Influence of Soil Models on Numerical Simulation of Geotechnical works in Bangkok subsoil

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Abstract

In the present paper, series of analyses of geotechnical works in Bangkok subsoil using three soil models with different levels of complexity are carried out by FEM and discussed with observed data. The analyses presented include pile load test, deep excavation and tunneling. The results obtained by using Mohr-Coulomb model, Hardening Soil model, Hardening Soil model with small strain stiffness are compared. The impacts of the chosen constitutive models on numerical analysis of such geotechnical works are shown. It can be concluded that using more sophisticated constitutive models which includes non-linear pre-failure and high stiffness under very small strain considerably improves the movement prediction.

1. Introduction

In dense urban environments where land is scarce and buildings are closely spaced, tunneling and cut-and-cover excavations are widely used for basement construction and development of underground facilities. One of the main design constraints in these projects is to prevent or minimize damage to adjacent structures. Since all buildings are supported by foundations, to eliminate or reduce the possibility of such damage, an effective method is needed to accurately ''predict" the excavation-induced movement of building foundations for such complex condition. The magnitude of the settlement and lateral movement and their distributions depend on a large amount of factors, such as soil profile and its geotechnical engineering properties, stiffness of structure and support system. The finite element method (FEM) is often used to predict ground movements induced by such soil-structure interaction problems. The interaction between existing structures and underground activities is a complex phenomenon in which the behavior of the surrounding ground is one of the main aspects to be taken into account. Consequently, a reasonable ground model is crucial in order to estimate the magnitudes and distribution of the strains. The constitutive model frequently used in numerical simulation of an underground work is linear elastic perfectly plastic with a Mohr–Coulomb (MC) failure criterion. The greatest advantage of MC is that only five parameters, which includes two elastic parameters (i.e., Young's modulus E and Poisson's ratio ν) and three plastic parameters (i.e., friction angle φ, cohesion c and dilatancy angle ψ), are sufficient in describing the plastic behavior. Moreover, the parameters can be easily determined. However, the model does not take into account the fundamental aspects of soil behavior, such as variation of modulus according to stress state and different modulus in loading and unloading conditions. Therefore, in general, the numerical results by MC are in good agreement with those of field observation at a certain strain range for each case. To achieve good agreement with the measured values, the parameters are usually obtained from back calculation by curve fitting with history records. Although this technique

RECEIVED 4 March, 2009

ACCEPTED 12 June, 2009

could improve the accuracy of prediction in practice, it is only applicable for specific range of each work. To overcome such shortages, it is necessary to consider at least an elasto-plastic model with isotropic hardening. In addition to previously mentioned aspects, the non-linear pre-failure initial stiffness of soil at small strain range is necessary for analysis of some geotechnical earthworks [1]. Figure 1 illustrates the reduction of stiffness with increasing strain and typical strain ranges of some geotechnical works.

In this paper, three constitutive models with enhancing levels of complexity are used to simulate three types of geotechnical works (deep excavation, tunneling and pile load test) in Bangkok subsoil condition. First, the selected models are described and the calibration of models on the basis of the extensive in situ test results for stiff clay and laboratory test results for soft clay are carried out. The impact of selecting a soil model for geotechnical work analysis on accuracy of the predictions of soil displacements is highlighted.

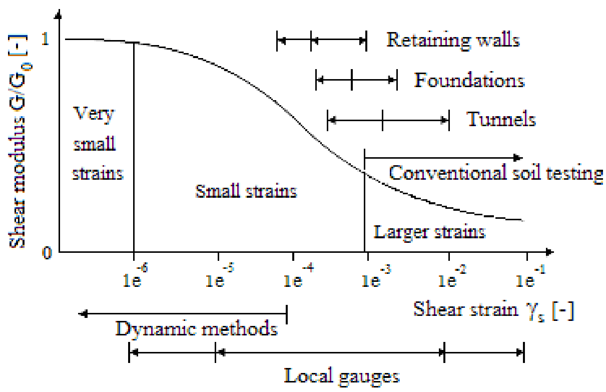


Figure 1 Characteristic stiffness-strain behavior of soil with typical strain ranges for laboratory tests and structure [2].

2. Soil models

2.1 Mohr-Coulomb model (MC)

The linear elastic perfectly plastic behavior, so-called the Mohr-Coulomb Model (MC) was used with its five parameters as mentioned above. Since it is a well-known model, the details will not be described herein.

The Mohr-Coulomb yield condition is an extension of Coulomb's friction law to general states of stress. In fact, this condition ensures that Coulomb's friction law is obeyed in any plane within a material element. The full Mohr-Coulomb yield condition consists of six yield function when formulated in terms of principal stresses [3]. The yield surface in principal stress is shown in Figure 2.

