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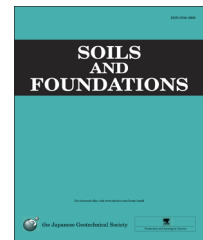
Small strain stiffness and stiffness degradation curve of Bangkok Clays

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Abstract

The small strain stiffness and the stiffness degradation curve of soils are required in advanced numerical analyses of geotechnical engineering problems. The shear modulus at small strain (G_{max}) and the reference shear strain parameter ($\gamma_{0.7}$) are, for instance, two of the input parameters in a finite element analysis with the hardening soil model with small strain stiffness. The stiffness and strength parameters for the hardening soil model of soft and stiff Bangkok Clays has recently been published (Surarak et al., 2012). This paper is a continuation on the previous study on the stiffness of Bangkok Clay, and focuses on the small strain characteristics. The data are from the Bangkok MRT Blue line project as well as comprehensive studies at Chulalongkorn University and the Asian Institute of Technology. Based on these laboratory and field testing data, the parameters G_{max} and $\gamma_{0.7}$ can be determined using well-known empirical correlations and the concept of threshold shear strain. Finally, a comparison between the measured data and predictions is made.
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Keywords: Shear modulus; Small strain; Stiffness degradation curve; Bangkok Clay

1. Introduction

The stiffness characteristics of soils have been known to be significant in geotechnical analyses, and especially so with finite element method since 1979 (Simpson et al., 1979). The small strain stiffness and strain dependent stiffness are two required input parameters. These two parameters are usually used to govern the small strain behaviour of soil such as in the hardening soil model with small strain stiffness (HSS). However, limited studies on the small strain stiffness of Bangkok Clays have been carried out. In this study, the small strain stiffness behaviour of Bangkok Clays was determined based on laboratory and field tests carried out at Chulalongkorn University by the first two authors (Teachavorasinskun et al., 2002a, 2002b; Teachavorasinskun and Lukkunaprasit, 2004; Likitlersuang and Kyaw, 2010; Ratananikom et al., 2012) and at the Asian Institute of Technology, where the last author was earlier affiliated (Ashford et al., 1996; Dong, 1998; Theramast, 1998). A recent study on the stiffness and strength parameters at large strain of soft and stiff Bangkok Clays based on experimental studies from the Asian Institute of Technology was presented earlier (Surarak et al., 2012). Moreover, an important

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construction project involving the construction of a mass rapid transit underground railway known as the MRT Blue line yielded useful geotechnical information about Bangkok sub-soils. In particular, the pressuremeter investigation data from the Bangkok MRT Blue line project were employed in this study. Finally, the analysis of small strain stiffness, with the concept of threshold shear strain taken into account (Vucetic, 1994), was performed to determine the two required imputed parameters for HSS.

1.1. Definition and roles of small strain stiffness

The initial stiffness modulus is an important soil parameter related to the predictions of the ground movements and field data interpretations. In soil dynamics and earthquake engineering, the small strain shear modulus (G_{max}) and the damping ratio (D) are important parameters in soil characterisation. A stiffness degradation curve is normally used to explain the shear stiffness for a wide range of shear strain. Atkinson and Sallfors (1991) categorised the strain levels into three groups: the very small strain level, where the stiffness modulus is constant in the elastic range; the small strain level, where the stiffness modulus varies non-linearly with the strain; and the large strain level, where the soil is close to failure and the soil stiffness is relatively small. This explanation was illustrated using the normalised stiffness degradation curve by comparing with the ground response from geotechnical construction and the measurement accuracy from laboratory investigation (Atkinson and Sallfors, 1991; Mair, 1993) as shown in Fig. 1.

The significance of small strain non-linear behaviour of soils in deep excavations was examined by Kung et al. (2009). Comparisons of the diaphragm wall deflections and the ground surface settlements in Taipei clays were observed from the field measurements and were predicted from finite element analyses based on the small strain non-linear type of soil model. The results showed that the analysis with the small strain model yielded a realistic settlement profile when compared to the field observations. Similarly, in the case of the ground movements induced by tunnelling, the finite element study of London underground tunnelling (Addenbrooke et al., 1997) revealed that non-linear small strain stiffness are necessary to achieve ground settlement predictions. The above discussions demonstrate the significance of non-linear small strain stiffness in enhancing the predictive capabilities of finite element based models. The current in-depth study on the small strain parameters of Bangkok Clays is presented in this paper.

1.2. Laboratory and in-situ studies of Bangkok Clays

The studies on the small strain stiffness characteristics of Bangkok Clays were mainly based on laboratory and in-situ tests at Chulalongkorn University and the Asian Institute of Technology. The studies at Chulalongkorn University were focused on the laboratory testing. Teachavorasinskun et al. (2002a) conducted a series of cyclic triaxial tests on Bangkok Soft Clay using precise external measurements at strain level of 0.01%. Teachavorasinskun et al. (2002b) also conducted a series of cyclic triaxial tests on the Bangkok Soft Clay with applied load frequency of 1 and 0.1 Hz. Recently, Ratananikom et al. (2012) investigated an anisotropic elastic parameter of Bangkok Clay using the triaxial apparatus equipped with local strain measuring systems and bender element. In addition, the in-situ tests such as the down-hole seismic test (Teachavorasinskun and Lukkunaprasit, 2004) and the surface wave analysis technique (Likitlersuang and Kyaw, 2010) have been carried out to determine the shear wave velocity profiles of Bangkok subsoil.

On the other hand, the in-situ testing was considerably studied at the Asian Institute of Technology. The down-hole seismic tests in Bangkok subsoil were firstly carried out by Ashford et al. (1996) to define the small strain shear modulus (G_{max}). The seismic cone penetration tests (Dong, 1998) and the bender element tests (Theramast, 1998) were performed in parallel on the soil specimen collected from the same site in Bangkok area. Likitlersuang et al. (2013) determined the in-situ shear modulus from the pressuremeter testing results taken from the Bangkok MRT Blue Line project.

The aforementioned laboratory and in-situ testing results are employed to determine the small strain stiffness characteristics of Bangkok Clays in this study.

