Back analysis of a deep excavation-pile interaction using 3-D finite element analysis

L'arrière analyse d'une action réciproque de tas d'excavation profonde en utilisant la 3ème analyse d'élément finie

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ABSTRACT
In the course of underground Mass Rapid Transit construction in Singapore, there was a cut and cover excavation that ran parallel to an existing expressway viaduct supported by piles. Engineers are concerned on the effects of excavation-induced soil movement on the adjacent existing pile foundations supporting the viaduct structure and had set stringent deflection limit for the pile caps during and after construction. This paper focuses on the bending moment and deflection responses of the 4-pile groups supporting the viaduct adjacent to the deep excavation. Back analysis was conducted using 3-dimensional finite element analysis and the calculated results were compared against the instrument observations. The results of the comparison reveal that the finite element analysis can predict the pile deflection reasonably well.

RÉSUMÉ
Au cours de la construction de Transport en commun rapide souterraine de Masse en Singapour, il y avait une coupe et une excavation de couverture qui a été parallèle à un viaduc d'autoroute existant soutenu par les tas. Les ingénieurs sont concernés sur les effets de mouvement de sol incité d'excavation sur les fondations de tas existantes adjacentes soutenant la structure de viaduc et avaient mis la limite de déviation stricte pour les bonnets de tas pendant et après la construction. Ce papier se concentre sur le moment de torsion et les réponses de déviation des groupes de 4 tas soutenant le viaduc adjacent à l'excavation profonde. L'arrière analyse a été accomplie en utilisant l'analyse d'élément finie 3 dimensionnelle et les résultats calculés ont été comparés contre les observations d'instrument. Les résultats de la comparaison révèlent que l'analyse d'élément finie peut prédire la déviation de tas raisonnablement bien.

Keywords : Excavation, pile, interaction, instrumentation, finite element analysis

1 INTRODUCTION

As excavation results in stress relief in the surrounding soils, lateral soil movements are induced towards the excavation. Such soil movements induce additional bending moments and deflections on adjacent piles and this may lead to failure of the piles or the pile deflection may exceed the desired specified limits (Bransby and Springman, 1996). Engineers are concerned on the effects of excavation-induced soil movement on adjacent existing pile foundations supporting a viaduct structure that runs parallel to a cut and cover excavation during the course of Mass Rapid Transit construction in Singapore, as shown in Figure 1.

There are various ways for the analysis of pile responses subject to such excavation-induced soil movements. One possible way is to use two-dimensional (2D) finite element (FEM) analysis and approximate the piles as plate element with "smeared" properties. However, uncertainties remain on how to derive the appropriate "smeared" properties of the piles in this approach. Another way is to use 2D FEM analysis to obtain the free-field soil movement at the location of the piles which is then fed to another software to take into account the pile-soil interaction. However, the most robust and realistic way is to use 3D FEM analysis to directly capture the actual geometry of the problem and the proper pile-soil interaction.

In the present case study, the piles were installed about 2 years before the commencement of excavation. Hence no instruments could be placed in the piles to measure the induced bending moments and deflections. Using the results of instruments such as inclinometers placed in the soil and the diaphragm wall as well as 3D prisms measuring pile cap movements as a benchmark, a back analysis was carried out using PLAXIS 3-D FOUNDATION Version 2.2 (2007) in order to gain insight into the responses of pile foundations due to excavation-induced soil movement.

The four-pile groups supporting Piers 37, 38 and 39 shown in Figure 1 have been chosen for the present study. This is because Pier 37 which is at a distance away from the excavation is not expected to show significant movements while Piers 38 and 39 are expected to move towards the excavation. To ensure the structural integrity of the viaduct will not be compromised, Piers 38 and 39 are only allowed to deflect by a small magnitude.

