Numerical analysis for deep excavation of Taipa Central Park in Macau

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ABSTRACT

The objective of this paper is to evaluate the performance of three constitutive soil models, i.e. the Morh-Coulomb model (MC model), the Hardening Soil model (HS model), and the Lade model (LD model), implemented in PLAXIS, for predicting ground movements induced by a deep excavation. A case history of deep excavation in Macau was adopted for the numerical analyses in this study. Site investigation data and empirical equations were used to determine input parameters. The observed data were compared with the calculated results at different phases. The comparison showed that the HS and LD models yield better predictions of the wall deflections, while the MC model gives less favorable results.

Keywords: deep excavation; SMW; wall deflection; constitutive model; PLAXIS

1 INTRODUCTION

Deep excavations are often very close to existing buildings in urban areas. As a result, they usually cause uncomfortable movements, which can affect the safety of adjacent buildings. Wall and ground displacements induced by deep excavations have been studied by many researches, for example Peck (1969), Hsieh *et al.* (2003), Kung *et al.* (2009), Hsiung (2009), Lim *et al.* (2010), and Ng and Lok (2011).

This paper aims to evaluate the performance of three constitutive soil models, i.e. the Morh-Coulomb model (MC model), the Hardening Soil model (HS model), and the Lade model (LD model), for predicting displacement induced by a deep excavation. In particular, the Lade model was implemented into PLAXIS as a user defined model in this study.

2 A CASE HISTORY OF DEEP EXCAVATION

A case history of deep excavation in Taipa Central Park, Macau was adopted for numerical analyses in this study. The scope of work consists of building an underground car park and a garden, equipped with various facilities. The shape of the site was rectangular with 280 m in length and 100 m in width.

The construction started in September 2009 with a 9.9 m deep excavation. An initial excavation of 1.5 m in depth was performed with mild slope around the construction site. Soil Mixing Wall (SMW), consisting of soil cement columns of 850 mm in diameter and 18 m in depth, with HN-700x300x13x24 steel beams inserted into the center of alternate columns to resist earth pressure, was constructed around the site. The axial stiffness (EA) and flexural (EI) stiffness of the SMW are 5564000 kN/m and 335000 kN·m^2/m, respectively. The excavation was then carried out using the top-down method with the permanent floor slabs supporting the retaining wall as the excavation progressed downwards in three stages. The stiffness of floor slabs is given in Table 1. Fig. 1 shows the cross section and the corresponding ground conditions. The groundwater table before excavation was about 2.0 m deep below the ground surface, and was lowered to a depth of 0.5 m below the excavation level at each excavation stage.

Table 1: Stiffness of floor slabs

Strut level	Section area (m^2)	EA(kN/m)
G/F	0.2	9990000
B1	0.2	7080000

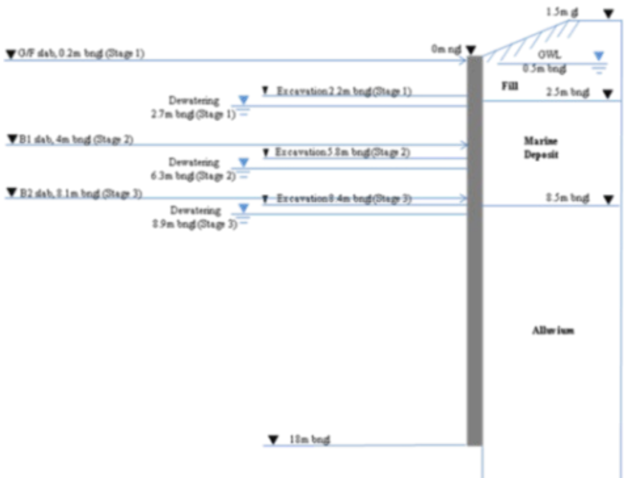

Fig. 1. Cross section and ground condition

3 NUMERICAL ANALYSES AND RESULTS

3.1 Morh-Coulomb Soil model simulation

The commercial software PLAXIS 2D, version AE (2012) , which is a product of the PLAXIS BV company, was used for numerical analyses in this study. Fig. 2 presents the finite element model. Only half of the excavation was modeled because of its symmetry. The base of the finite element model was placed at the top of Completely Decomposed Granite (CDG), i.e., at a depth of 32 m below the ground surface. The distance from the right vertical boundary of the model to the retaining wall was taken to be 50 m. The horizontal movement was restrained for the lateral boundaries, and both the vertical and horizontal movements were restrained for the bottom boundary of the model. The diaphragm wall and the floor slabs were simulated by elements of plates and fixed-end anchors, respectively.

