottom center node of the pile was pinned. Coulomb fiction model was used to simulate the soil-pile interaction, which is governed by a friction coefficient. Here a value of 0.3 was selected as the friction coefficient. Results obtained with different friction coefficients allowing slippage and separation at the soil-pile interface show that the behaviour of the laterally loaded pile is not much affected by the friction coefficient. Brown and

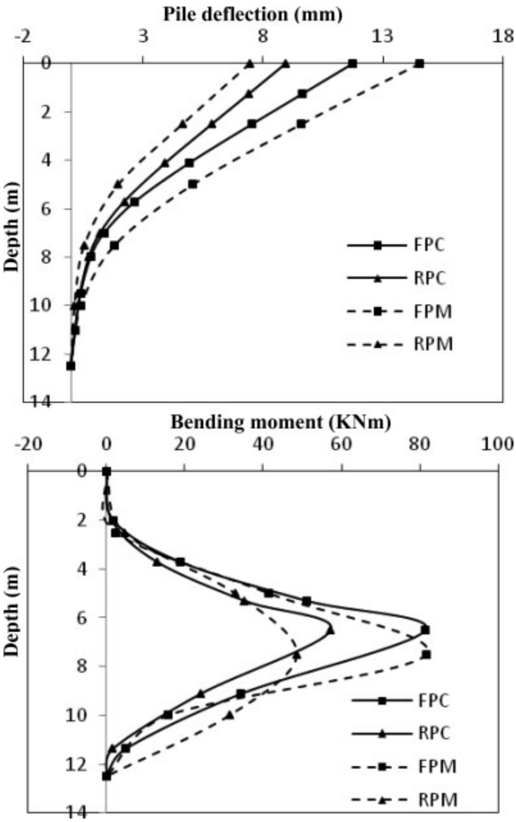


Figure 2. (a) Pile deflection and (b) Pile bending moment for Test 10.

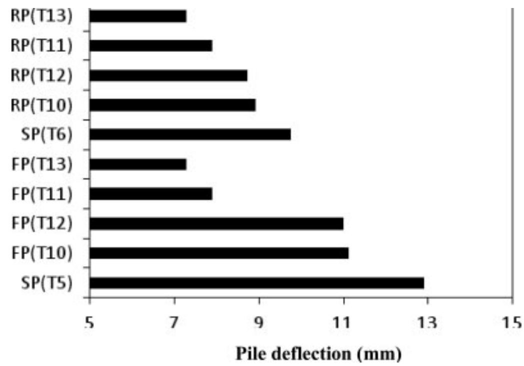


Figure 3. Comparison of pile head deflection.

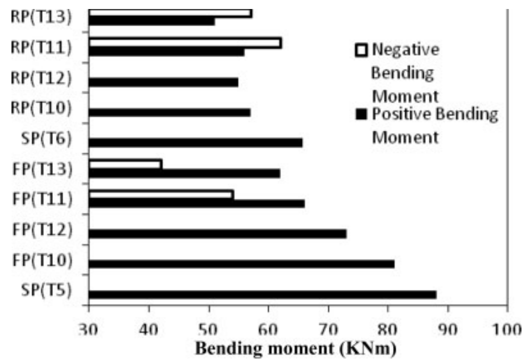


Figure 4. Comparison of maximum bending moment.

Shie (1990) also mentioned that when there is any room for slippage and separation, friction coefficient at the pile-soil interface does not have much influence on the pile behaviour. Another advantage of allowing slippage and separation at the pile-soil interface is that it will avoid the over estimation of the deflection and bending moment along the pile. Piles, wall and soil were modelled using twenty-node quadrilateral brick elements with reduced integration formulation.

3 RESULTS AND ANALYSIS

The deflection of pile towards the excavation is considered as positive. The bending moment induced along the pile is considered as positive, when the curvature is formed towards the excavation. The following notations are also used: SP – single pile, FP – front pile, RP – Rear pile, C – calculated values and M – measured value.

