ANALYSIS OF PILE BEHAVIOR ADJACENT TO A DEEP EXCAVATION

R.Nishanthan, D. S. Liyanapathirana and C. J. Leo
School of Computing, Engineering and Mathematics, University of Western Sydney, NSW 2747, Australia.

ABSTRACT

In urban areas, excavations for cut-and-cover tunnels and basement construction cause detrimental effects on adjacent piles. This paper presents a case study of a deep excavation carried out adjacent to a group of piles consisting of 22 m long step tapered piles. The pile group was located 0.6 m behind a temporary sheet pile wall. The excavation was primarily carried out through the layered sandy soils and supported by a sheet pile. The retaining wall system of the 15 m deep excavation was supported by tie back anchors at three levels. Finite element modelling is used to simulate the staged construction sequence, which involves excavation and installation of the prop system. Constitutive behaviour of the soil is modelled using an elasto-plastic model due to the limited amount of material properties available for the site. The pile deflection and the soil movements from two and three dimensional finite element analyses are compared with field data to determine the most appropriate way of modelling the pile group behaviour adjacent to an excavation.

Keywords: finite element modelling, deep excavation, pile response, construction sequence, soil movements.

1 INTRODUCTION

Due to the ever increasing population, the need for urban construction involving deep excavations for basement construction and underground infrastructures such as mass rapid transit and cut and cover tunnels are increasing. Stress relief caused by deep excavations lead to excessive lateral soil movements. The interaction of these lateral ground movements with nearby existing pile foundations develop additional loading on them. These additional loads will induce extra bending moments and deflections on nearby existing pile foundations and they should be taken into account to ensure the integrity of the foundations as well as the structures supported by them.

Properly instrumented case studies (Finno et al. 1991; Goh et al. 2003) are very useful to gain a clear insight into the problem and to verify the numerical models. Instrumentation of an existing foundation along the pile shaft is very difficult and it is only possible to measure the pile head deflection and the settlement unless the excavation and piling are carried out at the same time within one project (Goh et al 2003). Hence the amount of data available is limited for existing piles during nearby deep excavations. Theoretical studies were also 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, 2003) and clayey soils (Ong et al. 2006).

Numerical analyses are very cost effective when compared to the experimental analysis. The tools like finite element method can be used to simulate the construction sequences, complex support systems, water drawdown, pile-soil interaction etc. Response of single piles and pile groups to the excavation induced ground movements is a three-dimensional problem due to the soil flow between adjacent piles. But the three-dimensional analysis requires high computational effort and time. This paper presents a case study reported by Finno et al 1991. Finite element method was adopted to analyse the case history. The problem was modelled using both two and three dimensional finite element models and compared with the field data as well as the results obtained by Finno et al. (1991) and Poulos and Chen (1997). Finno et al. (1991) analysed the problem using a plane strain model where the soil was modelled using eight-node isoparametric elements and the pile, pile cap and wall were modelled using beam elements. Two equivalent plane strain flexural stiffnesses were assigned based on the pile spacing and pile group spacing. Poulous and Chen (1997) analysed the case study in two steps. In the first step, a finite element code was used to obtain free field soil movements, simulating the excavation in four steps. In the second stage of the analysis, a boundary element method was used to obtain the pile response while applying the free field ground movements obtained from the first step as input. Here, only an isolated pile, located very near to the excavation was considered. The effect of pile cap was incorporated by restraining the pile head rotation. In this paper, a coupled analysis is carried out to obtain the pile group response during the deep excavation. Problem is analysed by considering a two-dimensional plane strain model as well as a three dimensional model.