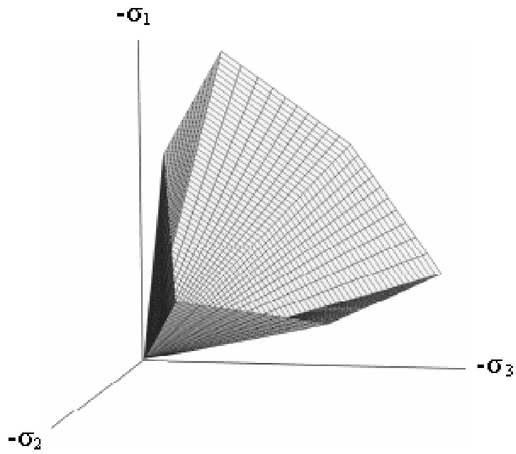


Figure 2 The Mohr-Coulomb yield surface in principal stress space (c=0) [4]

2.2 Hardening soil model (HS)

The hardening soil (HS) model is derived from the hyperbolic model of [5], with some improvement on the hyperbolic formulations. The model can be adapted to all types of soil. The input parameters are as follows:

• resistance parameters including friction angle, φ, the cohesion, c, and the dilatancy angle, ψ

• stiffness of the soil defined by parameters E_{50}, E_{oed}, which governs the volumetric behavior (where E_{50} denotes for secant modulus at 50 percent of maximum stress, E_{oed} is the slope of stress-strain curve obtained from oedometer test) and E_{ur} the unloading–reloading modulus, and a Janbu [6] type parameter *m* that governs stress dependent stiffness according to a power function.

• yield function, f which has the following formulation

$$f = \overline{f} - y^p \tag{1}$$

where $\overline{f}$ is a function of stress and hardening parameters calculated by

$$\overline{f} = \frac{1}{E_{50}^{ref}} \cdot \left(\frac{c \cdot \cot\varphi + \sigma_3}{c \cdot \cot\varphi + p^{ref}}\right)^m \cdot \frac{q}{1 - \frac{q}{q_a}} - \frac{2q}{E_{ur}^{ref}} \cdot \left(\frac{c \cdot \cot\varphi + \sigma_3}{c \cdot \cot\varphi + p^{ref}}\right)^m \tag{2}$$

and y^p is a function of plastic strains. The parameter q denotes deviatoric stress, E_{ur}^{ref} is the unload-reloading modulus at reference stress and q_a is the ratio of ultimate deviatoric stress to failure ratio.

$$y^p = \varepsilon_1^p - \varepsilon_2^p - \varepsilon_3^p = 2\varepsilon_1^p - \varepsilon_v^p \tag{3}$$

•The cap-type yield function, f^{cap} which has the formulation

$$f^{cap} = \frac{q^2}{M^2} + (p + c \cdot \cot\varphi)^2 - (p_p + c \cdot \cot\varphi)^2 \tag{4}$$

where

$$M = \frac{6 \cdot \sin\varphi}{3 - \sin\varphi} \tag{5}$$

and p_p is isotropic pre-consolidation stress.

2.3 Hardening soil with small strain stiffness model (HSsmall)

The HSsmall model constitutes an extension of the HS model. All the features of the HS model are included in the HSsmall model [7]. In addition to the HS model, the HSsmall model incorporates a formulation of small-strain stiffness. Many authors have studied the behavior of soils using high precision triaxial tests. They obtained a reversible behavior and high stiffness for strains less than 10^{-5} and showed that the shear modulus was constant under very small-strain (strains between 10^{-6} and 10^{-5}). This behavior is described in the HSsmall model using an additional strain-history parameter and two additional material parameters, i.e., G_0 and $\gamma_{0.7}$. G_0 is the small-strain shear modulus and $\gamma_{0.7}$ is the strain level at which the shear modulus has reduced to 70% of the small-strain shear modulus.

2.4 Small Strain Stiffness

Soil stiffness is an important soil parameter when ground deformations in engineering earthworks are analyzed. Soil stiffness exhibits strong non-linearity. The strain dependency of the stiffness is an important parameter to predict the precise ground deformation at small strain level. Jardine [8] reviewed the overall ground movement with typical working load from many case histories such as shallow foundations, piles, excavations and tunnels, in various type of soils. It was shown that the ground strains mostly range between 0.001 to 0.5 percent. According to case histories reported, the strains in the ground of hard soil are mostly less than 0.1 percent and the largest strain is 0.5 percent.

3. Bangkok subsoil conditions and calibration of soil parameters

Bangkok Soft Clay was deposited in marine conditions at the delta of the rivers in the Chao Phraya Plain. The typical Bangkok subsoil is shown in Figure 3. The soil properties used in the analyses are mainly determined from local investigated data correlation from comprehensive in-situ tests [9] and previous laboratory tests [10-13]. Figures 4 and 5 show the examples of in situ data for strength and coefficient of lateral earth pressure of Bangkok soils from previous mass transit projects [9] adopted in this study.