2 SITE CONDITION

Figures 1 and 2 show the plan and section views, respectively, of the excavation, which is is 17.8m deep and comprises of 2 different cross-section of 18.45m and 7.725m wide. The excavation is supported by 21.2m deep diaphragm walls, which is 1m thick along the 18.45m wide cross-section and 800mm thick along the 7.725m cross-section. Six levels of steel struts were installed to support the diaphragm wall. The 4-pile groups supporting the piers consist of 1.4m diameter bored piles with a centre-to-centre pile spacing of 4.2m extending to a depth of about 23m. Their pile caps are 5.9m by 5.9m and 2.1m thick. The soil profile was inferred from five boreholes located within the area of study, as shown in Figure 2. The ground water level is 0.5m below the ground surface. The plan layout of the piers and instrument layout at the site are also shown in Figure 1. The measured results obtained from soil inclinometers IS 4007 and IS 4013, wall inclinometer IW 4010 and 3D prisms XYZ 4938 and XYZ 4939 were used to compare with the FEM calculated results of soil and structure movements.

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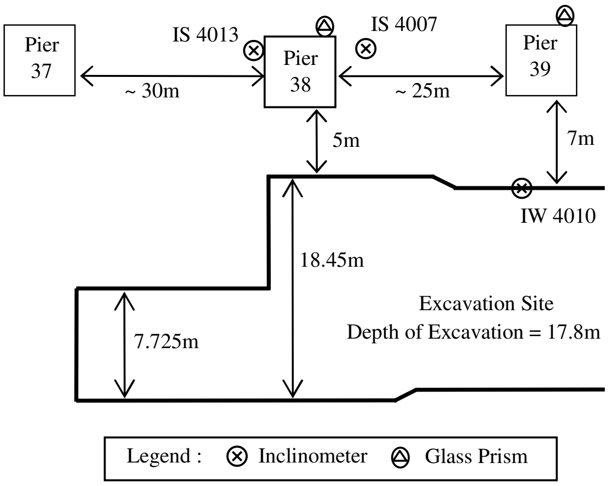

Figure 1: Plan view of excavation and instrumentation layout

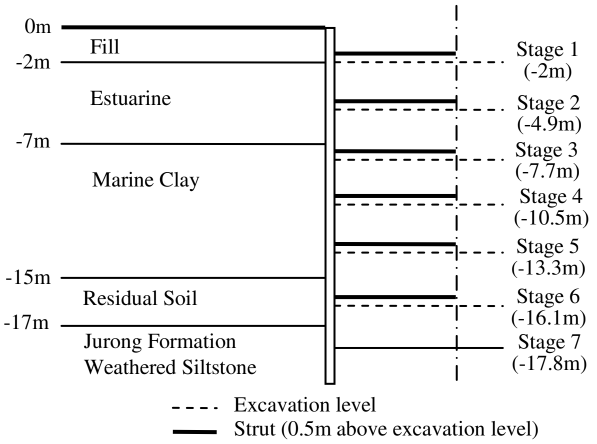

Figure 2: Section view of excavation and soil profile

3 OBSERVED INSTRUMENT RESULTS

3.1 *Wall deflection*

For the wall inclinometer IW 4010 placed in the diaphragm wall in front of Pier 39, about 23 mm wall deflection was observed prior to Stage 4 as excavation was conducted in soft marine clay. Figure 3 show the deflection profile obtained from wall inclinometer revealing insignificant increment of wall movement between excavation Stage 4 and Stage 7. This is not unexpected as excavation was conducted in stiffer soils from Stage 5 onwards.

3.2 *Soil movement*

Soil inclinometers, IS 4007 and IS 4013 were placed at a perpendicular distance of about 10m away from the edge of excavation, see Figure 1. The lateral soil movement profiles obtained from the two soil inclinometers at excavation Stage 4 and Stage 7 are shown in Figures 4 and 5. Unlike the wall movements which occurred till a depth of about 21m, soil movement generally occurred within the top 15m of the soft marine clay layers with relatively small soil movements in the bottom stiff soils of Jurong Formation beneath 17m depth. As before, when the excavation reaches the stiffer soil layers, the soil movements increase only minimally after Stage 4.

The results also reveal that the relative movements of soils are higher further away from the pile group which illustrates the arching effects of the soil due to obstruction of the pile group and its cap. The 1.4m diameter bored piles having much higher shear strength compared to surrounding soft clay and the piles have restrained the soil movements around them. However, this would induce additional bending moment on the piles.

