Identification of Bangkok Subsoil Parameters for Finite Element Analysis of Excavation and Tunnelling

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ABSTRACT: Finite element method has been widely used in geotechnical practice. In this method, modelling of soil behaviour and its parameters determination is an important procedure. Over the past decades, enormous efforts have been made to further develop elasto-plastic constitutive models. Nevertheless, the use of these constitutive models in engineering practice is still very limited. The required soil parameters of a particular subsoil condition is a challenging task for geotechnical engineers. Presently, the use of laboratory and in-situ tests, that helps determining the soil parameters, has been highly developed. This paper summarises the results of laboratory, field, and numerical studies related to Bangkok Clay behaviour. The stiffness and strength parameters, including small-strain parameters, of Bangkok Clay were main focuses. Anisotropy and non-linearity of Bangkok Clay were also introduced in this paper. The case studies of deep excavation and tunnelling of the Bangkok MRT were simulated using finite element modelling. The optimal parameters of Bangkok subsoils for deep excavation and tunneling works, based on Mohr-Coulomb and Hardening Soil Models, were finally summarised.

KEYWORDS: Finite element analysis, Constitutive model, Soil parameters, Laboratory tests, In-situ tests, Bangkok subsoil

1. INTRODUCTION

Bangkok subsoil is one of the most well-known sedimentary soils and has been extensively studied in the past by many researchers (e.g. Balasubramaniam & Chaudhry, 1978; Balasubramaniam et al., 1978, 1992; and Balasubramaniam & Hwang, 1980). It consists of a thick soft clay layer on the top deposit called "Bangkok Clay". This may lead to construction problems such as structural damages during the construction and throughout the life of structures. Recently, the stiffness and strength parameters of Bangkok Clay have been extensively investigated. Surarak et al. (2012) presented analysed stiffness and strength parameters for hardening soil model of soft and stiff Bangkok Clays. Likitlersuang et al. (2013a) presented geotechnical parameters from pressuremeter tests of Bangkok subway project. Likitlersuang et al. (2013b) described the small-strain stiffness and the stiffness degradation curve of Bangkok Clay. Ratananikom et al. (2013) and Yimsiri et al. (2013) studied anisotropic stress-strain and strength characteristics of Bangkok Clay. Ratananikom et a. (2015) presented relationships among undrained shear strength of Bangkok Clay from various laboratory techniques. Morover, the applications of soil parameters for finite element analyses were investigated with the case studies of deep excavations of the Bangkok subway station (Likitlersuang et al., 2013c) and tunnelling of Bangkok subway (Likitlersuang et al., 2014).

Finite element analysis of geotechnical problems become widely used in practice due to the development of more user-friendly softwares. This paper aims to summarise a set of optimal parameters for Bangkok subsoils to be used in finite element analysis. The hardening soil model and Mohr-Coulomb models are recommended for soft to medium stiff Bangkok Clay layers and the sand layer, respectively. The soil parameters were calibrated against the laboratory and in-situ testing results. Case studies of deep excavation and tunnelling of the Bangkok subway project were used to validate the finite element models. Some advanced parameters, such as anisotropy, non-linearity, and small-strain stiffness parameters, of Bangkok Clay for advanced soil models were also discussed.

2. BANGKOK SUBSOILS

2.1 Geological conditions

Bangkok is located on the delta flood plain of the Chao Phraya River, which traverses the lower central plain of Thailand (Fig. 1). The Quaternary deposits of the lower central plain represent a complex sequence of alluvial, fluvial, and deltaic sediments. The Quaternary stratigraphy consists of many aquifers, which are separated from each other by thick layers of clay or sandy clay. The depth of the bedrock is still undetermined, but its level in the Bangkok area is known to vary between 400 m to 1,800 m depth. Deep well pumping from the aquifers over the last 50 years or so has caused a substantial piezometric drawdown in the upper soft and highly compressible clay layer (Fig. 2).