2. Determination of small strain stiffness

2.1. Laboratory and in-situ measurements of small strain stiffness

The laboratory and in-situ tests for measuring the pre-failure small strain stiffness of soils are briefly reviewed here. Laboratory testing plays a vital role in determining the stiffness of soils, but it is already noted that they can suffer from various disadvantages such as sample disturbance, sample preparation and apparatus sophistication (Clayton, 2011). Two different methods are usually made in the laboratory tests. The first method involves measuring the local strain in triaxial testing (Goto et al., 1991; Scholey, et al., 1995). Standard instrumentation, such as the linear
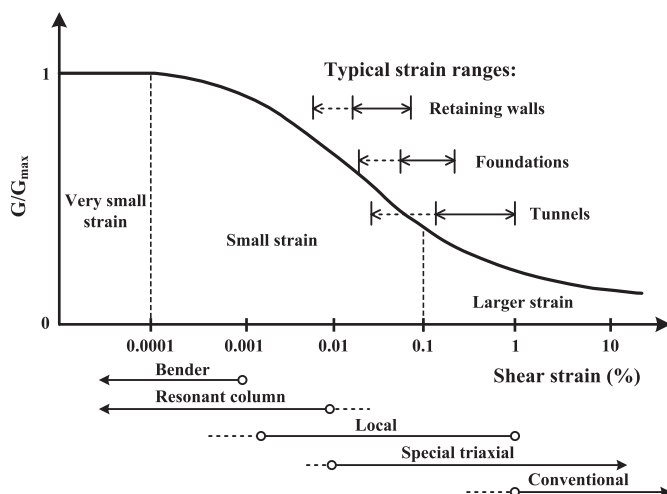

Fig. 1. Normalised stiffness degradation curve (modified after Atkinson and Sallfors (1991) and Mair (1993).

variable differential transformer (LVDT) (Jardine et al., 1984) and submersible proximity sensor (Hird and Yung, 1989), came into use in triaxial testing. This is because conventional triaxial apparatus could only measure soil stiffness at axial strain level of approximately 0.1%, which is already in large strain level. Most research laboratories now prefer high resolution (about 1 microstrain) and this can be achieved using LVDTs (Cuccovillo and Coop, 1997). For example, Santagata et al. (2005) reported the undrained stiffness modulus at the very small strain of 0.0001%.

The second method used in the laboratory is carried out under quasi-static loading, and includes the potential use of dynamic testing, such as resonant column, bender elements and cyclic triaxial testing. The resonant column testing process involves applying a series of cyclic forces to soil specimens at various frequencies. With this capability the shear modulus and damping ratio at a low strain level at 0.01% can be obtained. However, there are some shortcomings in the use of the resonant column test in geotechnical earthquake engineering (Towhata, 2008). For example, the number of loading cycles experienced by the soil specimen in the resonant column test is much higher than that from a real earthquake, and the high frequency of the shaking during the testing process dramatically reduces the permeability coefficient of the soil. In different circumstances, the bender elements employ the wave travelling properties of soil. The bender elements are a set of electro-mechanical transducers that produce a high frequency shake (mechanical energy) and create S-wave propagation from one end of the soil specimen to the other. This mechanical energy, received from the far end of the specimen, is then converted to electrical energy to calculate the S-wave velocity using travel time (t) and distance (L) (Viggiani and Atkinson, 1995). The interpretation of the bender element test result is to calculate first the shear S-wave velocity ($V_s = L/t$) and then determine the initial (elastic) shear modulus as

$$G_{max} = \rho V_s^2 \tag{1}$$

where ρ is the density of soil. One of the advantages of the use of bender elements over other testing devices is that it is possible to equip the bender element transducers with various types of standard laboratory equipment, i.e. the triaxial, direct shear and oedometer tests. Thus, the bender element test can be performed under various soil laboratory conditions, such as during isotropic or K_o consolidation and shearing.

On the other hand, the in-situ tests are only classified as a dynamic analysis such as the down-hole and cross-hole seismic test, and seismic cone test. The down-hole seismic test requires one borehole in which seismic receivers are placed at desired depths, and a set of wave generator (normally a hammer and steel planks) generate the S-wave propagation. The major shortcoming of the down-hole seismic test is that the S-wave propagation generally travels from the ground surface to the receivers at desired depths in the borehole. This S-wave is, therefore, travelling through multiple layers of soil and, thus, the resulting shear wave velocity is considered as an integration of the multi-layer soil property. By receiving

measurements from different depths, it is, therefore, possible for the in-situ small strain stiffness of each soil layer to be back-calculated. In contrast, the cross-hole test requires at least two boreholes, where an S-wave generator is placed in one hole and the receiver is mounted in the other. The seismic cone test is a hybrid test combining the original cone penetration test (CPT) with the down-hole seismic test. Moreover, several techniques from geophysics testing, such as a surface wave analysis, can also be applied to determine the shear wave velocity of subsoils.

2.2. Empirical prediction for small strain stiffness

Most of the published experimental data focusing on the small strain stiffness of soils were usually derived from dynamic laboratory tests on both natural and reconstituted samples. A review of literature shows that there are a number of factors affecting the estimation of soil stiffness at small strain level. These factors are not only related to the natural and soil inherent structures (e.g., gradation size and particle shape) but also to its geological history (e.g., stress state, stress history, aging, chemical processes).

A normalised empirical equation of small strain shear modulus was given by Rampello et al. (1997) as

$$\frac{G_{max}}{p_a} = Sf(e)\left(\frac{p'}{p_a}\right)^n \tag{2}$$

where G_{max} is the small strain shear modulus (typically at $\gamma \approx 0.0001\%$), $f(e)$ is an empirically defined function of the void ratio, p' is the mean effective stress, p_a is the reference stress (usually the atmospheric pressure, 100 kPa), S and n are dimensionless experimentally determined parameters. The small strain stiffness depends on the current stress state expressed by the mean effective stress, the current void ratio and the previous stress history experienced by the soil. The relationships in the form of Eq. (2) were presented by several researchers to fit the experimental data in different types of soil. Table 1 summaries the empirical relationship for small strain stiffness of clays.

An alternative way of correlating the parameter G_{max} in clays is to employ the undrained shear strength (s_u) or the blow count of the standard penetration test (N-value). Other researchers, for example, Ashford et al. (1996) and Likitlersuang and Kyaw (2010), have developed simple correlations of the shear wave velocity (V_s) and the undrained shear strength (s_u), based on reliable in-situ site investigations in Bangkok subsoils. Importantly, after the shear wave velocity and soil density (ρ) are estimated, the small strain stiffness (G_{max}) can be calculated using Eq. (1).

A generalised shear wave velocity profile was developed by Ashford et al. (1996) based on the down-hole seismic tests from 13 sites across the Bangkok area. The simple correlation between the shear wave velocity (V_s in m/s) and the undrained shear strength (s_u in kN/m^2) for the Bangkok Soft Clay is given as

$$V_s(m/s) = 23s_u^{0.475} \tag{3}$$

Table 1
Constants of empirical relationship $(G_{max}/p_a) = Sf(e)(p'/p_a)^n$ for small strain stiffness of clays.