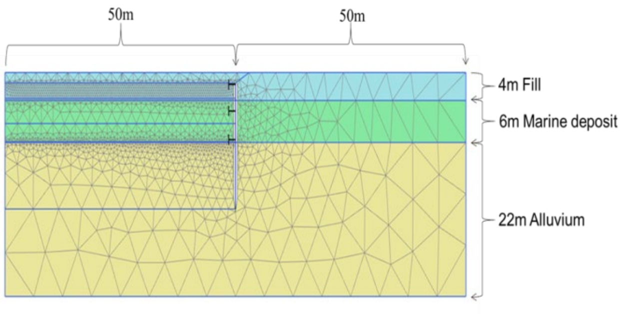

Fig. 2. Finite element model adopted in numerical analyses

The excavation was mainly in marine deposit, but the influence of the underlying alluvium is also significant. Three constitutive soil models, i.e. MC, HS, and LD model, were adopted to simulate the soil layers for evaluating their performances in predicting the wall deflections induced by the excavation. The duration for each construction phase is more than one to two hundred days, so that all soil layers are assumed to be drained condition for this study.

The drained Young's modulus is estimated using a simple empirical equation of E (MPa) = f • N, where N is the SPT-N value, and the coefficient 'f' is typically within 1 (MPa) to 4 (MPa), which is calibrated with case histories of excavation. The variation of SPT-N values with depth is presented in Fig. 3.

The values of 'f' and the corresponding Young's Moduli for each soil layer are shown in Table 2. Table 2 also shows the effective friction angles estimated based on the correlations with SPT-N and verified by laboratory tests. The cohesion of soil is assumed to be equal to 0.5 kPa to avoid numerical problems, and the Poisson's ratio is estimated based on the past construction experience.

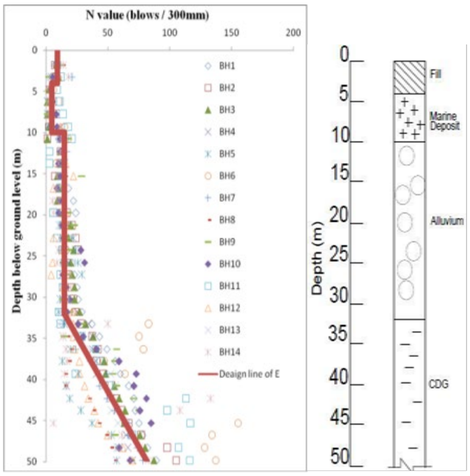

Fig. 3. Variation of SPT-N values

Table 2: Material parameters of Mohr Coulomb model

	Fill	Marine Deposit	Alluvium
c'(kPa)	0.5	0.5	0.5
γ_d(kN/m3)	18	18.35	18.6
γ_{sat}(kN/m3)	21	21.35	21.5
ν	0.3	0.3	0.25
φ'(degree)	30	28	33
f	4	1.5	2.5
E'(kPa)	36000	6900	25000~50000

Using the material parameters in Table 2, analyses were carried out to provide the calculated wall displacements (MC) shown in Fig. 4 together with the upper limit (UL) and lower limit (LL) of observed wall displacements from the inclinometers.

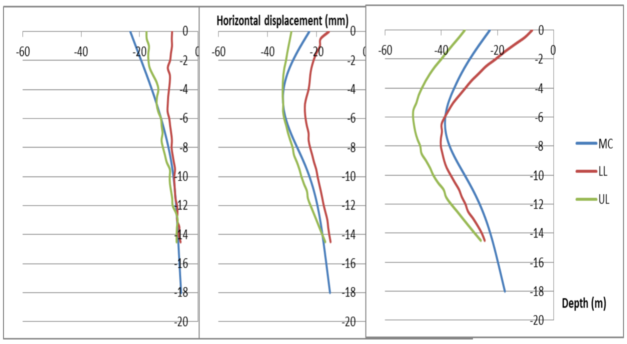

Fig. 4. Wall horizontal displacement vs depth with MC model

From Fig. 4, the magnitude and the trend of the calculated displacements are comparable to the observation. In general, the calculated displacements are smaller. As the stiffness of the soil in the MC model is constant, which does not reflect the nonlinear behavior of soil, it may lead to the smaller displacements comparing with the observations.

3.2 Hardening Soil model simulation

HS model is an elastoplastic model, in which the stiffness varies with the stress state. Comparing with MC model, the HS model can model soil nonlinear behavior more effectivelloky. Fill is simulated by MC model. For the MC and the HS models, strength parameters are the same, but the stiffness parameters are stress dependent in the HS model. Additional material parameters for the HS model are shown in Table 3. According to the recommendation in PLAXIS manual E_{ur}^{ref} and E_{oed}^{ref} are assumed to be $3E_{50}^{ref}$ and E_{50}^{ref}, respectively.