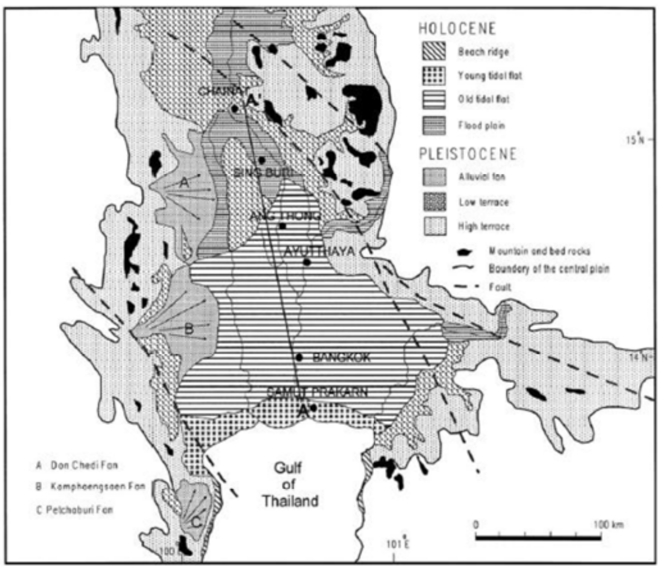


Figure 1 Geological map of Quaternary deposits in the lower central plain of Thailand (Sinsakul, 2000)

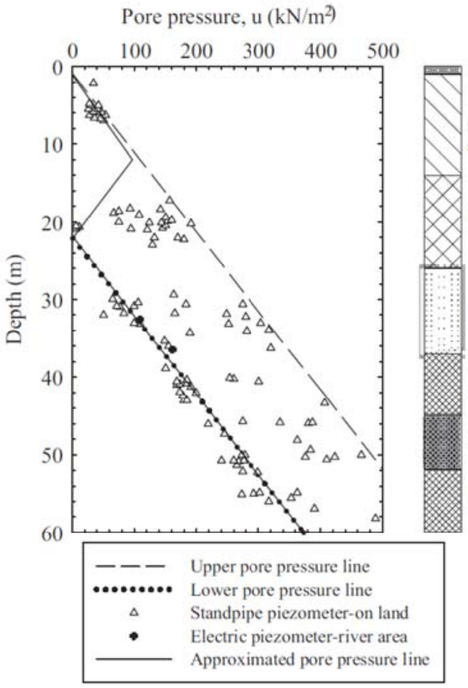


Figure 2 Pore pressure in Bangkok subsoils (Likitlersuang et al., 2013c)

2.2 Typical soil profile

Based on extensive field and laboratory studies carried out in the past by numerous researchers, the following descriptions have been proposed for a typical profile of Bangkok subsoils (Fig. 3).

- Weathered crust and backfill (MG: Made Ground) – The uppermost layer is the fill material (very loose to medium dense silty sand) and weathered crust (medium to stiff silty clay), which is light to yellowish gray in colour. The average thickness is about 1 to 3 m, with the SPT N-value ranging from 2 to 21. The water content is 10 to 35%. The groundwater table is found within this layer. The made ground layer is usually ignored in engineering practice.
- Very soft to medium stiff Bangkok Clay (BSC: Bangkok Soft Clay and MC: Medium Stiff Clay) – The very soft to soft Bangkok Clay layer located at depths of 3 to 12 m with medium gray to dark gray in colour. The undrained shear strength is ranging from 10 to 30 kN/m^2 and the natural water content is around 60 to 105%.
- Stiff to very stiff clay (1stSC: First Stiff Clay) – A layer of dark gray to brownish gray is found below the soft clay layer between depths of 15 to 35 m. The SPT-N values vary from 5 to 35, with undrained shear strength of 26 to 160 kN/m^2. The natural water content is around 15 to 60%.
- Medium dense to dense sand (CS: Clayey Sand) – First medium dense to dense clayey and silty sand, with a yellowish to grayish brown in colour, is present below the stiff clay layer, down to a depth of 30 to 40 m. This sand is fine to medium grained, with fine contents from 17 to 29%. The water content varies from 12 to 25%, with the SPT N values being greater than 25.
- Very stiff to hard clay (2ndSC: Second Stiff Clay and HC: Hard Clay) – The very stiff to hard clay layer is found below the medium to dense sandy layer, with a thickness of 10 to 12 m, and its colour varies from light gray to grayish brown. The SPT N value is greater than 30, with water content ranging from 15 to 22%. This hard clay may be absent at some locations.
- Very Dense Sand or Second Dense Sand (DS: Dense Sand) – The very dense layer is found below the very stiff to hard clay layer until the end of the boreholes, at about 50 to 60 m depth. The sand is silty and poorly graded with silt, being yellowish brown to brownish gray in colour. The SPT N values, in general, exceed 50.