3.1 *Pile group with two piles*

Figure 2 shows the computed and measured pile response for the Test 10 (T10 in Table 1) at the end of 1.2 m depth of excavation, where the two free-head piles are located 3 m and 5 m away from the excavation. Since this is an unbraced excavation, piles deform in cantilever shape. Deflection of front pile, obtained from the finite element analysis is slightly under predicted while the deflection of rear pile is over predicted.

This may be due to the inhomogeneous soil shear strength parameters in the lateral direction. As illustrated in Figure 1, different undrained shear strength profiles were obtained at distance of 1.5 m and 3 m from the excavation after the completion of excavation. The predicted bending moment profile is well agreed with the measured values. The maximum bending moment occurs near the mid span of the pile. The maximum induced moment in the rear pile is 15% less than that of a single pile at the same location as mentioned in Figure 4. This phenomenon reveals that the presence of front pile tends to shield the excavation induced movement on the rear pile and overall deflection also got reduced due to higher reinforcing effect of piles within the group with increasing number of piles. Here only a 2 m centre to centre spacing has been considered. The ratio of pile spacing over pile diameter may have a significant influence on the shielding effect. This should be investigated in detail in the future.

The predicted and measured values of deflection and bending moment along the pile located at a distance 1 m and 3 m from the excavation (T8) are shown in figure 5. The response of rear pile is well predicted using the numerical analysis while the response of front pile is highly under predicted. This discrepancy

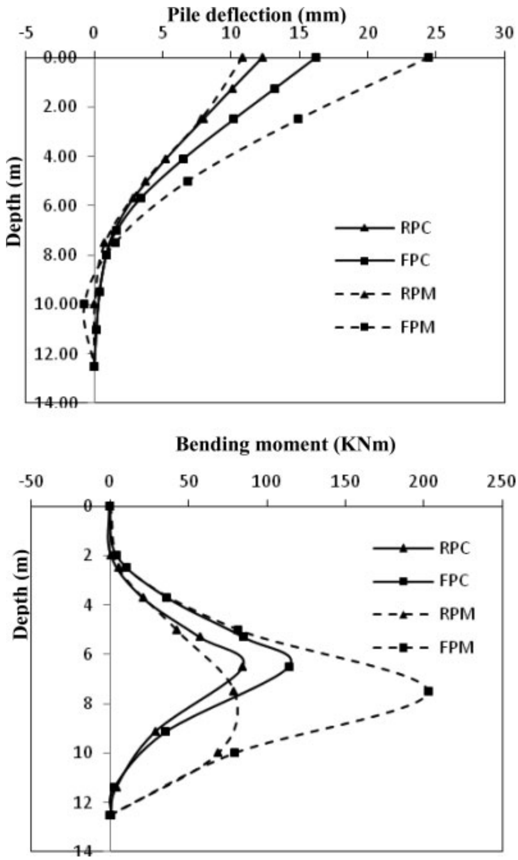


Figure 5. (a) Pile deflection and (b) Pile bending moment for Test 8.

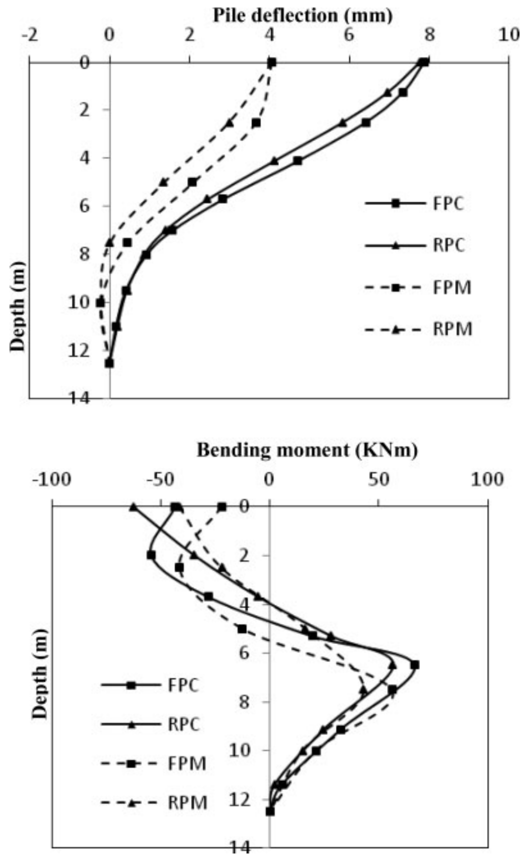


Figure 6. (a) Pile deflection and (b) Pile bending moment for Test 11.