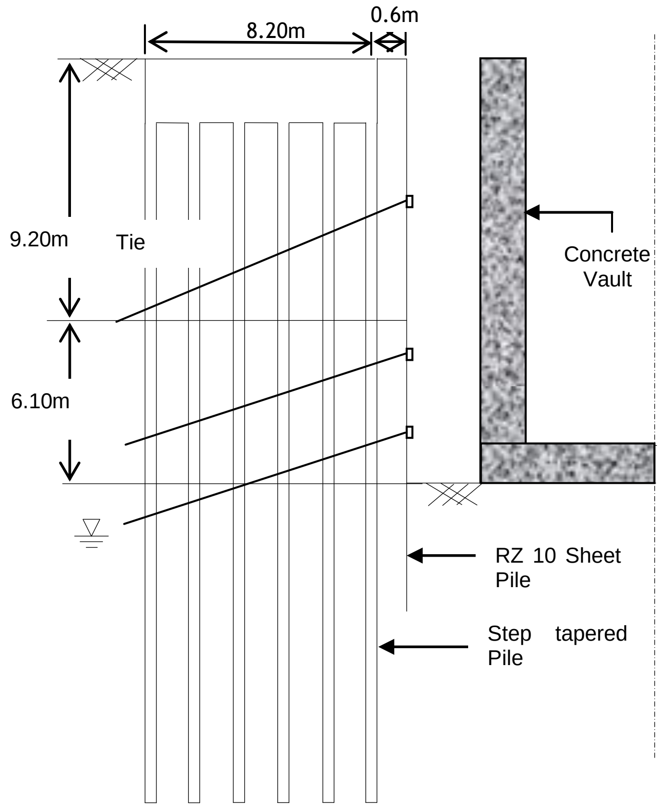

Figure 1. Cross section of the excavation with soil profile

2 PROJECT DESCRIPTION

Excavation was carried out within an existing structure which was supported by a group of piles located 15.25 m apart from each other. The 15 m deep excavation was temporarily supported by RZ10 sheet piles and three levels of tie backs. The excavation was 15.25 m wide and 106.75 m long. The pile cap was located as near as 0.6m to the sheet pile. The piles in the group were 21.35 m long and were made of four by six groups of step tapered piles with diameter of 375 mm at the top and 275 mm at the bottom. Outer piles were reinforced with a No 8 single rebar at the centre. Inner piles were not reinforced. An Intensive monitoring system was used to observe the changes in the surrounding soil domain throughout the excavation.

The site consisted of hydraulically placed sand layer over stiff clay, underlain by dense, alluvial sand as shown in Figure 1. Water table was maintained below the bottom of the excavation using dewatering well points. Soil parameters, obtained through Field SPT N values and limited laboratory tests can be found in Finno et al. (1991). Table 1 shows the equivalent soil parameters proposed by Poulos and Chen (1997) for this field study.

Table 1: Material properties used in the analysis

Material	Parameters
Hydraulic sand fill	E = 10 MPa, φ = 35^0, ψ =20^0, v = 0.45, γ = 17 kN/m^3
Clay and Silt	E = 18 MPa, c_u= 50 kPa, v = 0.45, γ = 19 kN/m^3
Alluvial Sand	E = 60 MPa, φ' = 30^0, ψ = 15^0, v = 0.45, γ = 20 kN/m^3

3 CONSTRUCTION SEQUENCE

Prior to the excavation, 19.5m deep long sheet pile was installed as a temporary retaining structure and the perched water level was lowered below the final excavation level. With the progression of the excavation, three levels of tie backs were installed as shown in Table 2. At the end of the excavation, a concrete vault was constructed and the gap between sheet pile and the wall was back filled and the tie backs were removed, when the back fill reached the tie back level. Finally the temporary sheet piles were extracted.

Table 2: Structural parameters used in the analysis.

Construction day numbers	Description
-	Installation of sheet pile
1-11	Excavation up to depth of 6 m
11-78	Installation of strut at level 1
84-108	Excavation up to depth of 9 m
96-120	Installation of strut at level 2
122-152	Excavation up to depth of 13 m
143-161	Installation of strut at level 2
160-185	Excavation up to depth of 15.3 m

4 FINITE ELEMENT MODELLING

Two- and three-dimensional modelling was carried out using ABAQUS/Standard finite element programme and the results were compared with values measured at the field. Since the main portion of the layer is made of granular soil, fully drained condition was assumed. Constitutive behaviour of both sand and clay was simulated using the Mohr-Columb failure criterion. Simple linearly elastic perfectly plastic model was used to simulate the behavior of the concrete wall. The maximum stress level during the analysis was tracked to ensure that the stress level is always below the yield strength of the concrete. No cracks and rebar configurations were considered. The analysis neglected the effect of installation of sheet pile and considered the wall is “wished in place”. That means no changes in the initial stresses during the construction of the slurry wall.

In the plane strain model, an eight-node quadratic isoparametric element without pore pressure degrees of freedom was used to simulate the soil, wall, piles and pile cap. Two plane strain cases were considered wherein piles were modelled using equivalent wall strips in the transverse direction.

In the first case, the group of piles with the pile cap was considered in the finite element model. Equivalent plane strain stiffness was calculated using the number of piles within the group in the transverse direction (4), pile spacing in the transverse direction (15.375 m) and pile group spacing (xxxx m) to get the lower and upper bound solutions as mentioned in Table 3. In the second case, the pile closer to the excavation was only considered in the finite element model, while neglecting the group effect. Here the equivalent flexural stiffness was calculated by dividing the flexural stiffness of a single pile by pile spacing in the transverse direction. The Finite element mesh used in the analysis is extended 100 m from the centre of the excavation and the bottom mesh boundary is located 13 m below the excavation bottom. Boundary conditions used in the model are as follows:(1) bottom of the soil are restricted from horizontal and vertical movements, (2) far end and along the centre line the mesh is free to move in the vertical direction and restricted to move in the horizontal direction.