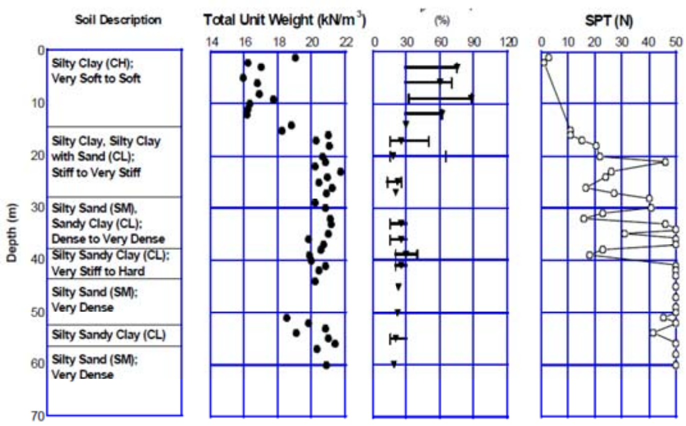


Figure 3 Typical soil profile and engineering properties of Bangkok subsoils (Likitlersuang et al., 2013a)

3. CONSTITUTIVE MODELS AND PARAMETERS

Finite element analysis of geotechnical problems has been becoming popularly used in research and practice due to the development of more user-friendly software. The Plaxis program is an example of such softwares. Constitutive models used in Plaxis are ranging from linear elastic to advanced plasticity models (e.g. Schweiger, 2009; and Surarak, 2010). However, only the Mohr-Coulomb and associated models are limited to be used among practicing engineers. This is because their parameters have obvious physical meanings and they are easier to be determined from typical site investigation reports. In this study, the hardening soil model throughout the Plaxis program was emphasised.

3.1 Undrained shear strength parameters

The undrained shear strength (s_u) is an important parameter for describing the consistency of cohesive soils during undrained loading. However, s_u is not a fundamental soil property because it is affected by mode of shearing, boundary conditions, rate of loading, confining stress level, initial stress state, and other variables (e.g. Wroth, 1984). In measuring s_u, various laboratory tests are used in practice; therefore, correlations of s_u measured from various laboratory tests can conveniently help the comparison of results among them.

3.2 Hardening Soil Model (HSM) parameters

The Hardening Soil Model (HSM) developed under the framework of the theory of plasticity was introducd in the Plaxis program as an extension of the Mohr-Coulomb model. Then an additional cap yield surface was added to the model to allow for the pre-consolidation pressure to be taken into account. The total strains are calculated using a stress-dependent stiffness, in which the stiffness is different for the loading and unloading/reloading parts. The strain hardening is assumed to be isotropic, depending on plastic shear and volumetric strain. A non-associated flow rule is adopted for the frictional hardening, while an associated flow rule is assumed for the cap hardening. A total of ten input parameters are required for the HSM, as tabulated in Table 1. Schanz et al. (1999) explained in detail the formulation and the verification of the HSM.

3.3 Small-strain stiffness parameters

The numerical analysis considering the effect of small strain could yield more realistic ground movements when compared to the field observations, especially in tunnellings and deep excavations (e.g. Addenbrooke et al., 1997; and Kung et al., 2009). In Plaxis, the Hardening Soil Model with Small Strain Stiffness (HSS) is available to model the soil behaviour at small strain. Two parameters, namely small-strain shear modulus (G_{max}) and reference shear strain ($\gamma_{0.7}$), are extra requests from the original HSM. The small-strain shear modulus (G_{max}) represents the (truly) elastic shear modulus of soil at small strain. The reference threshold shear strain ($\gamma_{0.7}$) is defined as the shear strain at G/G_{max} of 0.7, in which soils at the strain level below the threshold shear strain behave linear elastic (Vucetic, 1994; and Hsu & Vucetic, 2004).

3.4 Anisotropy

Most natural soils are deposited through a process of sedimentation followed by one-dimensional consolidation under accumulative overburden pressure for a long period of time. It is conceivable that under this condition, soils would be horizontally layered in both microscopic and macroscopic scales. This lead to anisotropy, called transversely isotropic or cross-anisotropic, in which soils have equal stiffness in all horizontal directions, but have a different stiffness in vertical direction. Graham & Houlsby (1983) proposed a simplified cross-anisotropic elastic model that required only three independent parameters. The anisotropic parameter (α) is defined as the ratio between horizontal and vertical moduli ($\alpha^2 = E_h/E_v$).

