Analysis of Pile Response to Excavation Induced Ground Movements

GEO MANITOBA
BUILDING ON THE PAST

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ABSTRACT
This paper presents a parametric study for the pile response and ground movements due to deep excavations supported by stable retaining walls. The coupled behaviour of a single pile subjected to excavation induced ground movements is analysed using the finite element method, which has the ability to simulate construction sequences and various lateral earth retaining systems. The constitutive behaviour of the soil is modelled using the Mohr-Coulomb model. The pile is assumed to have the linear elastic behaviour. The numerical model was verified using centrifuge test data found in the literature and a parametric study is carried out to develop design charts to establish variation patterns among number of influence factors such as geometry of the excavation, soil properties, structural system variables, pile fixity conditions and pile location with respect to the excavation.

RÉSUMÉ
Cet article présente une étude paramétrique de la réponse du pieu et de mouvements de terrain dus à des fouilles profondes soutenues par des murs de soutènement stables. Le comportement couplé d'une seule pile soumise à des mouvements de terrain induits par excavation est analysé en utilisant la méthode des éléments finis, qui a la capacité de simuler des séquences de construction et les différents systèmes de retenue latérale des terres. Le comportement constitutif du sol est modélisée en utilisant le modèle de Mohr-Coulomb. Le tas est supposé avoir le comportement élastique linéaire. Le modèle numérique a été vérifiée à l'aide des données de test de centrifugeuses trouvés dans la littérature et une étude paramétrique est menée pour développer des abaques de calcul pour établir des modèles de variation entre nombre de facteurs d'influence tels que la géométrie de l'excavation, les propriétés du sol, les variables structurelles du système, les conditions de fixité de pieux et la pile emplacement par rapport à l'excavation.

1 INTRODUCTION

Due to the rapid growth in population, the need for urban construction involving deep excavations for basement construction and underground infrastructure 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.

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. In these kind of situations centrifuge tests play major role in calibration of the numerical model. Properly instrumented case studies (Finno et al. 1991; Goh et al. 2003) are also very useful to gain a clear insight into the problem and to verify the numerical models. 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). The effect of working load applied on the pile head was investigated in Zhang et al. (2011). When performing parametric studies, numerical analyses are very cost effective compared to the experimental modelling. 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. However, the three-dimensional finite element analyses require high computational effort and time. This paper investigates the influence of various parameters described previously using three-dimensional finite element modelling. Prior to the parametric study, the model verification is carried out using centrifuge test results reported by Ong et al. (2006).

2 VERIFICATION OF THE MODEL

2.1 Description of the centrifuge test

The finite element model used in this analysis is verified against the Centrifuge test results reported by Ong et al. (2006).These tests are performed to investigate the behaviour of a single pile founded in clay, near to an unsupported excavation behind a stable wall. The test was carried out at 50g on the National University of Singapore geotechnical centrifuge. The model container has dimensions of 540 mm x 200 mm x 470 mm. The Kaolin clay was filled up to a depth of 130 mm above

Toyoura sand layer which has a thickness of 120 mm. 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 where 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 clay. The hollow square aluminium tube used to represent the pile has an outer diameter of 12.6 mm including the epoxy layer, which was applied to avoid the disturbance of the strain gauges instrumented along the pile. The pile is 12.5 m in length in prototype scale and has a bending stiffness of 2.2 x 10^5 kNm^2/m (prototype scale).The wall used in the centrifuge test is an Aluminum plate, which has a thickness of 3 mm and length of 160 mm. The wall has a prototype bending stiffness of 2.4 x 10^4 kNm^2/m. Here the excavation was carried out by draining the $ZnCl_2$ solution at 50g in six stages over 2 days (prototype scale).

2.2 Material models and properties

Total stress analysis was carried out assuming the undrained condition for clay. The stress-strain behaviour of clay was simulated using the Mohr-Coulomb criteria. The undrained shear strength profile of clay was approximated by the following equation (Ong et al. 2006), where undrained shear strength values vary linearly with depth.

$$c_u / \sigma'_{vo} = 0.29\, OCR^{0.85} \quad [1]$$

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' 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 value of the soil is 16.5 KN/m^3, which is given by Ong et al (2006). The Toyura sand layer below clay was modelled using the Mohr-Coulomb model with an internal fricton angle of 40° and a Young 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 was modelled assuming linear elastic behaviour.