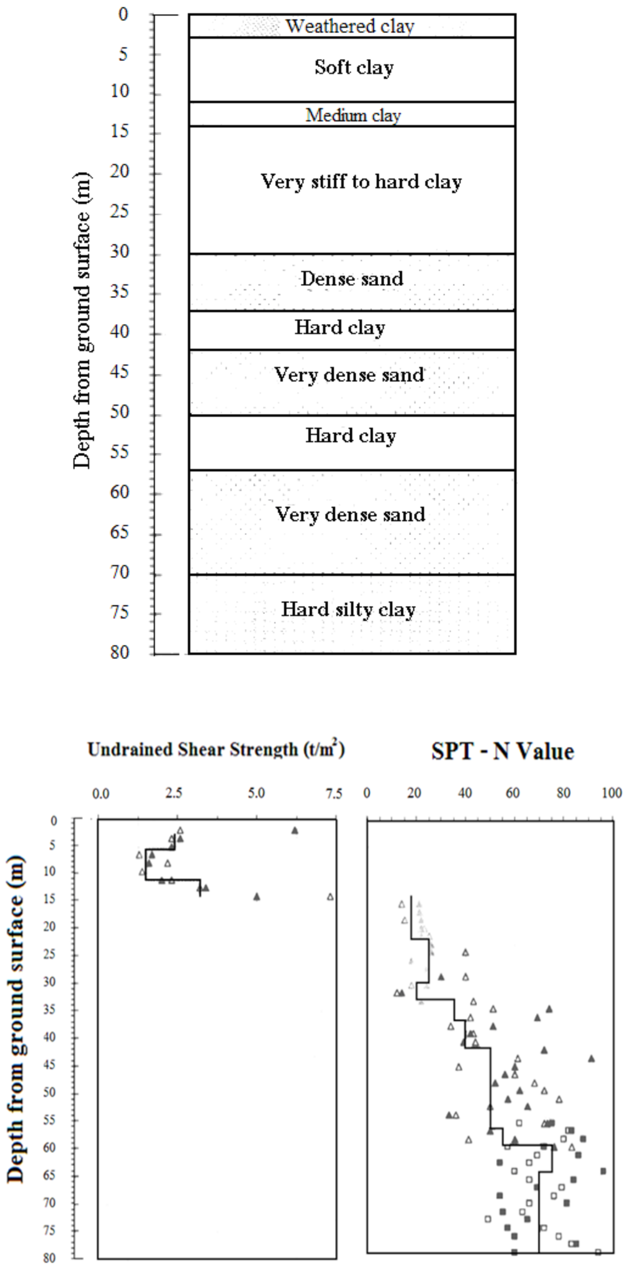


Figure 3 Description of typical Bangkok subsoil soil profile [14]

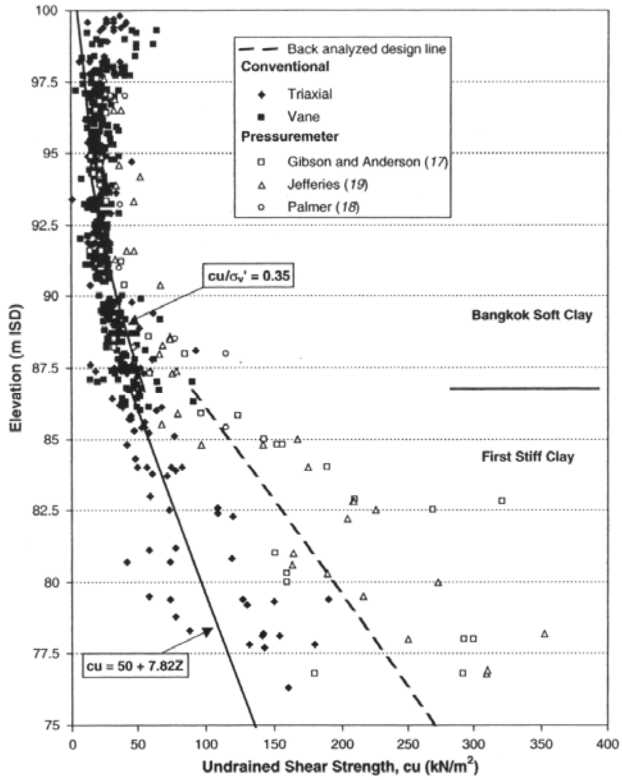


Figure 4 Shear Strength of Bangkok soft clay and first stiff clay [9]

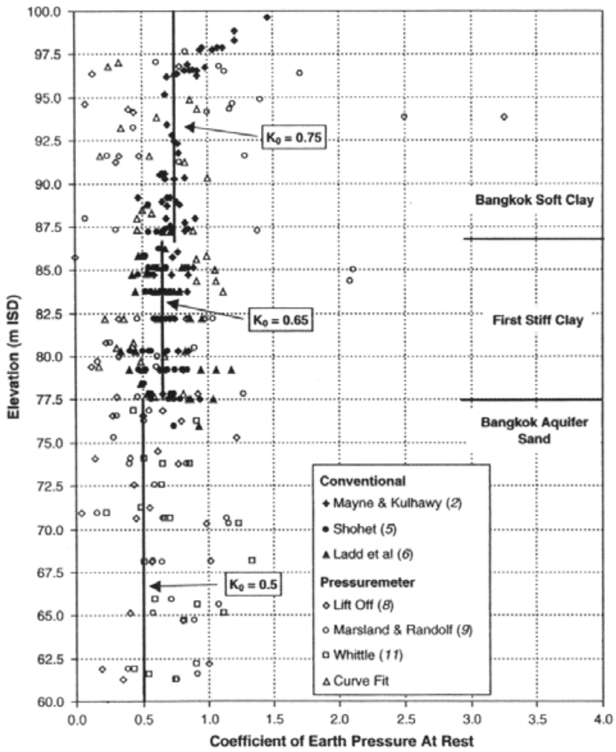


Figure 5 Coefficient of earth pressure for Bangkok clay, first stiff clay, and Bangkok aquifer sand [9]