Soil type	Test method[a]	S	f(e)	n	Void ratio range	References
Remoulded kaolin	RC	327	$\frac{(2.973-e)^2}{1+e}$	0.5	0.76–0.9	Hardin and Black (1968, 1969)
Reconstituted NC kaolin	RC	450	$\frac{(2.973-e)^2}{1+e}$	0.5	1.1–1.3	Marcuson and Wahls (1972)
Reconstituted NC bentonite	RC	45	$\frac{(4.4-e)^2}{1+e}$	0.5	1.6–2.5	Marcuson and Wahls (1972)
Several undisturbed silts and clays (NC range)	RC	74–288	$\frac{(2.973-e)^2}{1+e}$	0.46–0.61	0.4–1.1	Kim and Novak (1981)
Undisturbed NC clay	Cyclic TX	14	$\frac{(7.32-e)^2}{1+e}$	0.6	1.7–3.8	Kokusho et al. (1982)
Six undisturbed Italian clays	RC & BE	275–1174	$e^{-1.3}$ (average from e^{-x}: $x=1.11$–1.43)	0.40–0.58	0.6–1.8	Jamiolkowski et al. (1994)[b]
Several soft clays	SCPT	500	$e^{-1.5}$	0.5	0.5–5	Shibuya and Tanaka (1996)[c]
Several soft clays	SPCT	1070–3080 (average 2400)	$(1+e)^{-2.4}$	0.5	0.5–5	Shibuya et al. (1997)[c]

[a]*Remarks*:TX, triaxial test; RC, resonant column test; TS, torsional shear test; BE, bender element test; SCPT=seismic cone penetration test.
[b]Use both σ'_v and σ'_h rather p'.
[c]Use σ'_v instead p'.

A similar study was recently conducted on the V_s and s_u correlation for Bangkok subsoils (Likitlersuang and Kyaw, 2010). The shear wave velocity data were obtained from the down-hole seismic test and the multichannel analysis of surface wave (MASW) methods conducted at three different sites up to 30 m depth in Bangkok area. Two correlations were proposed, being based on the down-hole and MASW data, respectively:

$$V_s(m/s) = 187\left(\frac{s_u}{p_a}\right)^{0.372} \qquad (4)$$

$$V_s(m/s) = 228\left(\frac{s_u}{p_a}\right)^{0.510} \qquad (5)$$

It was found that the correlation in Eq. (3) suggested by Ashford et al. (1996) gives the average values between the Eqs. (4) and (5).

3. Stiffness degradation curve

3.1. Concept of threshold shear strain

The concept of threshold shear strains was introduced by Lo Presti (1991) and then was elucidated by Vucetic (1994). The threshold shear strains represent the boundaries of the fundamental cyclic behaviours of soils at very small, small and medium to large strain level. The soils at strain level below the linear threshold shear strain (γ_{tl}) behave linear elastic. Between the linear threshold shear strain (γ_{tl}) and volumetric threshold shear strain (γ_{tv}), the soils begin to exhibit non-linear behaviour, but remain largely recoverable, since the microstructure of the soils remains unchanged. Beyond the volumetric threshold shear strain (γ_{tv}), the soils become heavily non-linear and inelastic. In other words, the soil microstructure

changes irreversibly when the shear strain exceeds the volumetric threshold shear strain (γ_{tv}).

From various laboratory test results on clayey soils, Vucetic (1994) proposed a model of volumetric cyclic threshold shear strain which increases with the soil plasticity index. This model was later confirmed and refined by Hsu and Vucetic (2004). Vucetic (1994) also determined that the threshold shear strains have a negligible effect with the OCR value. Importantly, the normalised stiffness degradation curves tend to move up and to the right as soil plasticity index increase. This means that, regardless of the soil type and OCR value, the secant shear modulus decreases by the same proportion before the threshold shear strain is reached.

Moreover, when the threshold shear strains were plotted in the normalised stiffness degradation curves, the bands of threshold shear strains correspond with the values for G/G_{max} of 0.65–0.67. Based on this concept, Santos and Correia (2001) proposed a reference threshold shear strain ($\gamma_{0.7}$), defined as the shear strain at G/G_{max} of 0.7 within the normalised shear modulus degradation curves. This $\gamma_{0.7}$ was later utilised in the hardening soil model with small strain stiffness (HSS) in the PLAXIS program (Brinkgreve, 2002).

3.2. Parameter $\gamma_{0.7}$

The reference threshold shear strain ($\gamma_{0.7}$) is regarded herewith as a soil parameter to define the stiffness degradation curve. Two methods of calculation can be used to determine the parameter $\gamma_{0.7}$. The first method as suggested by Vucetic and Dobry (1991) and Vucetic (1994) assume that the parameter $\gamma_{0.7}$ is a linear function with plasticity index (I_p) as

$$\gamma_{0.7} = 0.0021I_p - 0.0055 \qquad (6)$$

This model can only fit with the full stiffness degradation curves of the soils that range from non-plastic material with $I_p = 0$ (sandy soil) to high plasticity soil with $I_p = 200$.

A slightly more rigorous method for the stiffness degradation curve was proposed by Ishibashi and Zhang (1993). This method takes the effects of both the mean effective confining stress (σ'_{m0}) and the soil plasticity index (I_p) on the stiffness degradation curves into account. The Ishibashi and Zhang (1993) stiffness degradation curves are given in mathematical form as

$$\frac{G}{G_{max}} = \mathbf{K}(\gamma, I_p)\sigma'^{\tilde{\mathbf{m}}(\gamma, I_p)}_{m0} \tag{7}$$

$$\mathbf{K}(\gamma, I_p) = 0.5\left[1 + \tanh\left\{\ln\left(\frac{0.000102 + \mathbf{n}(I_p)}{\gamma}\right)^{0.492}\right\}\right] \tag{8}$$

$$\mathbf{n}(I_p) = \begin{cases} 0 & \text{for } I_p = 0 & \text{(sandy soils)} \\ 3.37 \times 10^{-6}I_p^{1.404} & \text{for } 0 < I_p \leq 15 & \text{(low plastic soils)} \\ 7.0 \times 10^{-7}I_p^{1.976} & \text{for } 15 < I_p \leq 70 & \text{(medium plastic soils)} \\ 2.7 \times 10^{-5}I_p^{1.115} & \text{for } I_p > 70 & \text{(high plastic soils)} \end{cases} \tag{9}$$

$$\tilde{\mathbf{m}}(\gamma, I_p) = 0.272\left[1 - \tanh\left\{\ln\left(\frac{0.000556}{\gamma}\right)^{0.4}\right\}\right]e^{-0.0145I_p^{1.3}} \tag{10}$$