2.3 Finite element modelling

The centrifuge test was modelled using a three-dimensional finite element model according to the prototype scale. ABAQUS/Standard, the 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 in loading and geometry. Figure 2 shows the plan and side view of the finite element mesh used for the analysis. The structured meshing technique in ABAQUS was used to mesh the wall, pile and the soil. Swept meshing was used for the soil region near the pile.

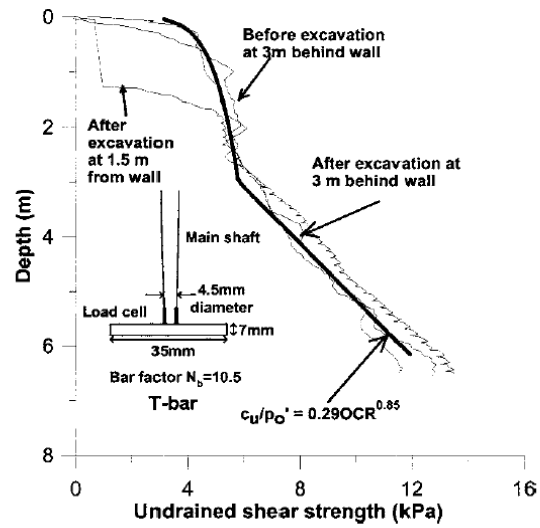

Figure 1. Variation of Undrained shear strength with depth (Ong et al. 2006)

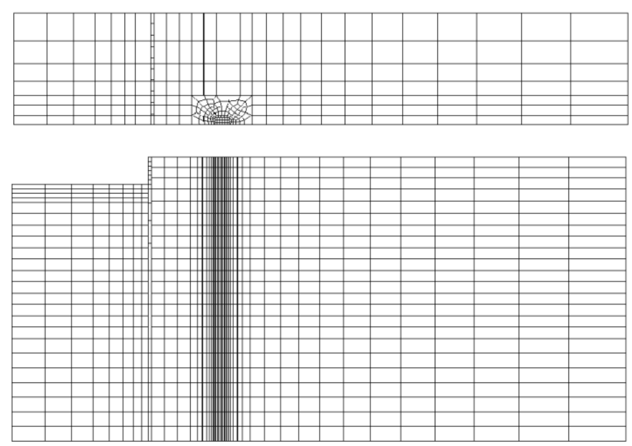

Figure 2. Plan and side view of the finite element used in the analysis.

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 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, resulting in 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 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. Pile, wall and soil were modelled using twenty-node quadrilateral brick elements with reduced integration formulation. Struts were modelled as single node spring elements during the parametric study.

2.4 Results

Figures 3 (a) and (b) show the measured and predicted pile deflections and bending moments at different excavation depths. Even though there are slight deviations in the early stages, the predicted pile deflection and bending moment agree fairly well with the measured values. The main reason for this discrepancy may be the lower prediction of soil shear strength parameters at shallower depths and the pinned condition assumed in the current analysis for the pile toe.

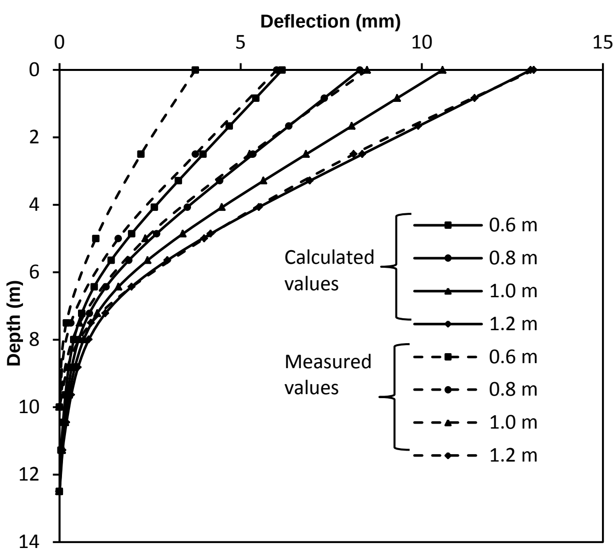

Figure 3(a). Pile deflection

3 PARAMETRIC STUDY

3.1 Scope of the study

This study focuses on the response of a single pile, where the pile is located behind an excavation. The study is conducted using a three-dimensional finite element model and the longitudinal section of the three-dimensional problem analysed is shown in Figure 4. Table 1 shows the variation of parameters for each case analysed.