The numerical models of drained triaxial tests corresponding to different stress paths are formulated to calibrate the soft clay parameters of each model. The clay samples referred in this analysis are undisturbed soft Bangkok clay taken from a site within the Asian Institute of Technology campus. Samples from a depth of 3 to 4 m are selected using the existing triaxial test results [12-13]. The stress paths followed in this calibration are compression loading for pile problem, compression unloading for deep excavation and extension unloading for tunneling. Using the K_o value from Figure 5, the results of triaxial tests for stress path of 72, 135 and 252 degree are chosen. Figures 6 to 8 show examples of the calibration results in terms of stress-strain and volumetric strain-shear strain, relationships corresponding to the experimental data from Uchaipichat [12] and Navaneethan [13]. The calibrated as well as correlated parameters for the three models of five relevant layers are summarized in Table 1 and 2.

Based on a study by Teparaksa [15], it was found that the elastic modulus of Bangkok clay can be calculated with investigated parameter, S_u from field vane shear test. Teparaksa suggested that the ratio of elastic modulus to undrained shear strength of soft Bangkok clay and stiff clay were E_u / S_u = 500 and 2000 for E (MC) and E_{50}^{ref} (HS and HSsmall), respectively. Additionally, the E_{oed}^{ref} is assumed as E_{50}^{ref} [4]. The stress dependent Young's modulus E_{ur}^{ref} for unloading/reloading has been calculated from the secant modulus E_{50}^{ref} by assuming a ratio of $E_{ur}^{ref} / E_{50}^{ref}$ equal to 3 [4]. Since the empirical formula from the above was proposed for total analysis, the Poisson's ratio of 0.5 is used to convert total parameters to effective parameters. The low strain stiffness of Bangkok clay is determined from previously investigated data correlation. For soft clay, by using the laboratory test and in-situ field vane shear test [10], it was found that the maximum shear modulus, G_0 is around $300S_u$ to $500S_u$, where S_u is undrained shear strength. Based on Bender element test [11], the maximum shear modulus ranges from $400S_u$ to $570S_u$. In this study, the value of $570S_u$ is used.

In stiff clay layers, it is difficult to collect an undisturbed sample for laboratory test. Thus, it is more convenient to relate the low strain shear modulus of clay to the shear wave velocity by using the empirical formula [16] as

$$G_0 = \rho V_s^2 \tag{6}$$

where ρ is density and V_s is the shear wave velocity. The shear wave velocity can be calculated by using the empirical equation correlated to SPT number (N) [16] as

$$V_s = 80.2 N^{0.292} \tag{7}$$

By using the above formulas, the calculated G_0 values of stiff soils are found to be in the range of $1500S_u$ to $3000S_u$ as suggested by Seed and Idriss [17].

The strain level where the shear modulus reaches 70% of initial value ($\gamma_{0.7}$) is calculated by using Hardin-Drnevich [18] relationship as

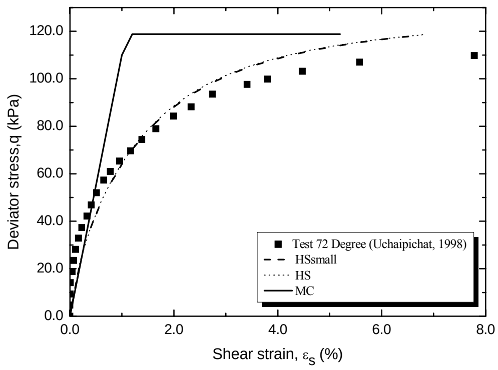
$$\gamma_{0.7} = \frac{1}{9G_0}[2c'(1+\cos(2\varphi')) - \sigma_1'(1+K_0)\sin(2\varphi')] \tag{8}$$

Table 1 MC soil model parameters

Soil layer	Wea. Crust	Soft Clay	Med. Clay	Stiff Clay	Sand
γ_{sat}[kN/m^3]	17	16	18	18	20
ν' [-]	0.32	0.33	0.33	0.33	0.3
φ' [°]	22	22	22	22	36
E' [kPa]	$570S_u$	$570S_u$	$1000S_u$	$2000S_u$	2600N