Table 1 Hardening Soil Model Parameters

Parameter	Description	Parameter evaluation
ϕ'	Internal friction angle	Slope angle of failure line based on Mohr-Coulomb
c'	Cohesion	Cohesion-intercept of failure line based on Mohr-Coulomb
R_f	Failure ratio	$(\sigma_1-\sigma_3)_f / (\sigma_1-\sigma_3)_{ult}$
ψ	Dilatancy angle	Ratio of $d\varepsilon_v^p$ and $d\varepsilon_s^p$
E_{50}^{ref}	Reference secant stiffness from drained triaxial test	Secant modulus at 50% peak strength at reference pressure, p^{ref}
E_{oed}^{ref}	Reference tangent stiffness for oedometer primary loading	Oedometer modulus at reference pressure, p^{ref}
E_{ur}^{ref}	Reference unloading/reloading stiffness	Unloading/reloading modulus at reference pressure, p^{ref}
m	Exponential power	Slope of trend-line in $\log(\sigma_3/p^{ref})$-$\log(E_{50})$ curve
ν_{ur}	Unloading/reloading Poisson's ratio	0.2 (default setting)
K_0^{nc}	Coefficient of earth pressure at rest (NC state)	1-sin ϕ' (default setting)

Notes: p^{ref} is a reference pressure (100 kN/m^2)
$(\sigma_1-\sigma_3)_f$ is a deviatoric stress at failure based on Mohr-Coulomb
$(\sigma_1-\sigma_3)_{ult}$ is an asymptotic value of shear strength

4. PARAMETERS FOR BANGKOK SUBSOILS

4.1 Undrained shear strength

Ratananikom et al. (2015) presented that values of s_u of Bangkok Clay are varied from several method of observations. The interrelationships among various s_u with $(s_u)_{CIUC}$ (isotropically consolidated undrained triaxial compression) are presented in Eq. (1). They also suggested that $(s_u)_{CIUC}$ can be used for embankment stability analysis for Bangkok Clay without any correction.

$$\left.\begin{array}{ll} (s_u)_{UC} = 0.843(s_u)_{CIUC} & R^2 = 0.763 \\ (s_u)_{UU} = 0.673(s_u)_{CIUC} & R^2 = 0.414 \\ (s_u)_{CIUE} = 0.873(s_u)_{CIUC} & R^2 = 0.430 \\ (s_u)_{CK_oUC} = 1.451(s_u)_{CIUC} & R^2 = 0.870 \\ (s_u)_{CK_oUE} = 1.287(s_u)_{CIUC} & R^2 = 0.571 \\ (s_u)_{DS} = 1.364(s_u)_{CIUC} & R^2 = 0.711 \\ (s_u)_{DSS} = 1.099(s_u)_{CIUC} & R^2 = 0.511 \end{array}\right\} \quad (1)$$

where UC – unconfined compression; UU – unconsolidated undrained triaxial; CIUE – isotropically consolidated undrained triaxial extension; CK_0UC and CK_0UE – K_0 consolidated undrained triaxial compression and extension; DS – direct shear; DSS – direct simple shear

4.2 HSM parameters

Surarak et al. (2012) studied the strength and stiffness parameters of Bangkok subsoils for HSM based on the triaxial and oedometer tests reported from previous literatures. The Mohr-Coulomb strength parameters were calibrated against the isotropically consolidated drained and undrained triaxial compression (CIDC and CIUC) and extension (CIDE and CIUE) tests. The nonlinear moduli for HSM and consolidation parameters were determined from oedometer tests. Finally, the sets of HSM parameters (Table 2) were numerically calibrated against undrained and drained triaxial results using a Plaxis finite element software.

Table 2 Parameters for Hardening Soil Model

Soil Type	γ_b (kN/m^3)	c' (kPa)	ϕ' (°)	ψ (°)	E_{50}^{ref} (MPa)	E_{oed}^{ref} (MPa)	E_{ur}^{ref} (MPa)
MG	18	1	25	0	45.6	45.6	136.8
BSC	16.5	1	23	0	0.8	0.85	8.0
MC	17.5	10	25	0	1.65	1.65	5.4
1stSC	19.5	25	26	0	8.5	9.0	30.0
FS	19	1	27	0	38.0	38.0	115.0
2ndSC	20	25	26	0	8.5	9.0	30.0
HC	20	40	24	0	30.0	30.0	120.0

Table 2 (continue)

Soil type	ν_{ur}	m	K_o^{nc}	R_f	G_{max} (MPa)	$\gamma_{0.7}$ (%)
MG	0.2	1	0.58	0.9	-	-
BSC	0.2	1	0.7	0.9	8.5	0.068
MC	0.2	1	0.6	0.9	12	0.09
1stSC	0.2	1	0.5	0.9	30	0.002
FS	0.2	0.5	0.55	0.9	-	-
2ndSC	0.2	1	0.5	0.9	50	0.002
HC	0.2	1	0.5	0.9	-	-