Bending moment (KNm)

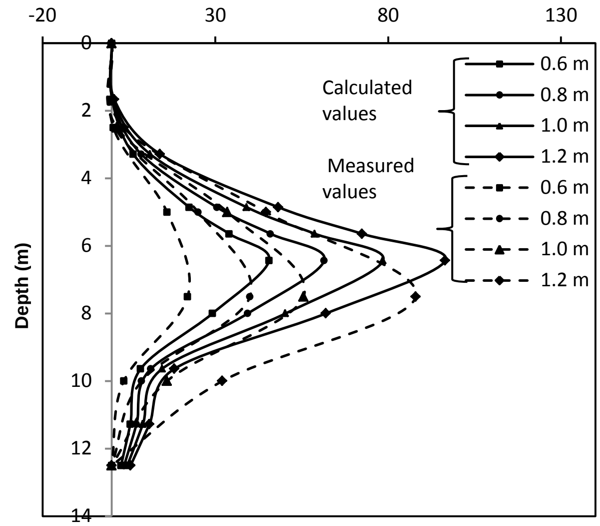

Figure 3(b). Bending moment along pile shaft

The excavation is supported by a 1 m thick diaphragm wall. The square pile is 50 m long and the side width is 1 m. The pile has no restraints at both ends. The soil domain was extended five times the width of the square pile in the lateral direction, from the centre of the pile, and two times the wall length in the vertical direction, from the centre of the excavation to avoid boundary effects on the pile response.

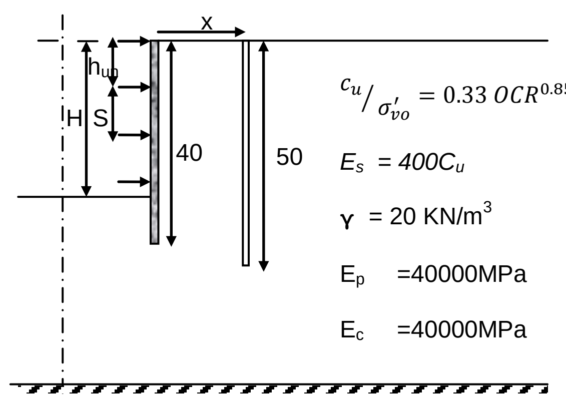

Figure 4. Geometry and the properties used in the analysis

In the longitudinal direction (x direction marked in Figure 4), the soil domain was extended five times the wall length, from the centre of the excavation. The analysis neglects installation effects of the wall and pile. It is noted that, since the geostatic equilibrium was achieved after the installation of wall and pile, the bending moment and deflection obtained were induced only due to the excavation.

Table 1. Parameters used in the analysis.