The mean in-situ effective confining stress (σ'_{m0}) defines as follows:

$$\sigma'_{m0} = \left(\frac{1 + 2K_0}{3}\right)\sigma'_{v0} \tag{11}$$

where σ'_{v0} is the effective vertical stress and K_0 the coefficient of earth pressure at rest.
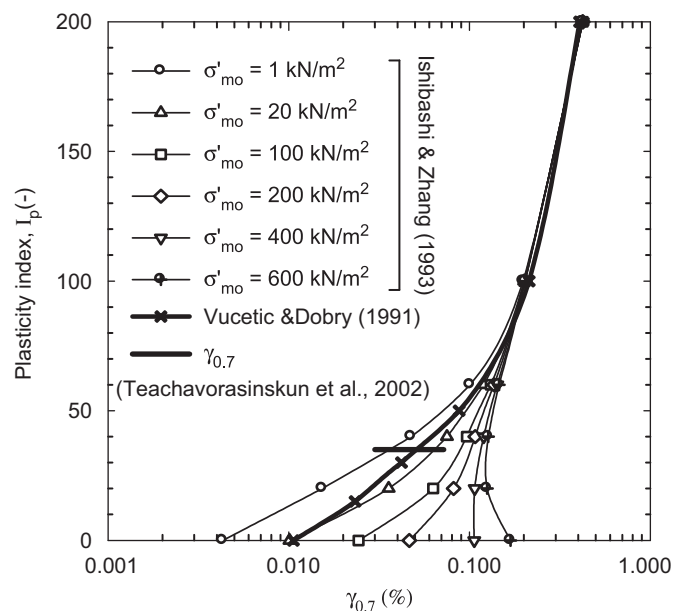
Fig. 2 shows the variations of $\gamma_{0.7}$ with the range of I_p from 0 to 200 and σ'_{m0} from 1 to 600 kN/m^2, as calculated from Vucetic and Dobry (1991) and Ishibashi and Zhang (1993) methods. Basically, $\gamma_{0.7}$ tends to increase with the increasing mean effective stress. After the plasticity index of soils goes beyond 100, the effect of the mean effective confining stress seems to become insignificant and negligible. It is interesting to see that the values of $\gamma_{0.7}$ from Vucetic and Dobry (1991) model are nearly identical to those values of Ishibashi and Zhang (1993) with σ'_{m0} of 20 kN/m^2.

4. Stiffness data of Bangkok Clays

The following section provides a brief overview of the studies related to the small strain parameters of Bangkok subsoils. The section also examines the empirical correlations of small strain stiffness to identify the most suitable for practical use. Additionally, both the parameters G_{max} and $\gamma_{0.7}$ of Bangkok Clays have been adopted in this study.

4.1. Bangkok subsoil conditions

The Bangkok metropolitan area is located on the low flat Chao Praya Delta in the Central Plain region of Thailand, as shown in Fig. 3. The terrestrial deposits in the city lie from 0 to about 4–5 m above the mean sea level, with the other soil layers being marine deposits, resulting from changes in the sea level during the Quaternary period. A multitude of construction activities, including deep excavations, high rise buildings, elevated expressways, a new airport, and even underground tunnels, have taken place or are taking place in this sedimentary marine deposit. The deposit consists of an extensive overlay of Bangkok soft marine clay, which is of low strength and high compressibility. The upper soft clay layer is underlain with several aquifers inter-bedded with clay and sand. Over several decades extensive ground water pumping from the aquifers has caused large piezometric drawdowns and alarming subsidence.

The Bangkok subsoil forms a part of the larger Chao Phraya Plain and consists of a broad basin filled with sedimentary soil deposits. These deposits form alternate layers of sand, gravel and clay. While the depth of the bedrock is still undetermined, its level in the Bangkok area is known to vary between 400 m and 1800 m depth. The Bangkok MRT Blue Line project was the first underground MRT construction project in Bangkok and its construction was completed in 2003. The project was constructed along highly congested roads in the heart of Bangkok city, which is 22 km in length and included 18 underground cut-and-cover subway stations as shown in Fig. 3. Field exploration and laboratory tests from the MRT Blue Line project show that the subsoils, down to a maximum drilling depth of approximately 60–65 m, can be roughly divided into (1) made ground at 0–1 m, (2) soft to medium stiff clays at 1–14 m, (3) stiff to very stiff clays at 14–26 m, (4) first dense sand at 26–37 m, (5) very stiff to hard clays at 37–45 m, (6) second dense sand at 45–52 m and then following by (7) very stiff to hard clays. The typical Bangkok subsoils and their basic properties are plotted in Fig. 4. This plot is similar to the field investigating results by Horpibulsuk et al. (2011). The soft and stiff Bangkok Clay layers from approximately

Fig. 2. Comparison of measured and computed $\gamma_{0.7}$ of Bangkok Soft Clay.

Fig. 3. Bangkok metropolitan map.

1–18 m below ground surface have been taken into consideration in this study.

4.2. Small strain stiffness of Bangkok Clays

The study of the G_{max} parameter in Bangkok subsoils here is mainly focused on the data from the Sutthisan site (Dong, 1998; Theramast, 1998), from the site at Chulalongkorn University (CU) (Warnitchai et al., 1996; Teachavorasinskun and Lukkunaprasit, 2004; Likitlersuang and Kyaw, 2010), from the site at Asian Institute of Technology (AIT) (Warnitchai et al., 1996; Likitlersuang and Kyaw, 2010); and from the site at Mahidol University (MU) (Teachavorasinskun

and Lukkunaprasit, 2004) as well as the data of six sites across Bangkok areas from Ashford et al. (1996). The locations of the study are in the Bangkok metropolitan area as shown in Fig. 3. It is noted that the Sutthisan site and the site at Chulalongkorn University are located close to the Sutthisan station and Sam Yan station of the Bangkok MRT Blue Line, respectively.