3.3 *Pile cap movement*

From the 3D prism measurements (Figures 6 and 7), there was only a slight increase in lateral pile cap movement between Stage 1 and Stage 2. After Stage 2, the lateral pile cap movement increased considerably. This is due to the corresponding larger soil movements as excavation was then made in the soft soil layers. Between Stage 4 to the last excavation stage, the wall and soil movements and pile cap movements all stabilized as excavation was made in stiff soils. The pile cap at Pier 39 deflected with a maximum magnitude of approximately 7mm while the pile cap at Pier 38 deflected with a maximum of about 9mm. Prism measurements of Pier 37 showed that there was negligible movement till the end of excavation.

4 3D FINITE ELEMENT ANALYSIS

Coupled Consolidation analysis was conducted for the 3D FEM analysis to take into account the excavation activities and time effects. The walls and struts were modelled using linear elastic beam elements while the pile cap and piles were modelled using solid elements. The hardening soil model was used to model the soil behavior as it facilitates automatic changes in the soil stiffness during unload-reload. Hardening soil stiffness parameters can be obtained using equations (1) and (2) (Plaxis manual)

$$E_{50} = E_{50}{}^{ref}\left(\frac{c'\cos\phi' - \sigma_3'\sin\phi'}{c'\cos\phi' + p^{ref}\sin\phi'}\right)^m \tag{1}$$

$$E_{ur} = E_{ur}{}^{ref}\left(\frac{c'\cos\phi' - \sigma_3'\sin\phi'}{c'\cos\phi' + p^{ref}\sin\phi'}\right)^m \tag{2}$$

where p^{ref} = 100kPa,
σ_3' = Average σ_x' of soil layer,
E_{50} = E' in Mohr Coulomb model, and
E_{ur} = E_{ur}' in Mohr Coulomb model.

It should be noted the input reference soil stiffness parameters $E_{50}{}^{ref}$ and $E_{ur}{}^{ref}$ refer to those under an effective confining pressure of 100kPa. For soil at various depths with different confining pressures, the input soil stiffness must be adjusted accordingly before input into the Plaxis program in order to derive the accurate soil stiffness parameters.

4.1 *Pre-loading struts*

The present version of Plaxis 3-D Foundation does not support the function of pre-stressing the struts. In order for the struts to be pre-loaded, horizontal point loads were applied at the points where the struts are connected to the diaphragm wall. The loads are first activated before the struts are activated. The next stage consists of de-activating the set of clusters that represent the soils to be excavated in the next excavation stage. The pre-load is also de-activated at this stage. This process continues until the final stage where the excavation has reached the formation level.

4.2 *Results*

4.2.1 *Wall deflection*

Figures 3 shows the wall deflection profile as well as the maximum wall top movement could be reasonably well replicated by the 3D FEM analysis.

4.2.2 *Soil movement*

From Figures 4 and 5, it can be seen that the general shape of the FE results are similar to the instruments. For IS 4013, FE results match to within 3mm. For IS 4007, the results match reasonably well at stage 4. However, at stage 7, there is a bulge within 4m to 10m in the instrument results which is not matched by FEM. This occurs at the area where the soft soils are and the reason for the bulge in inclinometer reading may be due to a relatively weaker soft soil at that area.

4.2.3 *Pile cap movement*

Figure 6 shows that the FE calculated pile cap movement is generally in good agreement with the results of prism XYZ 4938 although it slightly over-predicts the pile cap movement at Stage 4 onwards. From Figure 7, although the FEM analysis tends to over-predict the pile cap movement at Pier 39 for the initial stages, such over-prediction tends to become smaller from Stage 4 onward. The discrepancy between the calculated and measured values for the final Stage 7 is less than 2mm.

4.2.4 *Pile deflection and bending moment*

The piles nearer to the excavation site are labelled as front piles while the piles further away are labelled as rear piles. The predicted deflection and bending moment results for Stage 4 and Stage 7 for the front and rear piles are presented in Figures 8 to 11, respectively. The pile responses predicted from the 3D FE analysis show that the pile responses did not change significantly from Stage 4 to Stage 7. This is due to the fact that the later excavation stages are in the stiffer soil layers and do not cause much soil movements.