PARAMETERS	
Depth of Excavation (H) Unsupported Depth (h_{un})	0 – 24 m 0 – 6 m

Spacing (S)	0 – 6 m
Location (X)	1 – 100 m
Over Consolidation Ratio	1,2 & 4

4 RESULTS AND ANALYSIS

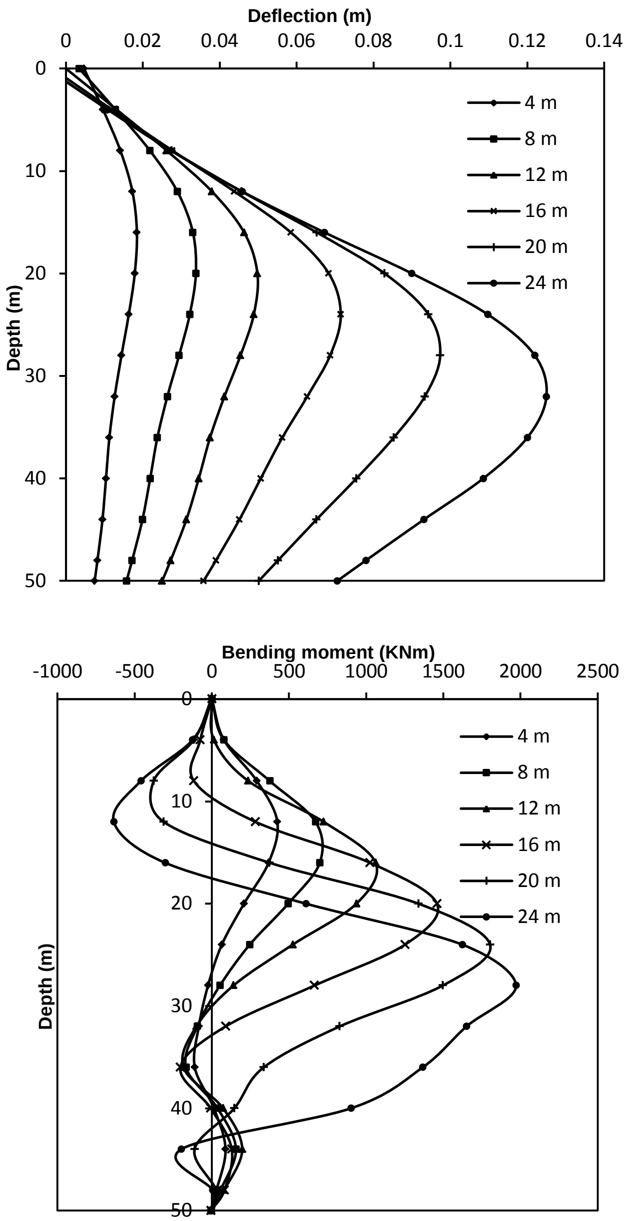

Figure 5. Effect of depth of excavation on (a) pile deflection and (b) bending moment.

Figure 5 shows the variation of pile movement and bending moment along the pile shaft during different stages of the excavation. Pile is located 3 m away from the excavation, which is supported by struts having a vertical spacing of 2 m. The first strut was assumed to be fixed at the surface level. The maximum deflection of pile changes from 0.4%H to 0.5%H when the excavation depth increases from 4 m to 24 m. The maximum deflection of the pile occurs well below the excavation as shown in Figure 5. There is a shift at the pile toe, which is about 60% of the maximum pile deflection. Since the first strut is installed at the surface level after excavating first 2 m depth, the pile has a zero deflection at the top. Figure 5 (b) shows the progression of the bending moment along the pile shaft during the excavation. Maximum Bending moment values are increasing approximately linearly with the depth of excavation, similar to the results obtained by Leung et al. (2000) during centrifuge tests.

When the excavation depth is shallower, the maximum curvature in the bending moment profile is observed in the upper part of the pile. As the excavation depth increases, the upper part tends to bend in the opposite direction due to the support provided by the struts and the maximum curvature in the bending moment occurs where the pile have maximum deflection. At both ends of the pile, very small value of bending moments were observed due to the shear stresses developed along the pile–soil interface. Since this analysis has not considered the working loads applied on the pile in the axial direction, these maximum moments are possible to develop for a particular pile section. Higher the bending capacity, lower is the axial load capacity if the combined pile strength envelop is considered. Hence the working axial load needs to be considered in the future parametric studies.

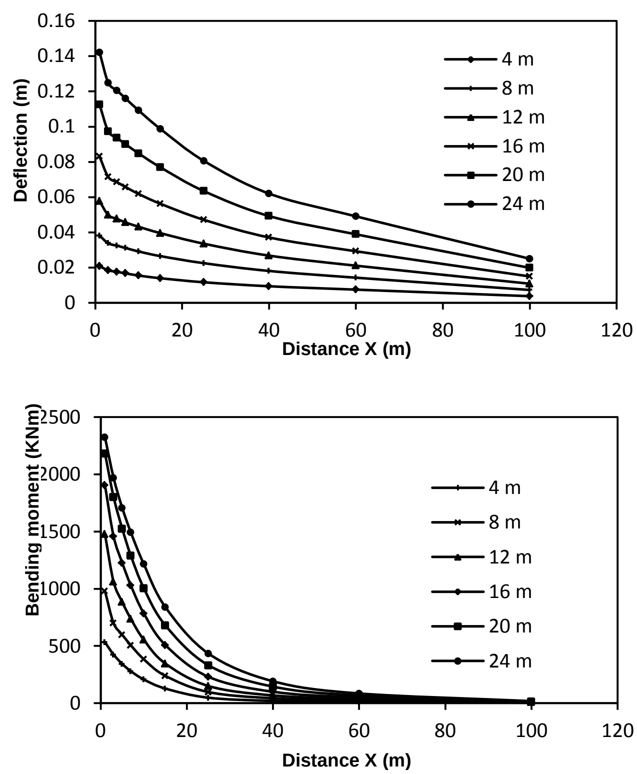

Figure 6. Effect of pile location on (a) pile deflection and (b) bending moment.