4.3 Small-strain stiffness parameters

Figure 4 shows a typical soil profile, moisture content, and Atterberg limits, as well as the G_{max} parameter from various sites in Bangkok. The trends of the G_{max} increase with depth in both the soft and stiff clays. However, the magnitudes of G_{max} are significantly higher in the stiff clay layers. Likitlersuang & Kyaw (2010) proposed empirical correlations between undrained shear strength and G_{max} from down-hole and muti-channel analysis of surface wave (MASW) in Eqs. (2a) and (2b), respectively. Likitlersuang et al. (2013b) proposed correlation between G_{max} and limit and net limit pressure (p_L and p_L*) from the lateral load tester (LLT) pressuremeter tests as shown in Eq. (3). In addition, Likitlersuang et al. (2013b) presented the small-strain stiffness parameter of the soft and stiff Bangkok Clays based on the cyclic triaxial results. The small-strain shear modulus (G_{max}) and the reference threshold shear strain ($\gamma_{0.7}$) of Bangkok subsoils for the HSM are also summarised in Table 2.

$$V_s(m/s) = 187\left(\frac{s_u}{p_a}\right)^{0.372} \quad (2a)$$

$$V_s(m/s) = 228\left(\frac{s_u}{p_a}\right)^{0.510} \quad (2b)$$

$$G_{max} = 50p_L \quad (3a)$$
$$G_{max} = 80\,p_L^* \quad (3b)$$

where $p_L^* = p_L - \sigma_{ho}$ is the net limit pressure and σ_{ho} = the total horizontal stress

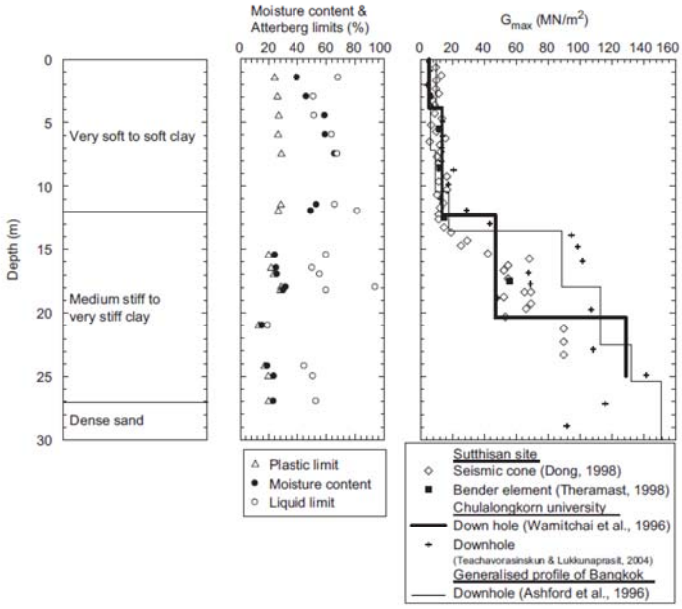


Figure 4 G_{max} profile in Bangkok area (Likitlersuang et al., 2013b)

4.4 Anisotropic parameters

Ratananikom et al. (2015) reported the ratio of s_u from triaxial extension to compression, i.e. $(s_u)_{CIUE}/(s_u)_{CIUC}$ = 0.87 and $(s_u)_{CKoUE}/(s_u)_{CKoUC}$ = 0.89 which shows that Bangkok Clay has more isotropic behaviour (in terms of undrained shear strength) than other marine clays.

Ratananikom et al. (2013) presented an investigation programme of cross-anisotropic elastic parameters of Bangkok Clay. A comprehensive triaxial tests were performed on vertically- and horizontally-cut undisturbed Bangkok Clay specimens. The experiments employed local-strain measuring systems and bender element; therefore, the soil stiffness at small strains can be measured. The anisotropic stiffness parameters were analysed following the three-parameter cross-anisotropic elastic model proposed by Graham & Houlsby (1983). The elastic moduli were assumed as power functions of isotropic confining stress. A complete set of cross-anisotropic elastic parameters of Bangkok Clay was proposed as shown in Eq. (4). The result concluded that undisturbed Bangkok Clay has a degree of stiffness anisotropy ($\alpha = \sqrt{E_h/E_v} = G_{vv}/G_{vh}$) of 1.14 or E_h/E_v = 1.3, where E_v and E_h are drained Young's modulus in the vertical and horizontal directions, respectively.