Table 2 HS and HSsmall soil model parameters

Soil layer	Soft Clay	Med. Clay	Stiff Clay
E_{50}^{ref} [kPa]	500 S_u	1000 S_u	2000 S_u
E_{oed}^{ref} [kPa]	E_{50}^{ref}	E_{50}^{ref}	E_{50}^{ref}
E_{ur}^{ref} [kPa]	3 E_{50}^{ref}	3 E_{50}^{ref}	3 E_{50}^{ref}
G_0^{ref} [kPa]	570 S_u	$\rho(80.2N^{0.292})^2$	$\rho(80.2N^{0.292})^2$ *
$\gamma_{0.7}$ [-]	Eq(8) [18]	Eq(8) [18]	Eq(8) [18]
m [-]	1[4]	1[4]	0.5[4]
p_{ref} [kPa]	100	65	95



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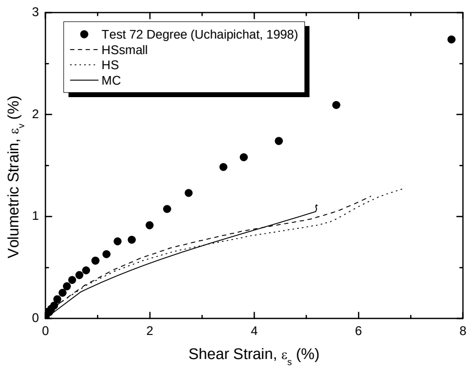
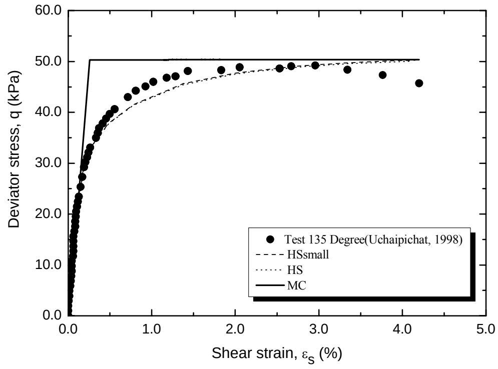


(b)

Figure 6 Deviator stress-shear strain (q-ε_s) (Figure a) and Volumetric strain–shear strain (ε_v-ε_s) (Figure b) for compression loading-stress path of 72 degree (comparison between model predictions and drained test data)



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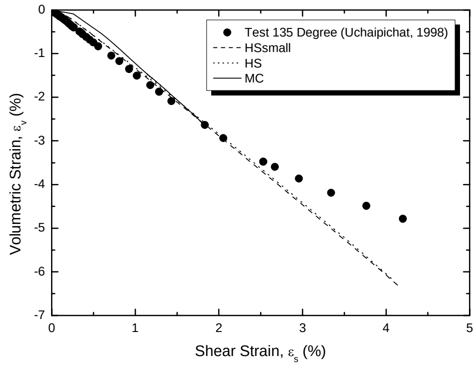

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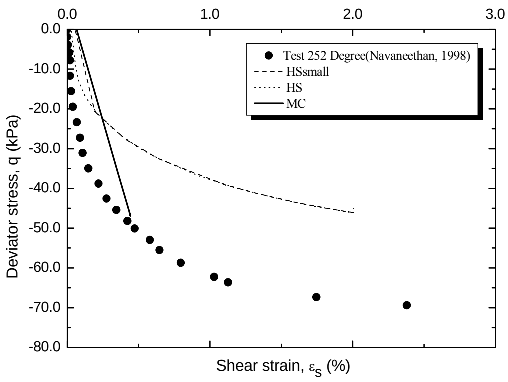
Figure 7 Deviator stress-shear strain (q-ε_s) (Figure a) and Volumetric strain-shear strain (ε_v-ε_s) (Figure b) for compression unloading-stress path of 135 degree (comparison between model predictions and drained test data)



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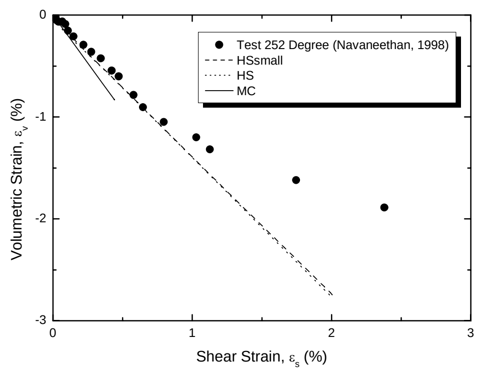

(b)

Figure 8 Deviator stress-shear strain (q-ε_s) (Figure a) and Volumetric strain-shear strain (ε_v-ε_s) (Figure b) for extension unloading-stress path of 252 degree (comparison between model predictions and drained test data)

4. Finite element models and analysis conditions

In this section, the finite element Program, PLAXIS 2D, is used to analyze three types of problem. In the analyses, soil layers are divided and the average value parameters are assumed for each layer. The ground water table is assumed to be 1.5 m below the ground surface. The sand layer is modeled with Mohr-Coulomb model only. The extension of the finite element mesh is wide enough for each case to ensure the accuracy of the analysis. The model is fixed in horizontal direction on both left and right sides. The bottom boundary is fixed in vertical direction. The well-documented case histories of each type of problem are collected. The measurement data of each case are compared with simulated results to evaluate the impact of soil model chosen for surrounding soils in geotechnical analyses.