Figure 6 shows the variation of pile deflection and bending moment with the pile location at various stages of the excavation. Both the maximum deflection and maximum bending moment decreases exponentially with the distance away from the excavation. The maximum bending moment diminishes to 5% of the maximum bending moment, which can be obtained when the pile is located as close as 1 m from the excavation, at a distance of 8H and 2H, when the excavation was 4 m and 24 m, respectively. But the deflection values are substantial. About 20% of maximum deflection, even at a distance of 100 m away from the excavation, regardless of the depth of the excavation. These results indicate that when the pile is located at a far distance, the total pile is subjected to shifting rather than bending. The similar decaying pattern was observed by Poulos and Chen (1997) and Leung et al. (2000).

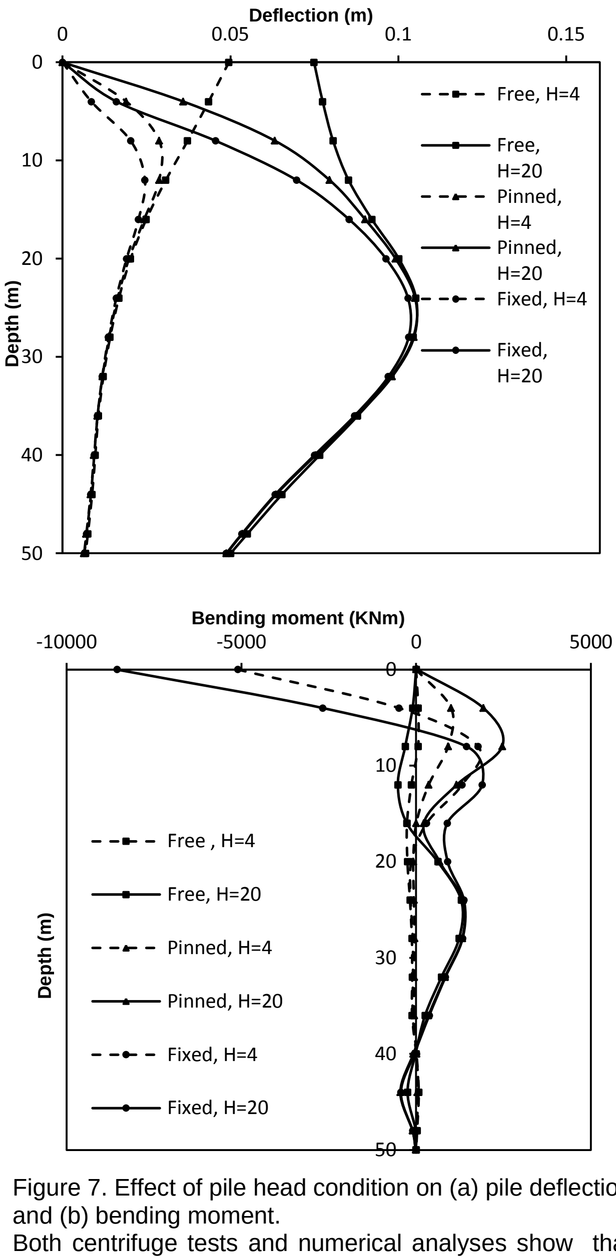

Figure 7. Effect of pile head condition on (a) pile deflection and (b) bending moment.