Table 3: Additional material parameters of Hardening Soil Model

	Marine Deposit	Alluvium
ν_{ur}	0.2	0.2
E_{ur}^{ref}(kPa)	6900	36750
E_{50}^{ref}(kPa)	2300	12250
E_{oed}^{ref}(kPa)	2300	12250
m	0.5	0.5
P^{ref}(kPa)	100	100

Analyses were performed using the parameters in Table 3, and the results are presented in Fig. 5.

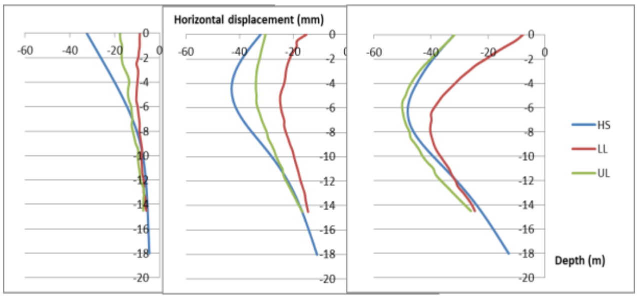

Fig. 5. Wall horizontal displacement vs depth with HS model

Comparing the calculated results with observations, in Stage 1 and 2, the calculated maximum displacement is larger than the observation, but the position of maximum value is close. In stage 3, the prediction is very close to the upper limit.

HS model is a non-linear model. It can reflect that the stiffness becomes smaller at higher stress level, which leads to calculation of larger wall deflection.

In addition, HS model can simulate the loading and unloading condition. Although the soil mainly unloads in excavation, some area is still in loading condition.

3.3 Lade model simulation

In this analysis, Fill is simulated by MC model. In addition, marine deposit and alluvium are modeled by LD model.

For the LD model, its strength criterion is different from the MC criterion, and the stiffness is also stress dependent. In the LD model, the unloading-reloading modulus is determined by K_{ur}, which can be calculated according to the Young's modulus in MC model. The soil parameters are presented in Table 4.

Analyses were performed using the parameters in table 4, and the results are presented in Fig. 6.

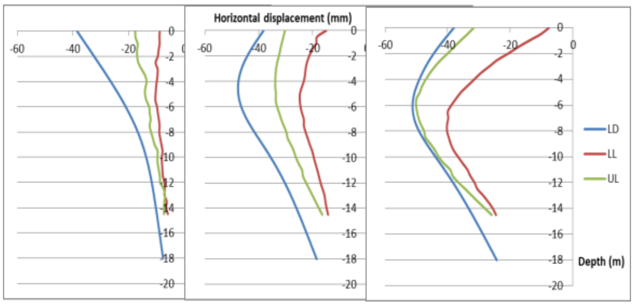

Fig. 6. Wall horizontal displacement vs depth with LD model

Table 4: Material parameters of Lade Model

	Marine deposit	Alluvium		
		1	2	3
K	200	360	297	266
v	0.2	0.2	0.2	0.2
n	0.6	0.6	0.6	0.6
C	2.8e-4	2.8e-4	2.8e-4	2.8e-4
p	0.94	0.94	0.94	0.94
η_1	16	17.18	17.69	18.00
m	0.093	0.093	0.093	0.093
s	0.43	0.43	0.43	0.43
t	0	0	0	0
R	-1	-1	-1	-1
α	3	3	3	3
β	-0.076	-0.076	-0.076	-0.076
P	0.11	0.13	0.12	0.11
l	1.25	1.25	1.25	1.25
ξ	1e-4	1e-4	1e-4	1e-4

Comparing the results with the HS model, the LD model calculates larger displacements. Examination of stress-strain response shows that with increasing strain, the LD model exhibits slight strain softening behavior, but the HS model does not have this problem, which may lead to larger displacements.

3. CONCLUSION

Numerical analyses were conducted for a deep excavation project using three constitutive models. In general, the more advanced soil model is adopted in the numerical analyses, the better predictions of the wall deflection are obtained from the analyses. This study shows that the HS model and LD model are better than the MC model for analysis of deep excavation.

At the stage 1, the calculated displacements from these 3 models deviate greatly from the observation. In general, the deformation at small strain cannot be modeled accurately by these three models. On the other hand, the comparison improves at latter stages.

Comparing the HS and LD models, the LD model gave larger displacements, probably due to the slight strain softening behavior in the model. However, for further study, more detailed calibration of the material parameters should be carried out for these advanced models with laboratory tests.

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