4.1 Pile problem

Axis-symmetric models of six cases listed in Table 3, are created to simulate the pile load test. The concrete pile is treated as linear elastic material using solid elements with E of 2 x 10^7 kPa and ν of 0.3. After the initial condition of the ground is set up, the installation of pile is simulated. The load increment is gradually applied at the pile head following the actual values and the settlement at pile head is recorded. In the analysis, the value of 0.9 for interface of soil-structure is used. The results in terms of pile load-settlement curve from the simulation and measurement data are compared. Special attention is paid to the maximum pile settlement at the working and maximum loads.

Table 3 location of bored piles data of Bangkok soft clay

No	Project name	Process of pile	Diameter of pile (m)	Depth of pile (m)
1	8^{th} Tower Ladpraow rd.	Dry process	0.6	24
2	Krungdhon Hospital	Wet process	0.80	44.99
3	6^{th} Tower Rama6 rd.	Dry process	0.8	27
4	Bangplee over fly bridge	Wet process	0.8	47.2
5	Popular Tower	Wet process	0.8	45.7
6	Becthai	Wet process	0.8	50.31

4.2 Deep excavation

Plane strain models with suitable boundary conditions and FE mesh are constructed for 4 cases including Green Tower, Thammasat University, Orifleme Building and BMAH project. All projects were excavated using diaphragm wall. In the analyses, the actual excavation depth, wall thickness and excavation sequence are employed. Only one half excavation width is modeled. The diaphragm wall and basement slab are modeled with plate elements. The joint connection between basement slabs and diaphragm wall are modeled as semi-rigid connection by rotational stiffness. At the level of final excavation depth, the existing bore piles are considered. The bore piles are modeled by elastic model. The elastic modulus of bore pile is transformed to plane strain analysis using Lee et al. [19] approach. The measurement data of few construction stages for each case with the total number of 12 sets are collected. The selection of inclinometer location is done with consideration of plane strain condition. Only the measured points that are far away from edge of construction are selected.

4.3 Tunneling

The constructions of tunnel in MRTA project are chosen as well-document case studies. The observation points are CS-9A, CS-8E and CS-8C in section C Rama IX - Phetchaburi zone located at the depths of 17.1, 19 and 20.5 m, respectively [20]. The outer diameter of tunnel is 6.3 m and the tunnel lining is 0.3 m thick. Plane strain models are constructed to analyse tunnel excavation by shield tunneling method. The excavation is modeled by deactivation of soil elements situed in excavation zone after the initial conditions have been applied. Immediately after the soil elements are removed with application of volume loss, the tunnel lining elements which are treated as solid elements having liner elastic properties with E of 3.1 x 10^7 kPa and ν of 0.2, are activated. In the analyses, the volume loss was varied to get the appropriate distance of surface settlement and then the values of surface settlement at tunnel center (maximum), 5.5 and 11 m from tunnel center are discussed with measurement data.

5. Numerical results and comparison with measured data

5.1 Pile load test

From the results of pile load test, comparisons between the analysis results of the FEM and the observed values for 6 bored pile cases from 6 locations of Bangkok are employed for evaluation purposes. As an example, the boundary condition and mesh of FEM analysis case Krungdhon Hospital project is shown in Figure 9. Figure 10 shows comparisons between the analysis results from FEM and the observed values. It shows that the tendencies of the analytical results of HSsmall models are satisfactory in terms of the magnitude and the shape of the wall compared with other models. The simulated settlement from MC model is highly over-estimated whereas slightly over-estimated settlement is obtained from HS model. Figure 11 shows the maximum pile settlement at the working and maximum loads. Pile settlement from analyses compared with the observation results of all case studies. The result from FEM by using HSsmall model gives highest accuracy for the entire range of the observed settlement. The HS model can predict the pile settlement fairly well but slightly over-predict the values. The results from MC model seem to over-predict the settlement for all observed data. The difference between the observed and predicted loads seems to increase with magnitude of the settlement.