4.2.5 *Arching effect*

One major advantage of 3D analysis is that the magnitude of lateral soil movements of the pile group and its vicinity can be deduced. Figure 12 shows the lateral soil movements at 5m and 10m depths for Stage 7 of the excavation, along a line parallel to the wall at the front edge of the pile cap. There is a gradual transition in the magnitude of soil movement just outside of the pile cap. The extent of the reduced soil movement zone extends about 5m from the centre of the pile cap. It is evident that due to obstructions of the pile and pile cap, the lateral soil movement at the pile cap is significantly less than the free field soil movement without the pile cap. This arching effect of the soil and soil-structure interactions is reasonably represented with the 3-D FEM model.

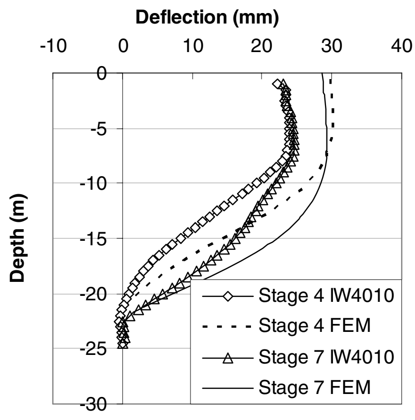

Figure 3: Wall deflection profile in front of Pier 39

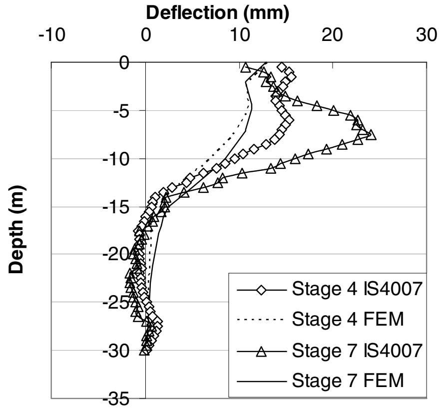

Figure 4: Graph of measured and predicted soil movement at IS 4007

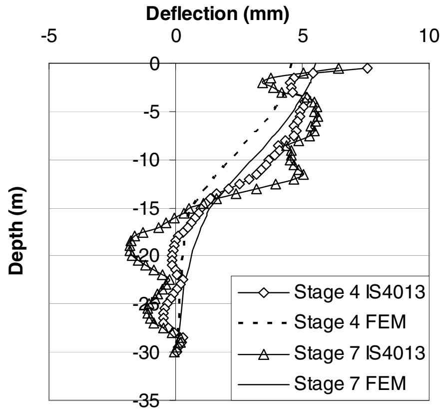

Figure 5: Measured and predicted soil movement profile at IS 4013

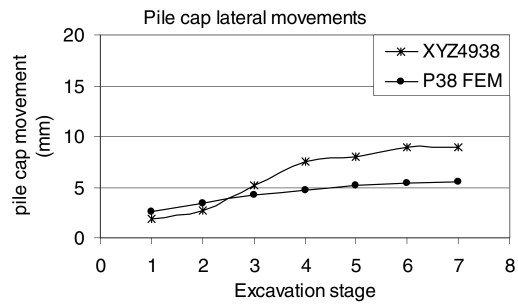

Figure 6: Pile cap movements at Pier 38

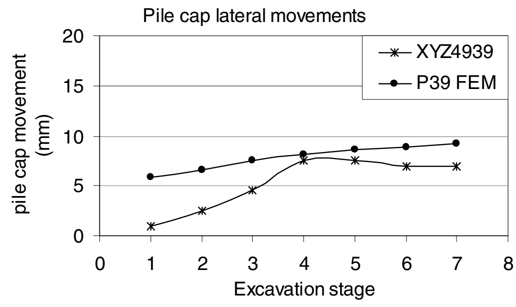