$$\left.\begin{aligned} \frac{E_v}{p_a} &= 231\left(\frac{p'_0}{p_a}\right)^{0.62} \\ \frac{E_h}{p_a} &= 300\left(\frac{p'_0}{p_a}\right)^{0.62} \\ \frac{G_{vh}}{p_a} &= 122\left(\frac{p'_0}{p_a}\right)^{0.62} \\ \frac{G_{hh}}{p_a} &= 139\left(\frac{p'_0}{p_a}\right)^{0.62} \\ \nu_{hh} &= 0.077 \\ \nu_{vh} &= 0.068 \\ \nu_{hv} &= 0.088 \end{aligned}\right\} \quad (4)$$

Yimsiri et al. (2013) investigated the undrained strength-deformation characteristics of Bangkok Clay under general stress condition by using torsional shear hollow cylinder (TSHC) apparatus. They presented the dependency of strength and stiffness on intermediate principal stress parameter (b) and major principal stress direction (α). The results also showed that Bangkok Clay is stiffer in vertical direction under undrained condition with the ratio of $E_{u,v}/E_{u,h}$ = 1.5-1.7, where $E_{u,v}$ and $E_{u,h}$ are undrained Young's modulus in vertical and horizontal directions, respectively. They also investigated the failure surface and the plastic strain increment vectors on the principal stress and strain axes and reported that undisturbed Bangkok Clay has the Drucker-Prager failure criterion with an associated flow rule.

4.5 Underground pore water pressure conditions

The pore water pressure in the Bangkok area is not hydrostatic, due to the effect of deep well pumping as shown in Fig. 2. Likitlersuang et al. (2013c) presented that a simulation of realistic drawdown pore water pressure is crucial for a finite element analysis of the Bangkok subsoil.

5. CASE STUDIES

The sets of Bangkok subsoils parameters were applied through the Plaxis finite element software. Two case studies of deep excavation and tunnelling of the Bangkok subway blue line project were chosen to validate the finite element model and related soil parameters (Likitlersuang et al., 2013a; 2014). A 2D plane-strain finite element analysis approach was adopted in this study. The following is a brief explanation of the two case studies.

5.1 Deep excavation

The Sukhumvit station located underneath Asok Road was selected in this study. The geometry of the Sukhumvit station is illustrated in Fig. 5. As the ratio of the length (L) to width (B) of Sukhumvit Station box was high (L/B = 8.7), the 3D effect along the long sides of the station was small, thus the 2D plane strain approach was considered appropriate. Only the right half of the station box was modelled because the station configuration was symmetric. The simplified geological condition similar to Fig. 3 was used in this study. The constitutive models and their parameters summarised in Table 2 was utilised to evaluate their performances for deep excavation problem. All soil layers were modelled using the 15-noded triangular elements. For the structural components (i.e. diaphragm wall, platform and base slab, column, and pile), the non-volume plate elements were used. The stiffness of the concrete was reduced by 20% to take into account the possibility of cracking. The finite element models and their mesh generation are shown in Fig. 6. The model has an average element size of 2.53 m and a total element number of 649.

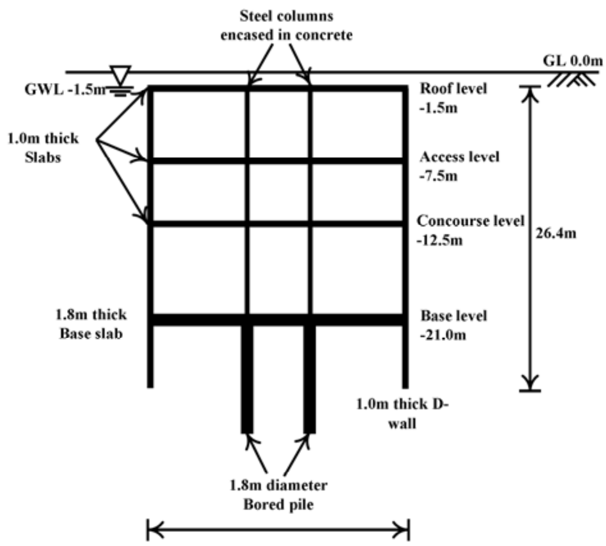


Figure 5 Geometry of Sukhumvit Station (Likitlersuang et al., 2013c)