In terms of the soil profiles, the Bangkok subsoils are fairly uniform in thickness and parameter wise as presented in Fig. 4. In terms of the G_{max} parameter, its uniformity is shown in Fig. 5; this figure also presents a typical soil profile of the Bangkok, its basic moisture content, and Atterberg limits, as well as the G_{max} parameter from the Sutthisan site (Dong, 1998; Theramast, 1998), the CU site (Warnitchai et al., 1996),

and the generalised data for the six sites in Bangkok (Ashford et al., 1996). Two parallel studies, related to the small strain behaviour of soils, were conducted. Dong (1998) concentrated on the in-situ measurement aspects and Theramast (1998) carried out laboratory bender element tests. Importantly, the values of G_{max}, as shown in Fig. 5, were calculated from the measured shear wave velocity (V_s) and the soil density (i.e., $\rho_s = 1.6$ and 1.8 Mg/m³ for soft and stiff clays, respectively). Similarly, the G_{max} values were calculated in the same way at the CU site. The values of G_{max} from both sites are fairly similar, especially for the soft clay layer. The generalised data from Ashford et al. (1996) also falls into a narrow band. Indeed, the trends of the G_{max} values increase with depth in both the soft and stiff clays. However, the magnitudes of G_{max} are significantly higher in the stiff clay layers. The results can be compared to the small strain shear moduli of London Clay. It is well-known that the London Clay is the geologically old, often overconsolidated, and stiff clay, which has plasticity index (I_p) in the range of 30–50 (Gasparre et al., 2007b). The G_{max} values investigated from the bender element, static hollow cylinder and resonant column tests are in the range of 65–88 MPa at the depth of 0.8–7.9 m (Gasparre et al., 2007a). It can be seen from Fig. 5 that the G_{max} of Bangkok Soft Clays is around 5–7 times lower than the small strain shear moduli of London Clay but it is comparable in Bangkok stiff clay layer.

The empirical correlations for the small strain stiffness (G_{max}), as summarised in Table 1, can also be applied to the Bangkok subsoils. For the methods proposed in Table 1, the void ratio (e) and the in-situ stresses (σ'_v) are essential. The void ratio values of Bangkok Clays calculated from the natural moisture content (w_n) are in the ranges of 0.5–2, as depicted in Fig. 6. The approximated drawdown piezometric line as shown in Fig. 7 was used in the

in-situ stress calculation. Hardin and Black (1969) with constant S parameters of 327 is chosen in this study. However, for Shibuya and Tanaka (1996), and Shibuya et al. (1997), the constant S, in Eq. (2) and Table 1, is dependent on the soil type, structure and ageing effects. For this reason, in the current study, this parameter was adjusted to obtain suitable values for the Bangkok Soft and Stiff Clays. The results of the best fit parameter are illustrated in Fig. 6. Best fit constant parameters S as adopted are tabulated in Table 2.

An alternative way to obtain the G_{max} is to use the shear wave velocity (V_s) and undrained shear strength (s_u) correlation. Eq. (5) (Likitlersuang and Kyaw, 2010) was selected as it gave a close approximation of V_s compared to the measured values. The undrained shear strengths of Bangkok Soft and Stiff Clays were calculated from the s_u/σ'_{v0} ratios of 0.33 and 0.2 (Surarak et al., 2012). These two ratios are verified with the undrained shear strength from the in-situ vane shear in soft clay and the undrained triaxial tests in stiff clay as illustrated in Fig. 8. The G_{max} values of the soft and stiff clays, as calculated from the correlated shear wave velocity and the soil density, are plotted in Fig. 6. The predicted G_{max} gives the same trend as the measured values. These simple correlations, therefore, can be used as the first approximation when only the in-situ vertical effective stress is known.

Lastly, the G_{max} parameter on the Bangkok Clays also relates to the limit and net limit pressures (p_L and p_L^*) from the lateral load test (LLT) pressuremeter tests. The result of the LLT pressuremeter tests from the Bangkok MRT Blue Line project showed that p_L and p_L^* can be reasonably correlated with the undrained shear strength of Bangkok Clays. It can be seen in more detail in Likitlersuang et al. (2013). As illustrated in Fig. 9, the pressuremeters p_L and p_L^* of the Bangkok Soft and Stiff Clays, when plotted with depth, provide a similar trend when compared with the G_{max} values. Indeed, they are in excellent agreement with the
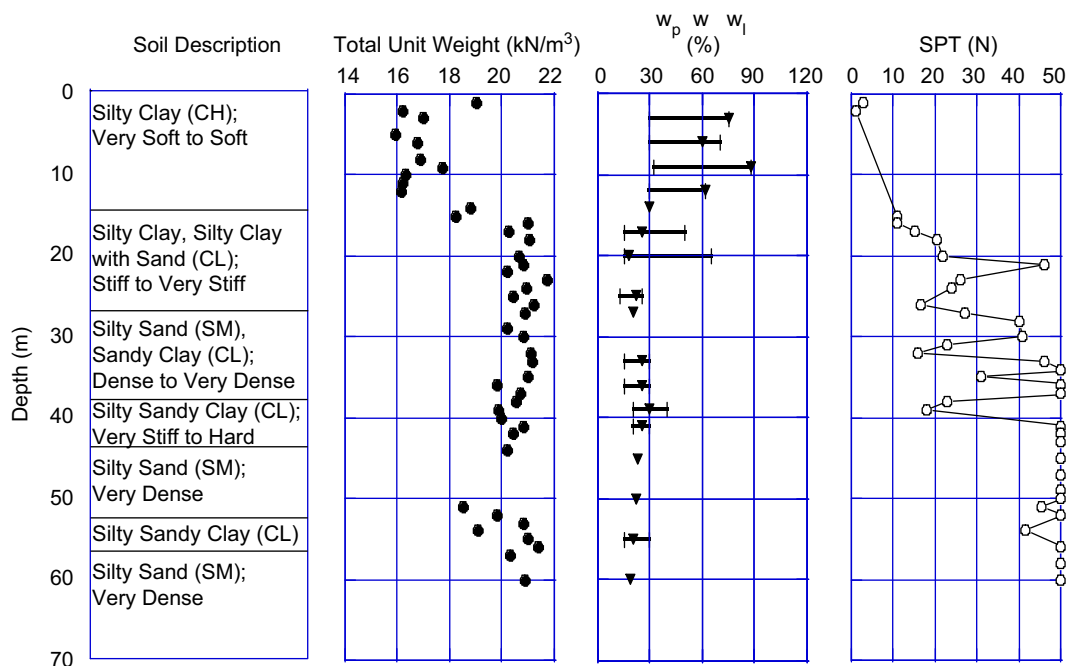