Both centrifuge tests and numerical analyses show that the pile head fixity condition have a huge impact on the lateral pile response, when the pile is located adjacent to the excavation (e.g., Poulos and Chen 1996, Leung et al. 2000). Hence the effect of pile head condition is investigated in the current analysis through three different boundary conditions at the pile head for translation and rotation: (i) both free, (ii) both fixed and (iii) translation fixed and rotation free. Since the solid elements were used to model the pile, pile head condition with free translation and fixed rotation has not been considered. Figure 7 shows the pile response at the end of 4 m and 20 m depths of excavation for the case with unsupported excavation depth of 6 m. For this case, the pile is 3m away from the excavation. It can be clearly seen that greater the fixity on the pile head higher are the bending moments developed in the upper part of the pile. Due to the nature of the problem considered, high negative bending moment values were obtained, 8.5 MNm and 5 MNm for the very extreme fixity condition at the end of 4m and 20 m excavation depths respectively. These bending moments exceed the flexural capacity of the pile and quite unrealistic compared to the field conditions. The negative end moments are 4.5 times larger than the maximum positive moment at the end of the excavation and a similar trend has been reported by Poulos and Chen (1997). Normally in practice, piles are tied using either pile caps or tie-beams and connected to the superstructure. The pile head connection to the superstructure can be assumed as a pinned or free boundary condition accordiing to the degree of fixity. For all three cases, the maximum positive bending moments do not differ much as mentioned by Poulos and Chen (1997) and Leung et al. (2000). Leung et. al (2000) investigated the effect of pile head fixty condition on pile response, where the pile is located 3 m behind the unbraced excavation. When both translation and rotation are fixed at the pile head, the negative moment observed near the top of the pile was less than the maximum positive bending moment and the bending moment pattern also differs compared to the results obtained at the end of 4 m depth excavation, where the excavation has an unsupported depth of 6 m. This case can be considered as an unbraced excavation. The results for the case with both translation and rotaion fixed at the pile head are similar to the case where the excavation was braced at the surace level.

Stress histories of soil significantly effect the deformation chaaracteristics of the soil and subsequently the deformation characteristics of structures supported by them. Since the Mohr- Columb model was used to model the constitutive behavior of soil, over consolidation ratio (OCR) was used to change the undrained shear strength and elastic stiffness of the soil. Figure 8 shows the effect of OCR on the pile deflection and bending moment. Constant OCR values of 1, 2 and 4 were considered throughout the depth. As expected, for high OCR values the deformations are low. When th OCR changes from 1 to 4 deformation was decreased by 75%. Bending moment curve experience a double curvature when the soil is normally consolidated It shows a single curvature for stiffer clays with OCR values of 2 and 4.