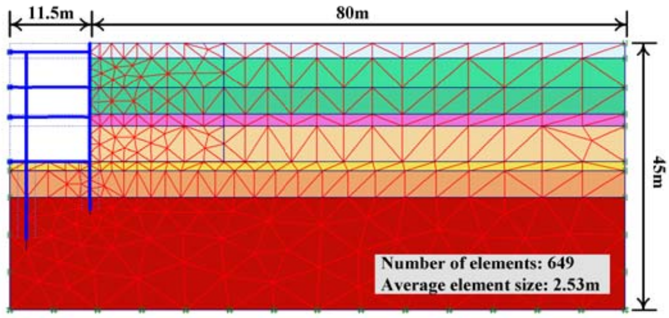
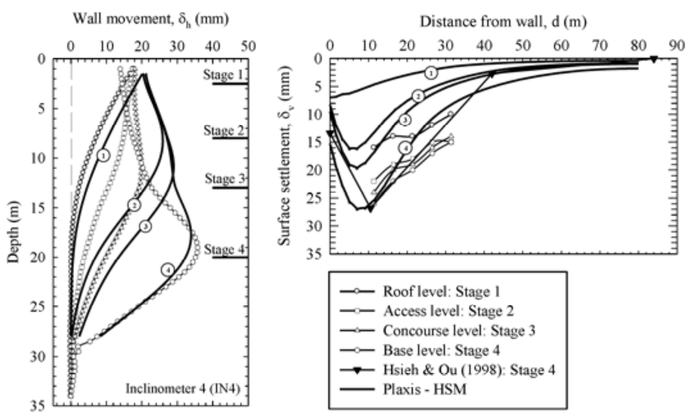
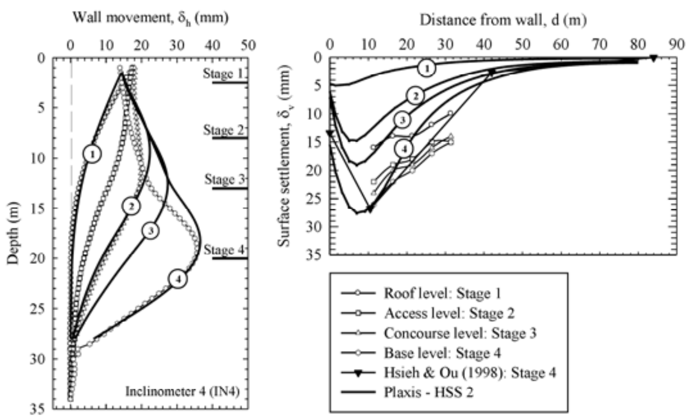


Figure 6 Finite element modelling of Sukhumvit Station (Likitlersuang et al., 2013c)

Figs. 7(a) and 7(b) compare the measured lateral wall movement and the ground surface settlement with those predicted by the HSM and the HSM with small strain parameters, repectively. The predicted lateral wall movements at all excavation stages within the BSC layer (2.5 to 12 m depth) were slightly higher than the field measurements. This overestimation extends further into the deeper layers for the excavation stages 1 to 3. The maximum lateral movement in the last excavation stage (located in 1st and 2nd SC layers) agrees well with the measured values. In the case of the ground surface settlement comparison, the HSM with small-strain parameters predicted better settlement envelopes compared to those predicted by the HSM. However, the settlements within the influence zone were still slightly larger than the predictions using Hsieh & Ou (1998) method. For more detail, it can bee seen in Likitlersuang et al. (2013c).



(a) HSM



(b) HSM with small strain stiffness parameters

Figure 7 Measured and predicted lateral wall movement and surface settlement (Likitlersuang et al., 2013c)

5.2 Tunnelling

The simplified finite element methods were used to model the shield tunnelling of the Bangkok subway blue line project (Likitlersaung et al., 2014). The study focused on the use of three simplified methods, which are contraction ratio method, stress reduction method, and modified grout pressure method, to back-analyse ground settlement due to tunneling works. All the back-analysis results are compared with the field monitoring data in order to assess the validity of the chosen methods.

A section from Khlong Toei to Queen Sirikit twin tunnels, with a side-by-side pattern, was selected. The tunnels of this section are located partially in the stiff clay and partially in the hard clay layers. A high face pressure of 175 and 170 kN/m^2 was applied to the SB and NB tunnels, along with a penetration rate of 25 mm/min, a high grout pressure of 250 to 400 kN/m^2, and a high percentage of grout filling of 150%. The maximum surface settlement after both shields had passed was about 25 mm. The soil profile illustrated in Fig. 3 was employed in the analysis. The soil constitutive model adopted was the HSM. The input parameters of the HSM finite element analysis study as presented in Table 2 were retained. The tunnel lining was modelled using the plate element with EA = 8000 MN/m and EI = 56 MNm^2/m. Fig. 8 depicts the finite element mesh generation of the section. The number of elements is 1670 with an average element size of 1.55 m.