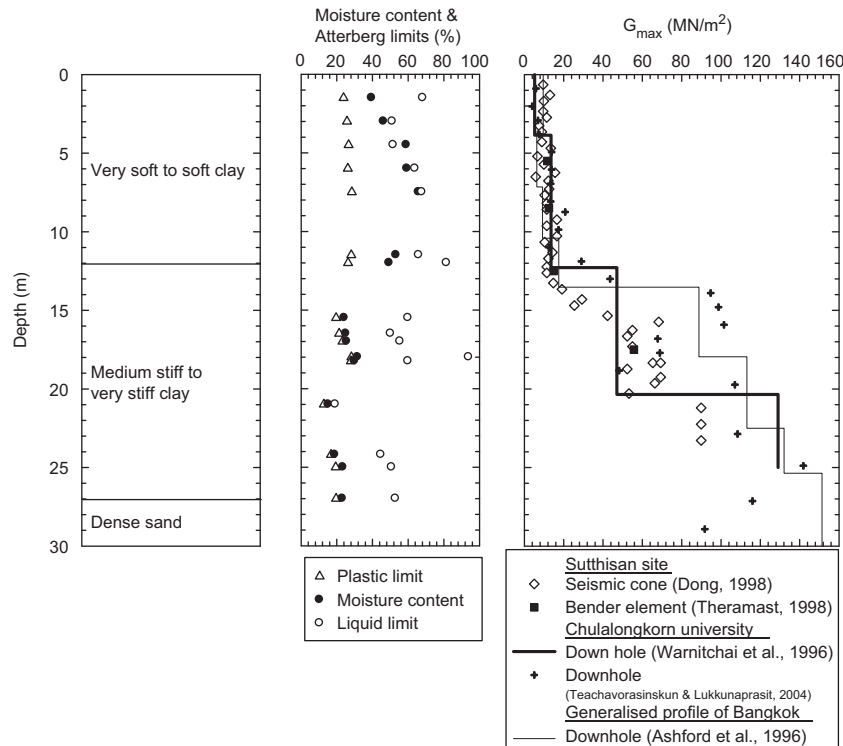Fig. 4. Typical profile and properties of Bangkok subsoils.

Fig. 5. Comparison of G_{max} from laboratory and in-situ tests in Bangkok area.

simple linear correlations, as follows:

$$G_{max} = 50p_L \tag{12}$$

$$G_{max} = 80p_L^* \tag{13}$$

where $p_L^* = p_L - \sigma_{h0}$ is the net limit pressure and σ_{h0} the total horizontal stress.

Further, Eqs. (12) and (13) can be treated as a simple rule of thumb, in practice, when the results from the pressuremeter tests are available. More details of the LLT pressuremeter tests of Bangkok Clays and their interpretation are given in Likitlersuang et al. (2013).

4.3. Stiffness degradation curve of Bangkok Clay

Unlike the small strain stiffness (G_{max}), the knowledge of $\gamma_{0.7}$ on Bangkok Clays is still limited. To obtain the measured values of $\gamma_{0.7}$, one needs to first know the values of the G_{max}, and then measure the shear modulus (G) at a small level of shear strain amplitude (approximately $10^{-4}\%$) to a large strain level. Next, the normalised stiffness degradation curve can be constructed and the $\gamma_{0.7}$ can be read from the curve. An example of such a normalised stiffness degradation curve was provided in the study by Teachavorasinskun et al. (2002a). They conducted a series of cyclic triaxial tests on Bangkok Soft Clay using a precise external measurement at strain level of 0.01%. The results from two sites were chosen i.e., Chulalongkorn University (CU) site and Mahidol University (MU) site. The CU site is located in the centre of Bangkok closed to the Sam Yan MRT station; on the other hand, the MU is located in the west of Bangkok (approximately 20 km from CU site) as shown in Fig. 3. The plasticity index (I_p) of soil

samples were 30 and 40 for the CU and MU sites, respectively. Fig. 10 shows the normalised G/G_{max} curves resulting from the cyclic triaxial tests at 50, 150 and 250 kN/m² for the confining pressure (σ'_c). Importantly, the G_{max} used in this normalised curve was taken from the down-hole seismic test for the CU site and from the Hardin and Black (1969) correlation for the MU site. According to Teachavorasinskun et al. (2002a), the stiffness degradation curves fell within the ranges of plasticity index similar to those reported by Vucetic and Dobry (1991). However, the effects of the different load frequencies at 0.1 and 1.0 Hz were insignificant. Further, the parameter $\gamma_{0.7}$, directly observed from Fig. 10, is approximately 0.03–0.07%. These ranges of values correspond to $\gamma_{0.7}$ of the Bangkok Soft Clay with I_p of 30–40 and with the confining pressure of 50–250 kN/m².

Teachavorasinskun et al. (2002b) also conducted a series of cyclic triaxial tests on the Bangkok Soft Clay at the CU site. The applied load frequency of this series was 1 and 0.1 Hz. The soil samples had I_p of 40 and the confining pressure ranged from 50 to 100 kN/m². The authors reported that excess pore pressure started to built up at the level of the axial strain (ε_a) of 0.02–0.2%, which corresponds to the cyclic shear strain amplitude (γ_c) of 0.03–0.3%. These levels of the shear strain amplitude, where the soil commences to behave non-linearly and irrecoverably, refer to the threshold shear strain (γ_{tv}). The concept of using $\gamma_{0.7}$ is closely related to γ_{tv}, as discussed above. The ranges of $\gamma_{0.7}$ as observed from Fig. 10 and γ_{tv} as reported by Teachavorasinskun et al. (2002b) of the Bangkok Soft Clay also coincided with the levels of 0.03–0.07% and 0.03–0.3%, respectively.