is due to the pile location. The front pile is located within the 1.5 m zone, where the undrained shear strength reduced drastically after the excavation. In this case, there is only 5% reduction in the maximum moment induced in the rear pile due to the presence of front pile located 1 m away from the excavation. Hence, we can conclude that shielding has less influence on lateral pile response when the free head piles are closer to the excavation.

Figure 6 compares the predicted pile response with the measured values for the Test 11. Here a concrete pile cap with planar dimensions of 3 m × 1.25 m and 1.55 m is used. The predicted pile deflection values are very much higher than the values recorded during the centrifuge test. Similar pattern of bending moment profiles are obtained from the numerical analysis and centrifuge test, except the maximum and minimum values.

During the centrifuge test analysis, bending moment values were obtained using strain gauges and the deflection curves were deduced by integrating the bending moment profile twice. Significant negative bending moment values were observed at the tip of the front pile and the rear pile and this value is higher in the rear pile when compared to the front pile. Bending moment profile of the front pile experiences a double curvature. The positive and negative bending moment values are comparable as shown in Figure 6.

Due to rigidity of the pile cap and pinned condition at the toe, similar deflection values were observed for front pile and rear pile. The front pile tends to bulge more compared to the rear one. There is about 40% and 20% reduction in the maximum defection of the front and rear piles, respectively, when there is a provision of pile cap according to Figure 3. As shown in figure 4, when the pile cap is provided, maximum bending moment value in the front pile reduced by 18.5% while the maximum bending moment in the rear pile increased by 9%. This phenomenon indicates that the provision of pile cap is useful to control the lateral movement as well as maximum induced moment.

3.2 *Pile group with four piles*

Figure 7 shows the measured and predicted response of 2 × 2 free head piles (T12) and those are similar to the Test 10 results. The predicted location of maximum bending moment slightly differs from the measured one. Due to the arching effect between the piles in a row, parallel to the wall, the pile deflection and bending moment values are slightly less when compared to Test

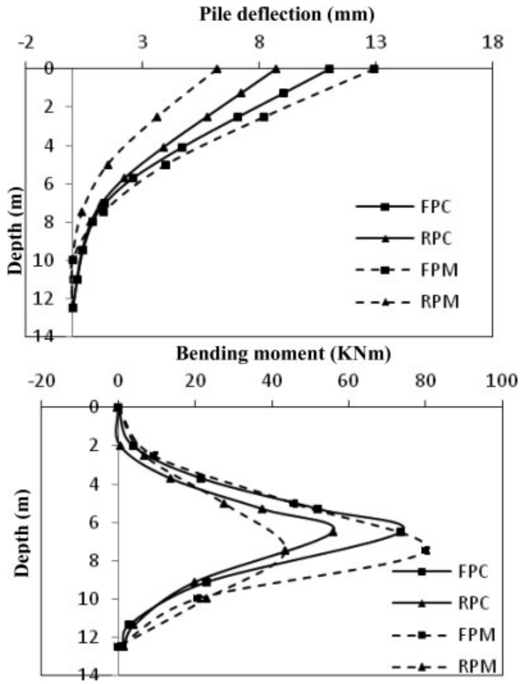


Figure 7. (a) Pile deflection and (b) bending moment for Test 12.