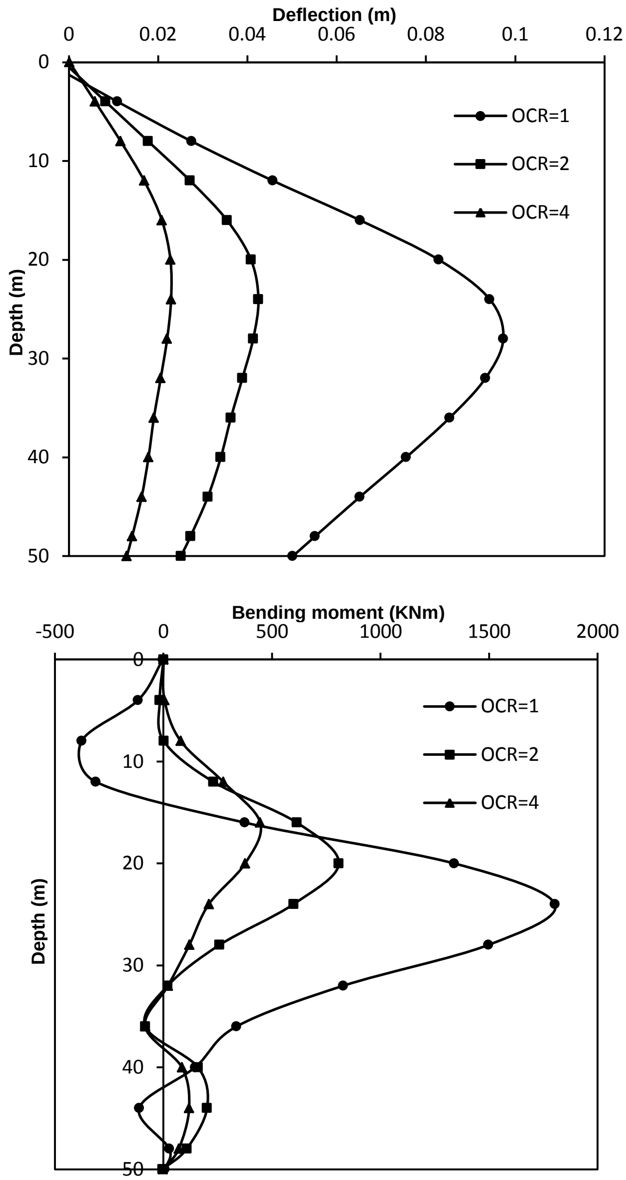

Figure 8. Effect of OCR on (a) pile deflection and (b) bending moment.

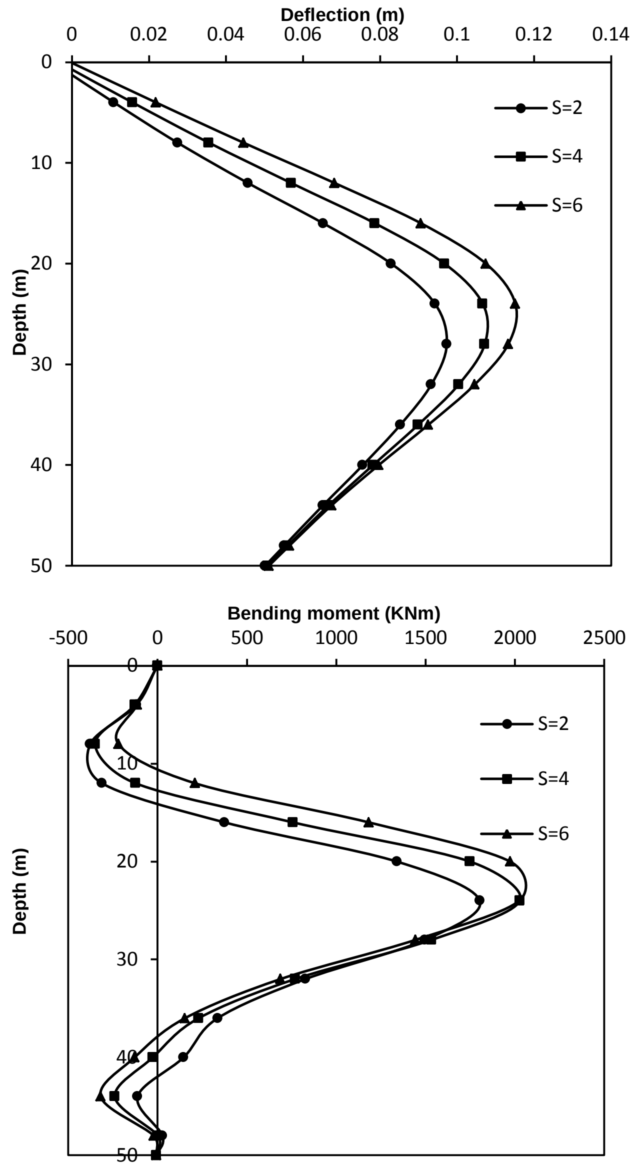

Figure 9. Effect of Vertical spacing on (a) pile deflection and (b) bending moment.

Figure 9 shows the influence of support spacing on pile deflection and bending moment, when the pile is located 3 m away from the excavation. When the vertical spacing of struts changes from two to six meters, there is a negligible amount of increment in deflection (18%) and bending moment (10%) at the end of the excavation (depth of 20 m). This trend is largely different from the results presented by Poulos and Chen (1997), when the pile is located within 5 m from the excavation. According to their results, when there is a change in support spacing from 2 to 6 m, the increment in deflection was 100% and the increment in moment was 15% at the end of the excavation depth, which is 10 m. When the struts are closely spaced, the maximum deflection occurs well below the excavation. However, for the minimal support case, the maximum deflection can be observed near the excavation level.

Similar trend of supporting wall was observed by Hashash and Whittle (1996).