In the 3D analysis, both the group of piles and a single pile response were analysed. Due to the symmetry of loading and geometry, a quarter of the model was considered. Soil and pile were modelled using 20 node brick elements. The all boundary conditions mentioned for the 2D plane strain model were used for the 3D case. In addition, bottom, front and back soil movements were restrained in the transverse direction. For single pile analysis, rotation is restrained at the pile head in all directions to simulate the effect of the pile cap. No slip condition was assumed between the soil and the wall and the soil and the pile. In all cases, due to the insufficient data, vertical load transferred from the existing building was neglected as in the previous analyses carried out by Poulos and Chen (1997)

& Finno et al. (1991). However, the vertical loading might have a huge impact on the lateral deformation of the pile. Structural properties of the wall and supports were extracted from Poulos and Chen (1997).The complicated bracing system was idealized due to insufficient data and was modelled as one node spring elements with only one degree of freedom.

Construction sequence was simulated based on the actual site records given in Table 2 and simulated in 20 stages. In the first stage, geostatic, equilibrium is achieved using the initial stresses and the coefficient of earth pressure at rest. During the step by step excavation process, removal of soil and installation of struts were simulated by adding and removing elements, respectively, at the appropriate time of the analysis. Extraction of sheet pile, construction of reinforced concrete vault and back filling were not considered in the current analysis.

Table 3: Structural parameters used in the analysis

Parameters	Values
Young Modulus of concrete	30GPa
Stiffness of the strut	10000 kN/m/m
Flexural stiffness of the wall	11 X 10^4 kNm^2/m
Equivalent Plane Strain stiffness (lower bound solution)	4EI/4.575
Equivalent Plane Strain stiffness (upper bound solution)	4EI/15.375

5 ANALYSIS OF RESULTS

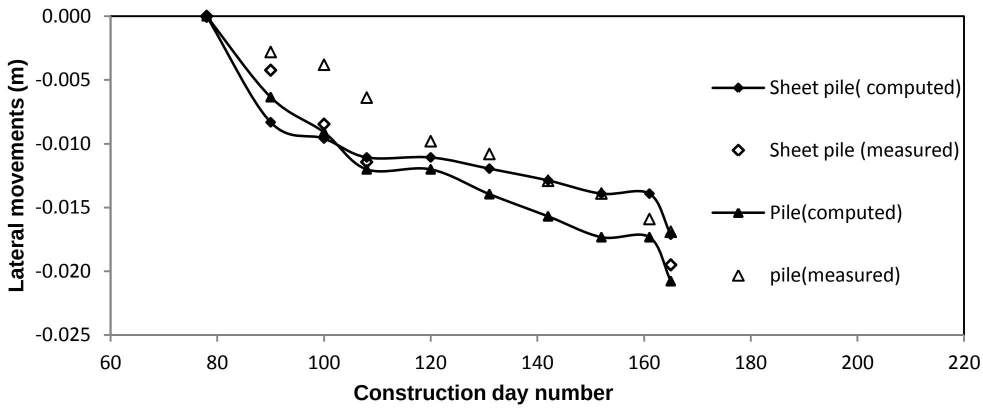

Figure 2. Deflection of pile and sheet pile wall with time

At the time of the installation of the First tie back, 75% of the final wall deformation has already occurred. Figure 2 shows the measured and the predicted deflections of the wall and the pile after the first strut installation, assuming that the deflection at that time is equal to zero. These results are obtained using the plane strain finite element model with the pile group. According to Figure 2, the predictions slightly over predict the measured incremental deformation after the first strut installation. Under estimation of soil strength parameters of the clay layer may be the reason for the over predicted value of incremental deflections.

Wall movements are significantly affected the pile configuration behind the wall. For both plane strain and 3D models with the pile group, sheet pile deformations are almost the same as shown in Figure 3. In the case of single pile (3D), the wall movements are fairly high when compared to the case with the pile group. This discrepancy may be due to the tendency of the group of piles to act as a supportive wall for the existing sheet pile wall. Results with the single pile (3D) are similar to the results reported by Finno et al (1991) for the plane strain model. This may be due to the equivalent soil parameters chosen in this analysis as mentioned in Poulos and Chen (1997). There is not much difference in the pile deflection values for the different cases considered in this study, where the pile group has been modelled as a two dimensional (upper and lower) and a three dimensional problem, as shown in Figure 4.