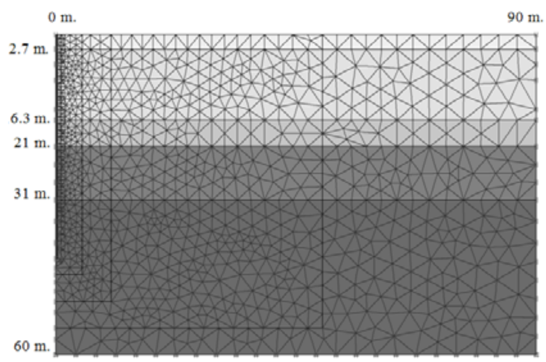

Figure 9 Finite element mesh of Krungdhon Hospital project

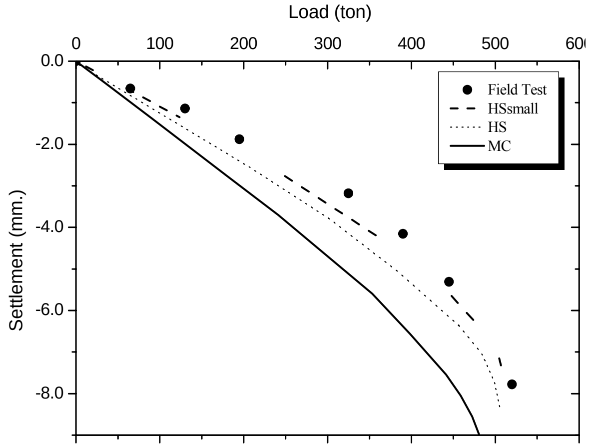

Figure 10 Pile settlement curves from analysis using different soil models compared to measurement data of Krungdhon Hospital project

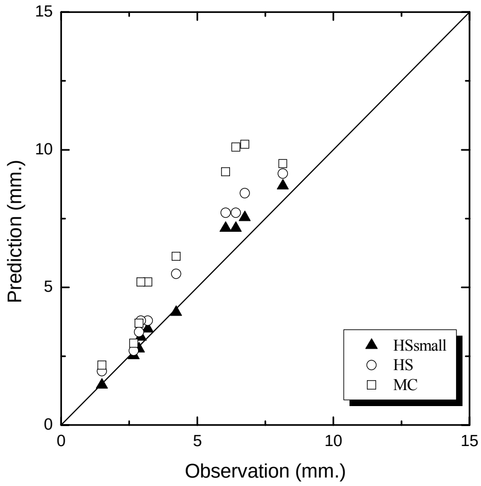

Figure 11 Comparison between predicted maximum pile settlement by different soil models and observation of all case studies.

5.2 Deep excavation

For deep excavation work, the measurement is commonly expressed by inclinometer readings which are the horizontal movements of retaining wall during and after excavation. As an example, the boundary condition and mesh of FEM analysis case BMAH project is shown in Figure 12. Figure 13 compares between the analysis results of the FEM and observed values. It shows that the tendencies of the analytical results of HS and HSsmall models are satisfactory in terms of the magnitude and the shape of the wall while those of MC model are too high.

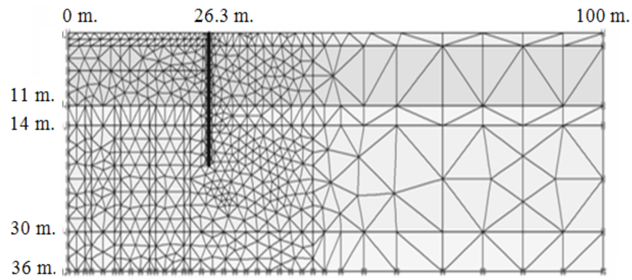

Figure 12 Finite element mesh of BMAH case study

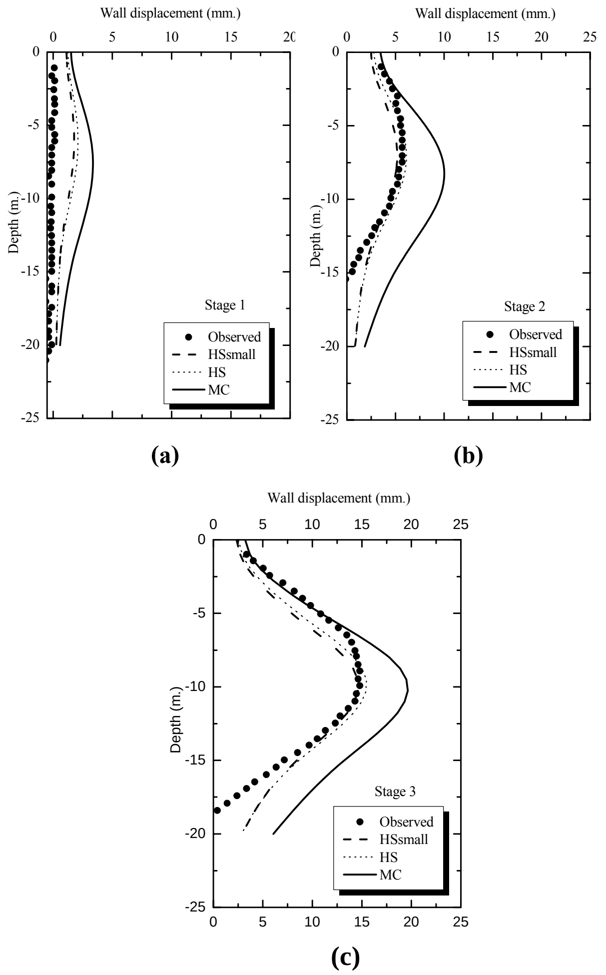

Figure 13 Horizontal wall displacements from analyses with different soil model compared to observed data; (a) stage 1 (-2.0 m.), (b) stage (-5.7 m.), (c) stage 3 (-11.0 m.)