Figure 7: Pile cap movements at Pier 39

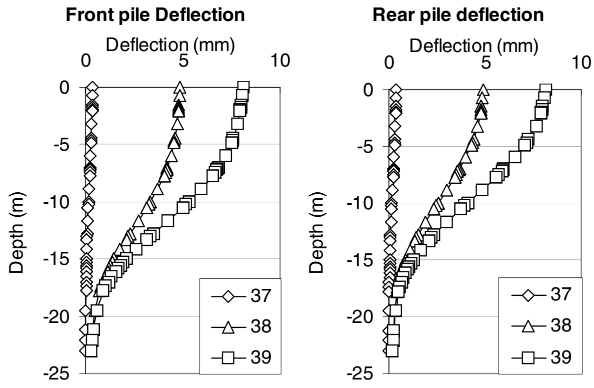

Figure 8: Graph of pile deflections at excavation Stage 4

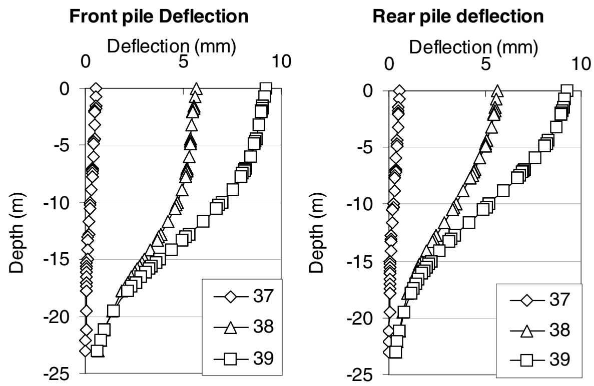

Figure 9: Pile deflection profiles at excavation Stage 7

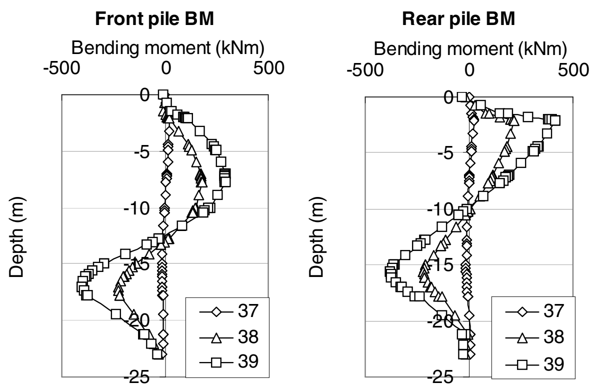

Figure 10: Pile bending moment profiles at excavation Stage 4

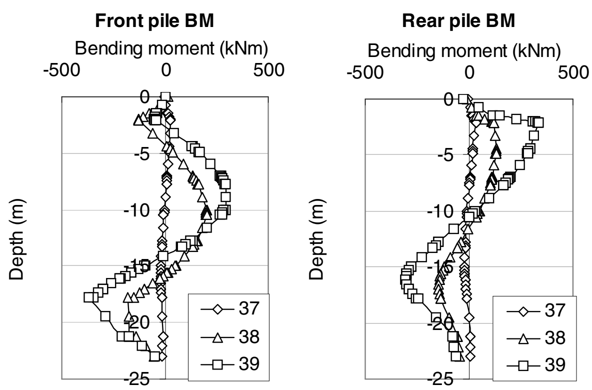

Figure 11: Pile bending moment profiles at excavation Stage 4

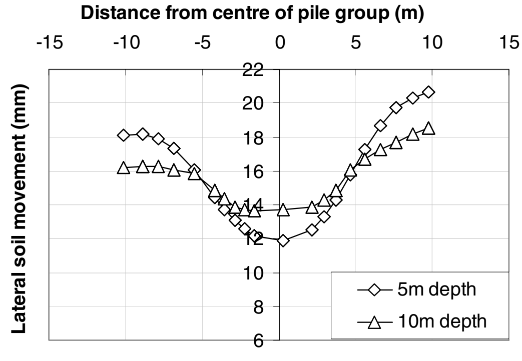

Figure 12: Soil movements around pile group at pier 39

5 CONCLUSION

3D FEM analysis was used to back analyse the response of four-pile groups due to excavation-induced lateral soil movement. The FE analysis matched the measured wall deflection, ground surface movements at various distances behind the wall and the pile cap deflection reasonably well. The arching effect of the soil and soil-structure interactions can also be reasonably represented with the 3-D FEM model. As the piles were not instrumented, the bending moments and deflection values calculated from the analysis is able to provide engineers an idea of how the piles respond in the field and provide reassurance that the integrity of the piles is not compromised.

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