Effect of unsupported depth of excavation on pile deflection and bending moment was investigated for two different stages of the excavation at depths of 8 m and 20 m in normally consolidated clay. Even though the unbraced depth does not have much influence on the maximum lateral deflection of the pile, it has a significant influence on the deflection profile as shown in Figure 10(a). Before the installation of the initial strut, the wall and soil behind it experience higher lateral deformations near the surface level. Since the pile head is located 3 m way from the excavation and free to move, cantilever deformation occurs at the end of the first strut installation. Further bulking occurs with the installation of subsequent struts. Most of the researchers interested about the wall movement and settlement of the soil behind the excavation. The similar pattern of wall movements were observed and investigated by Hashash and Whittle (1996), and O'Rouke (1981).

Figure 10 (b) shows the variation of bending moment. At the end of the 8 m depth excavation, with an unsupported depth of 6 m, less bending moment values were observed. This happens due to the high flexibility of the pile caused by less support and the distribution of bending moment along the shaft. There was a 30% reduction in maximum bending moment values, when the unsupported depth of the excavation was increased from 0 to 6 m.

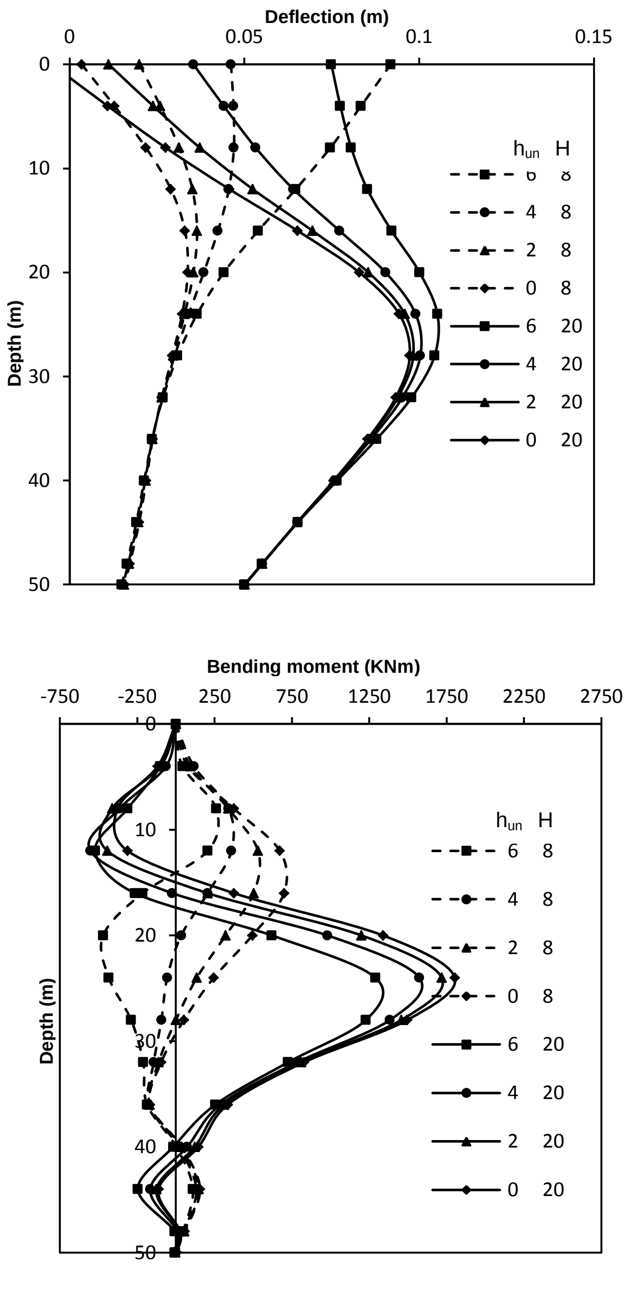

Figure 10. Effect of Unsupported depth of excavation on (a) pile deflection and (b) bending moment.

5 CONCLUSIONS

This study indicates that the three-dimensional finite element modelling has the ability to predict the pile response during the excavation at various stages. Numerical modelling results of pile deflection and bending moment agree well with the centrifuge test results. Both deflection and bending moment of the pile decay exponentially with the distance from the excavation. Even at a far distance from the excavation, pile is subjected to shifting towards the excavation. However, the bending moments are negligible. Pile head fixity has a huge impact on the development of bending moments. Unsupported depth and the stress history of the soil have a significant effect on the pile deflection and bending moment, but the vertical spacing has a minimal impact on pile response.

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