Figure 14 shows the maximum lateral wall movement from analyses compared with the observation results of all cases.

The predicted results by HSsmall give high accuracy for the entire range of observed displacement. Both HS and MC model could closely predict the wall movement when the wall movement is greater than 30 mm. It can be obviously seen that for soil models which do not take the small-strain stiffness into account, the lateral wall movement is over-predicted.

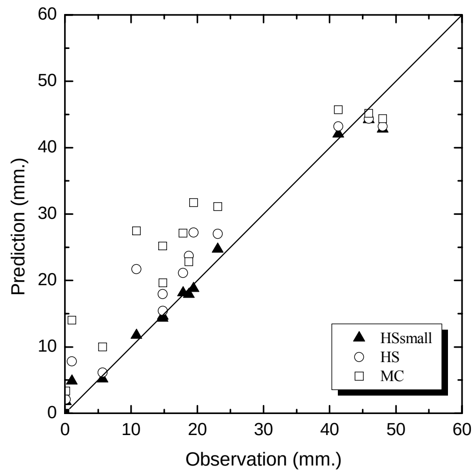

Figure 14 Comparison between predicted maximum horizontal displacement by different soil models and observation of all case studies.

5.3 Tunneling

For tunneling work, the ground surface settlements were measured by settlement marker. As an example, the boundary condition and mesh of FEM analysis case MRTA point CS-8E is shown in Figure 15. The results of ground surface settlement are shown in Figure 16 for comparisons of the FE analysis results and the observed values for case of MRTA point CS-8E in section C Rama IX - Phetchaburi. It shows that the analytical results from HSsmall models give satisfactory tendencies in terms of the magnitude and the shape of the wall compared to other models.

The results of surface settlement at tunnel center (maximum), 5.5 and 11 m from tunnel center are discussed with measurement data at point CS-8E, CS-8C and CS-9A as shown in Figure 17. From FE analyses, HSsmall and HS model give high accurate prediction for the entire range of observed settlement. The results from MC model seem to be under-predict for all observed data.

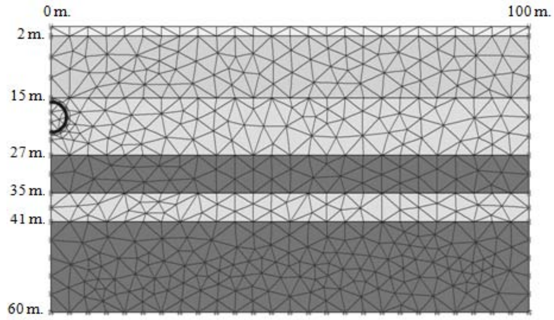

Figure 15 Finite element mesh of MRTA point CS-8E case study

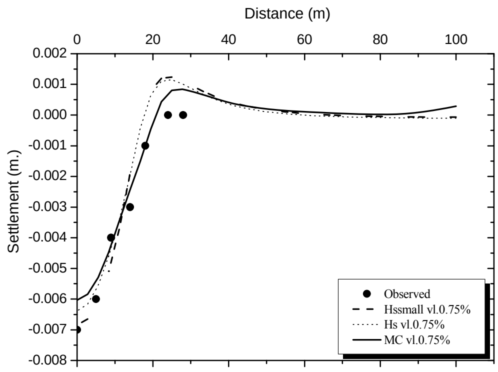

Figure 16 Ground surface settlement in CS-8E point (prediction by different soil models and observed data)

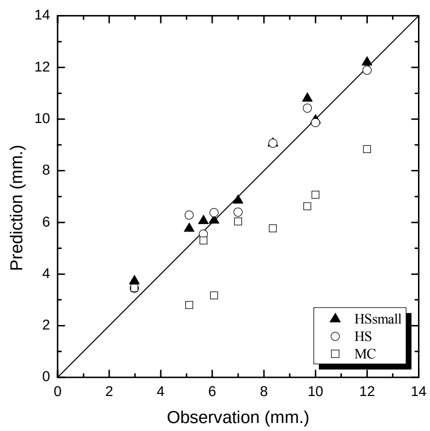

Figure17 Comparison between maximum predicted Ground surface settlement by different soil models and observation of all case studies.

6. Conclusion

The analyses of three kinds of geotechnical works carried out in this paper shows the impact of soil model on the simulations. Using more sophisticated soil models considerably improves the prediction of movements. The HSsmall model which includes non-linearity prefailure and high stiffness under very small strain gives high accuracy of movement prediction for all three types of work covering the entire range of observed data. The HS model gives acceptable movement prediction for analysis of pile and tunnel whereas the movements in excavation work are over-predicted. MC model over-predicts the movement for analysis of pile and excavation work whereas the movements in tunnel work are under-predicted, particularly, at range of small movement.

Acknowledgement

The authors would like to thank the following; Seafco Public Co. Ltd, STS instruments Co. Ltd, Mr. Kittisak Daungpan from Department of Highways for their support on observed data. The permission to use V.I.Plaxis Service Program by PLAXIS ASIA (PLAXIS BV) is gratefully acknowledged.

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