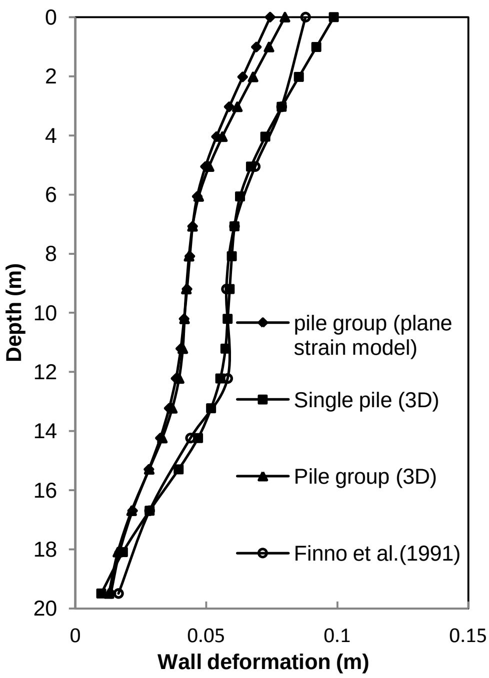

Figure 3. Wall deformation

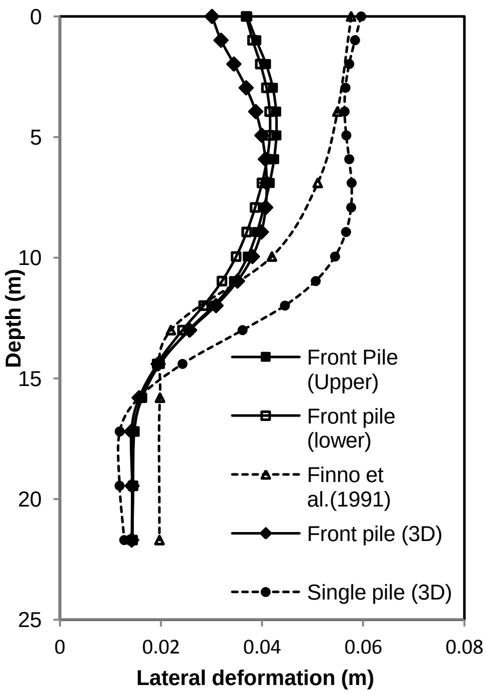

Figure 4. Pile deformation

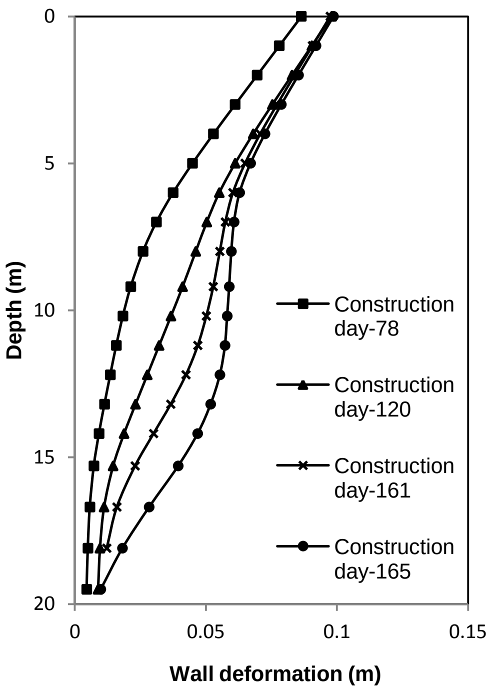

Figure 5. Wall deformation

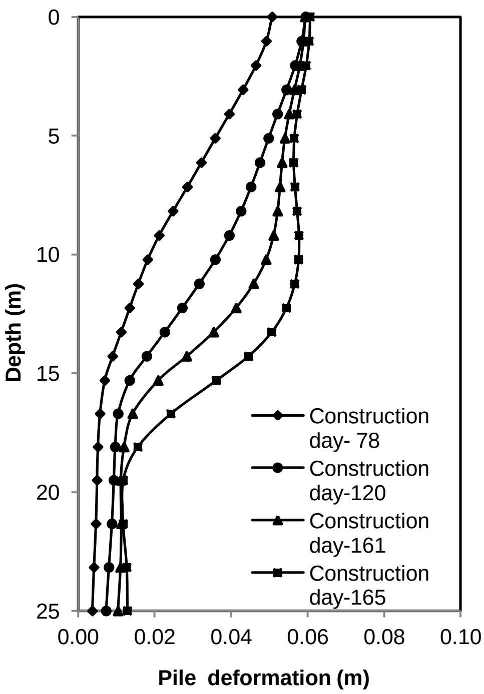

Figure 6. Pile deformation

.