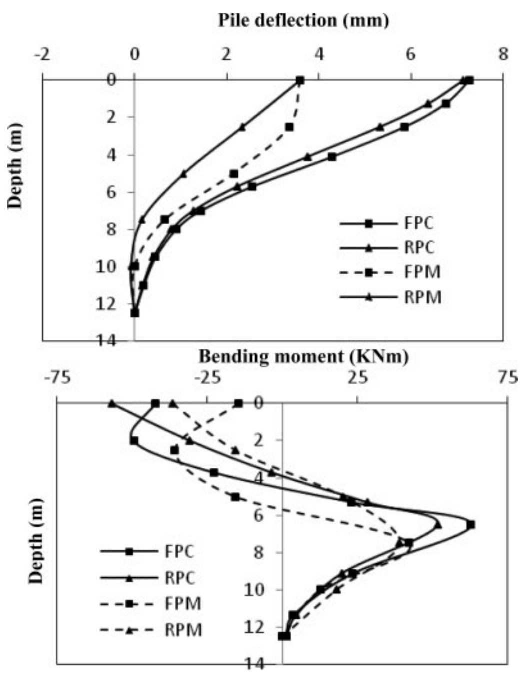


Figure 8. (a) Pile deflection and (b) bending moment for Test 13.

10 (T10), where two piles are located in a line perpendicular to the wall as shown in Figure 4. This difference may be significant when the ratio of pile spacing over pile diameter decreases.

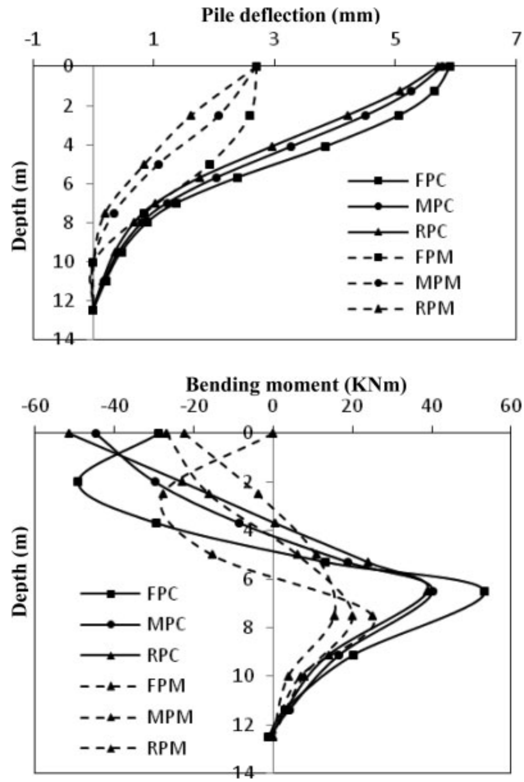


Figure 9. (a) Pile deflection for Test 14, (b) Pile deflection for Test 14.

The predicted and measured values of pile deflection and bending moment for Test 13 are presented in Figure 8. In this case 2 × 2 pile group was capped using a 3 m wide concrete pile cap, which has a thickness of 1.55 m. The front and rear piles are located 3 m and 5 m from the excavation. Finite element analysis highly over predicts the pile response when compared to the results obtained from the centrifuge test. These results are similar to the case with capped-head pile group with two piles. Again a substantial negative bending moment is developed along the upper part of the pile, whereas the positive bending moment induced along the lower part of the pile is smaller than that of free head pile group with four piles. Provision of pile cap leads to a 15% reduction in maximum bending moment induced in front pile and only 3% increment in bending moment developed in the rear pile.

3.3 *Pile group with six piles*

Two capped head pile groups with six piles were analysed as shown in the Table 1. The first case involved with a group of 3 × 2 piles (test 14) connected by a concrete pile cap, which has planar dimensions of 5 m × 3 m and 1.55 m thickness. The induced deflection and bending moment profiles at the end of the excavation for piles located at 3, 5, and 7 m are shown in Figure 9. In this case also predicted pile response values are very much higher than the measured values. It is worthy to note that, in the finite element analysis, the maximum positive moment is induced on the leading pile row, similar to other cases, where as the maximum negative bending moment induced along the upper portion of the rear pile and this is on the contrary to the results obtained for the free head piles, where the maximum bending moment decreases with the pile distance. These results indicate that the pile cap helps to moderate the bending moments induced in the piles within the pile group by transferring part of the bending moment to rear piles. The maximum deflection of the pile cap is also less than that of the single pile located at the rear pile position of the pile group.

4 CONCLUSION

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 in this study. 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. The presence of front piles tends to reduce the excavation induced moments in rear piles. In capped–head piles, the maximum lateral movement is less than that of 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.

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