The two methods allowed for the calculation of $\gamma_{0.7}$ (Vucetic and Dobry, 1991; Ishibashi and Zhang, 1993) are employed in
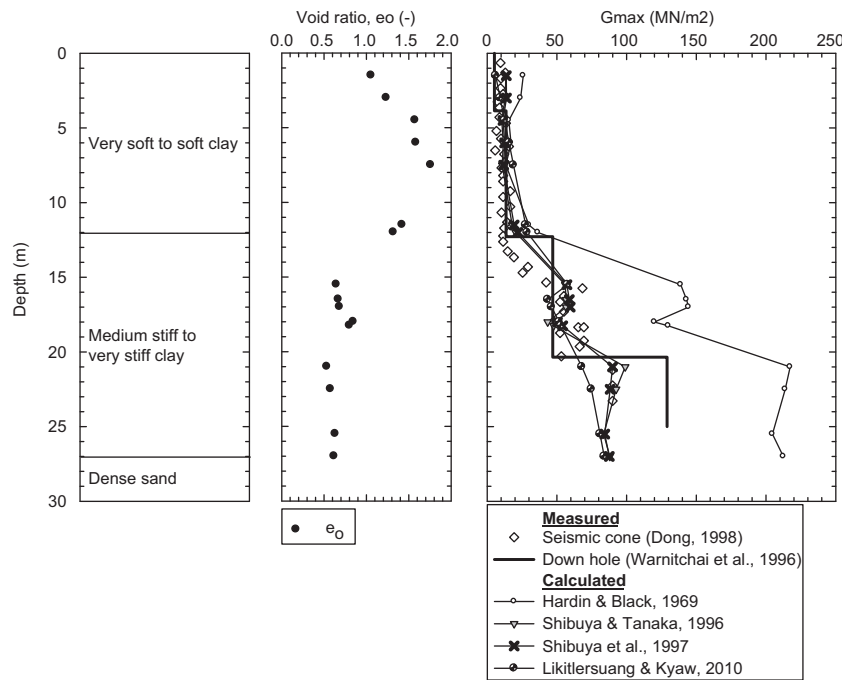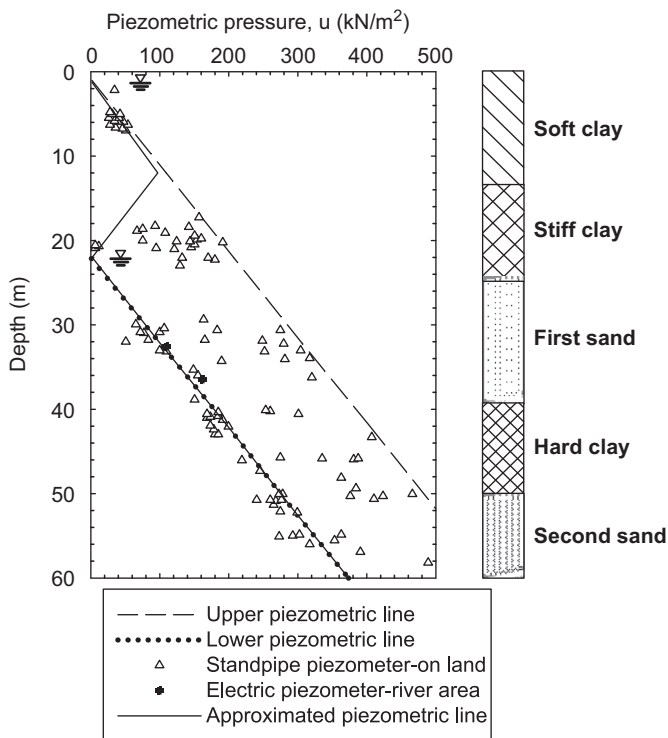
Fig. 6. Measured and predicted G_{max} of Bangkok Clays.

Fig. 7. Piezometric pressure of Bangkok subsoils.

Table 2
Best fit values of constant parameter S for Bangkok Soft and Stiff Clays.

Soil type	Constant parameter S in Eq. (2)	
	Shibuya and Tanaka (1996)	Shibuya et al. (1997)
Soft clay	300	1600
Stiff clay	200	1300

mean stress exceeds 200 kN/m². The ranges of $\gamma_{0.7}$ of 0.03–0.07% were obtained from a soil sample with I_p of 35%–40% and an effective confining pressure of 50–250 kN/m².

With the average values of the plasticity index and the effective mean stress of the Bangkok Soft and Stiff Clays, $\gamma_{0.7}$ is computed using both Vucetic and Dobry (1991) and Ishibashi and Zhang (1993) methods, as shown in Fig. 11. Both methods provide close approximations of $\gamma_{0.7}$ within the very soft to soft clay layers, up to the depth of 12 m. These values of $\gamma_{0.7}$ are also confined within the ranges of $\gamma_{0.7}$ and γ_{tv} of the Bangkok Soft Clay of 0.03–0.07% and 0.03–0.3%, as observed in Fig. 10. On the other hand, the values of $\gamma_{0.7}$ from both methods gave different trends in the medium stiff to very stiff clays, at a depth of 12–27 m. Ishibashi and Zhang (1993) method calculated an approximate constant $\gamma_{0.7}$ of 0.1% along the stiff clays, while Vucetic and Dobry (1991) method gave a trend of $\gamma_{0.7}$ reducing with depth, and values ranging from 0.06% to 0.02%. Unlike the case of the Bangkok Soft Clay, there was no comparable information from the laboratory $\gamma_{0.7}$ of the stiff clays.

5. Discussions and conclusions

The main purpose of this paper is to present the stiffness characteristics at a small strain level of Bangkok Clays, with a focus

this study. Comparisons of $\gamma_{0.7}$, calculated from both methods, are compared and replotted with the measured $\gamma_{0.7}$ of the Bangkok Soft Clay in Fig. 2. Vucetic and Dobry (1991) method gave results within the range of the measured $\gamma_{0.7}$. Ishibashi and Zhang's (1993) method also exhibited good agreement when the lower values of the effective mean stress are applied. Slightly over-predicted values of $\gamma_{0.7}$ are observed when the effective
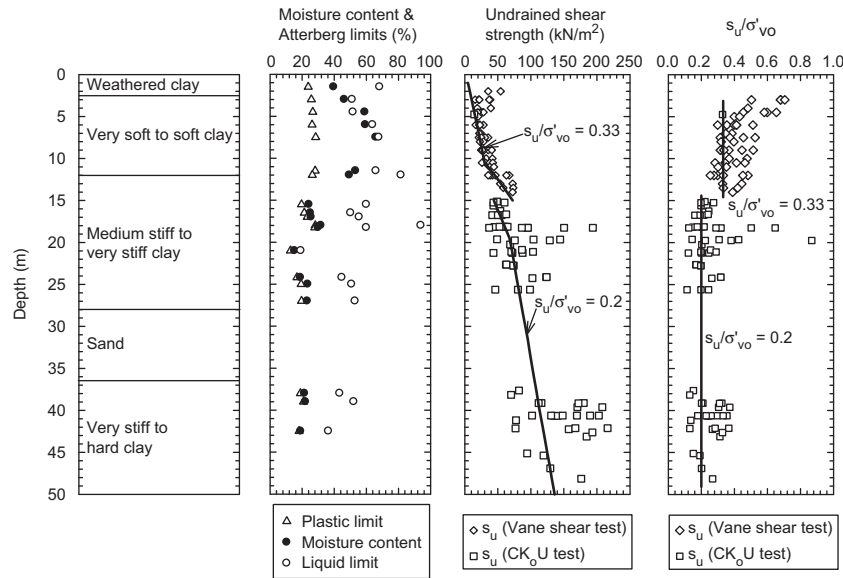
Fig. 8. Undrained shear strength of Bangkok Clays resulting from vane shear and CK$_o$U triaxial tests.
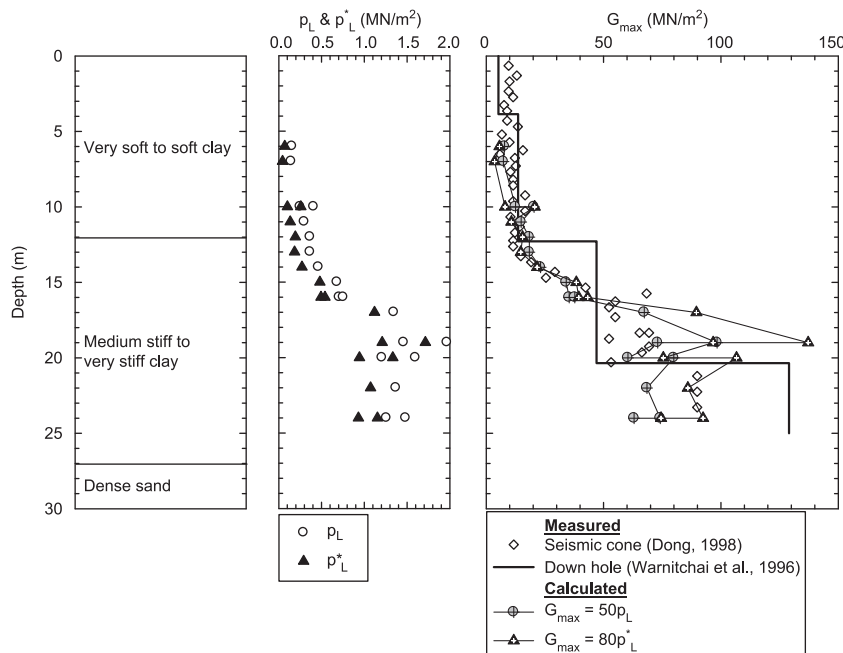