The three fold stiffness difference between upper and lower bound cases does not have much influence on the pile deflection. Even though the deflection values for single pile (3D) shows good agreement with the results reported by Finno et al. (1991), analysis of pile group should provide a more realistic prediction than the single pile analysis. The reason for the contradiction may be due to the soil parameters considered in this paper. They differ from the soil parameters used by Finno et al.

(1991). Hence, we can come to the conclusion that, analysing a single pile from a group of piles while fixing the rotation at the pile head results in high deformation values as shown in Figure 4. In reality, the pile cap tends to moderate the lateral deformation of the front pile, while transferring the moments to the rear piles which are located in the less lateral soil movement zone as mentioned by Leung et al. (2003). The vertical loads transferred from the super structure may be one of the reasons for the low deformations predicted in the three dimensional finite element analysis. The influence of axial loads already applied on the pile from the superstructure before commencement of the excavation are not negligible on excavation induced pile deformations as mentioned by Zhang et al. (2011). Figure 5 shows the wall movements computed using the single pile (3D) model at various stages of the excavation. A significant amount of deformation occurred, when the excavation depth reached 6 m. Cantilever deformed shape of the wall can be clearly observed due to the lower level of first tie back. With the installation of subsequent struts, wall tends to bulge below the first strut level. A similar trend has been reported by O'Rouke (1981). Due to the higher depth of unsupported excavation, the maximum deflection was observed at the tip of the pile and the bottom of the sheet pile was shifted by a small amount. Pile also exhibits similar deflection pattern when the excavation proceeds, except the pile head, which was restrained from the rotation as shown in Figure 6.

6 CONCLUSION

A typical instrumented section of an excavation adjacent to a group of piles was analysed using the finite element method. A two dimensional plane strain model of the problem with the pile group is appropriate to investigation the pile response instead of a three dimensional model of the pile group and the excavation. There is a significant difference between the results for cases with an equivalent single pile and the group of piles. It was found that when the unsupported depth of the excavation is high, maximum wall deflection occurs closer to the tip of the sheet pile wall and the maximum pile deflection occurs at the pile head.

7 ACKNOWLEDGEMENTS

The authors would like to acknowledge the financial assistance provided by the Australian Research Council for this research under the Discovery grant DP1094309.

REFERENCES

Finno, R. J.,Lawrence, S. A.&Harahap, I. S. (1991). "'Analysis of performance of pile groups adjacent to deep excavation." Journal of Geotechnical and Geoenvironmental Engineering,vol. 117, no. 6, pp. 934-55.

Goh, A. T. C., Wong, K. S., Teh, C. I. & Wen, D. (2003), "Pile response adjacent to braced excavation." Journal of Geotechnical and Geoenvironmental Engineering,vol. 129,no. 4, pp. 383-386.

Leung, C. F., Chow, Y. K. &Shen, R. F. (2000), "Behaviour of pile subject to excavation-induced soil movement", Journal of Geotechnical and Geoenvironmental Engineering, vol. 126, no. 11, pp. 947-954.

Leung, C. F., Chow, Y. K. &Shen, R. F.(2003), "Behaviour of pile groups subject to excavation-induced soil movement", Journal of Geotechnical and Geoenvironmental Engineering, vol. 129, no. 1, pp. 58-65.

O'Rouke, T. D. (1981). "Ground movements caused by braced excavation." Journal of Geotechnical and Geoenvironmental Engineering,vol. 107, No. 8, pp. 1159 -1178.

Ong, D. E. L., Leung, C. E. & Chow, Y. K. (2006), "Pile behaviour due to excavation-induced soil movement in clay I:Stable wall", Journal of Geotechnical and Geoenvironmental Engineering, vol. 132, no. 1, pp. 36-44.

Poulos, H. G. & Chen, L. T. (1997), "Pile response due to excavation-induced lateral soil movement", Journal of Geotechnical and Geoenvironmental Engineering, vol. 123, no. 2, pp. 94-99.

Zhang, R.,Zheng, J., Pu, H. & Zhang, L. (2011), "Analysis of excavation-induced responses of loaded pile foundations considering unloading effect. "Tunnelling and Underground Space Technology, Vol. 26, no. 2, pp. 320-335.

Zhang, C.R., Huang, M.S. & Liang, F.Y.(2009), "Geotechnical Aspects of Underground Construction in Soft Ground." Proceedings of the 6th International Symposium, pp. 849-855.