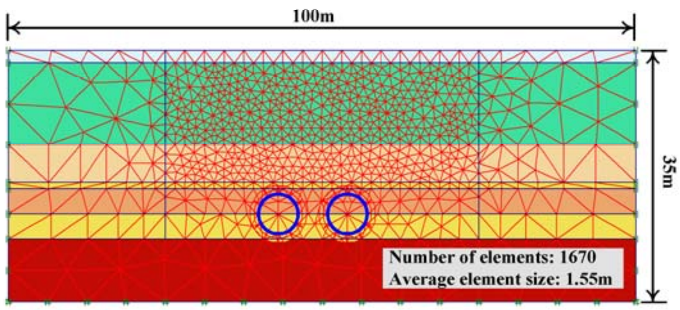


Figure 8 Finite element modelling of Bangkok subway twin tunnel (Likitlersuang et al., 2014)

A modified grout pressure method was applied to the finite element analysis. In this method, the face and grout pressures were modelled by an applied pressure which increased linearly with depth. The unit weight of the slurry and grout material were assumed to be 12 and 15 kN/m^3, respectively. In the first attempt, the average face and grout pressures, as measured from the earth pressure chamber and the shield tail, were used as the face and grout pressures. These face and grout pressures were averaged from fluctuating data. As a consequence, it gave an over-prediction of the ground settlement, when compared to the field measurements. Therefore, it was decided that a series of finite element back-analyses were performed in the second attempt and the result is shown in Fig. 9. In general, the predictions of the surface settlement agree well with the field measurements. The ratios of the calculated and measured face pressure were 1.06 and 1.03 for SB and NB, respectively. More details of other cases can be seen in Likitlersuang et al. (2014).

6. CONCLUSIONS

This study aims to identify the sets of optimal parameters for Bangkok subsoils, which can be used for the finite element analysis. The following conclusions can be made:

1) To obtain the optimal soil parameters required in advanced constitutive model, the comprehensive experimental results, such as triaxial and consolidation tests, are essentially required. The shear strength parameters shall be determined from the undrained and drained triaxial tests. The non-linear stiffness parameters shall be calibrated against the oedometer results.

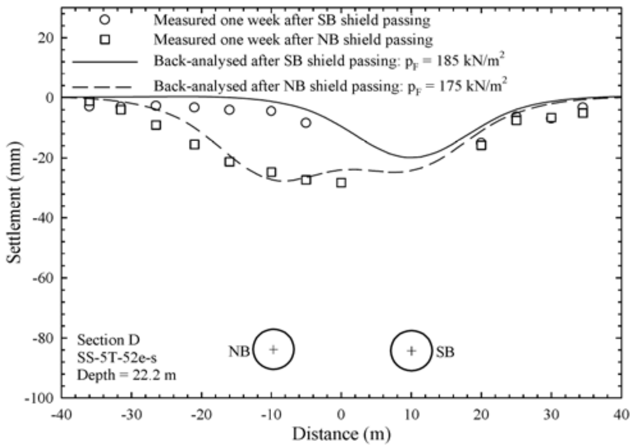


Figure 9 Finite element analysis results of the Bangkok subway twin tunnels (Likitlersuang et al., 2014)

2) The small strain shear modulus (G_{max}) and reference shear strain ($\gamma_{0.7}$), are additionally required for the finite element analysis with considering the small-strain effect. These parameters shall be determined from special tests that can capture the stiffness at small to medium strain ranges. In this study, the small-strain parameters for Bangkok clays were determined from the cyclic triaxial tests.
3) A special triaxial apparatus equipped with local-strain measuring devices and bender elements was successfully used to determine the cross-anisotorpic elastic parameters of Bangkok clay. In addition, the failure surface and the plastic strain increment vectors on the principal stress and strain axes were found to be the Drucker-Prager failure criterion with an associated flow rule.
4) The finite element analysis, employed the HSM with soil parameters interpreted from the laboratory and field tests, provides good agreement with the lateral wall movement and surface settlement field observations of the Bangkok subway.
5) Simulations of realistic initial stress and pore water pressure conditions as well as construction sequences are crucial for a finite element analysis of excavation and tunnelling.

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