Fig. 9. Correlations of G_{max} from p_L and p_L^* of the LLT pressuremeter test from the Bangkok MRT Blue Line Project.

on two parameters, namely, the small strain shear modulus (G_{max}) and reference shear strain ($\gamma_{0.7}$). For the small strain shear stiffness of Bangkok Clays, the following conclusions can be drawn:

(1) G_{max} of the Bangkok Clays is fairly uniform across Bangkok area. The values of the G_{max} tend to increase with depth with a clear distinction between the soft and stiff clays.
(2) Bangkok Clays are lightly overconsolidated, soft clays and thus the G_{max} values are relatively low compared to the well-documented values of London Clay.
(3) All empirical equations estimated the G_{max} values reasonably well; however, the constant parameters used in the equations need to be calibrated.

(4) Simple correlations between the G_{max} and the LLT pressuremeter parameters are proposed. Basically, the G_{max} values in both the soft and stiff clays are approximately equal to 50 and 80 times that of the limit pressure (p_L) and the net limit pressure (p_L^*), respectively as obtained from LLT pressuremeter tests.

Similarly, conclusions from the studies of $\gamma_{0.7}$ can be summarised as follows:

(1) The concept of using the reference shear strain ($\gamma_{0.7}$) to approximate the volumetric threshold shear strain (γ_{tv}) is validated. For this reason, $\gamma_{0.7}$ can be employed as an input
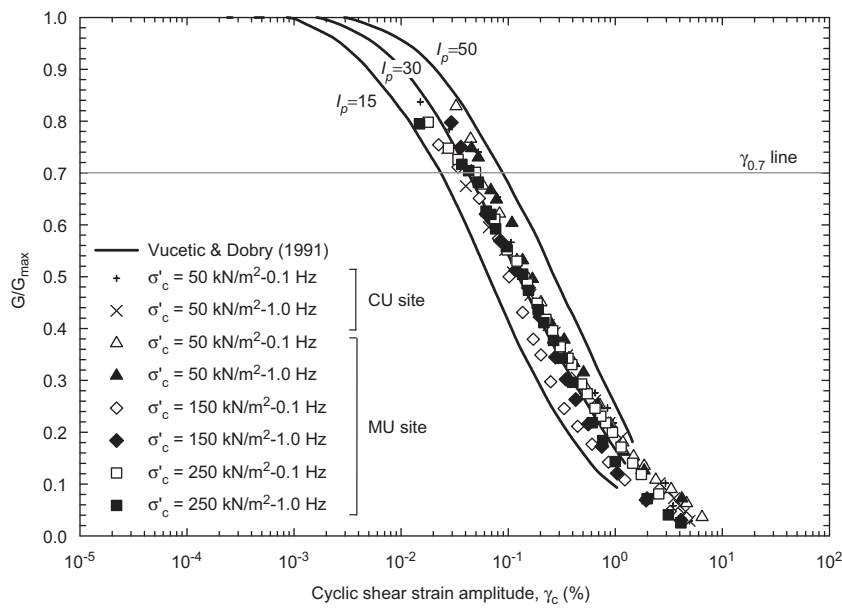
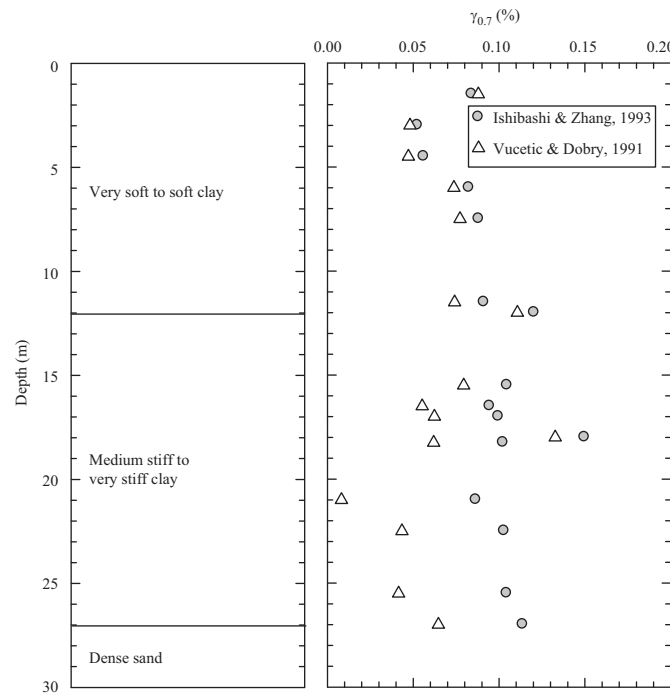Fig. 10. Stiffness reduction curves and $\gamma_{0.7}$ of Bangkok Soft Clay.

Fig. 11. Computed $\gamma_{0.7}$ of Bangkok Clays.

soil parameter in the hardening soil model with small strain stiffness (HSS).

(2) For Bangkok Soft Clay, the $\gamma_{0.7}$ values predicted from Ishibashi and Zhang (1993) and Vucetic and Dobry (1991) are nearly the same. Their predictions are also comparable with the measured values from the laboratory tests. However, for the stiff clays, the predictions from both methods do not completely agree. As a consequence, the predicted values of both the soft and stiff clays may need to be confirmed by further studies.

Acknowledgements

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