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Geotechnical parameters from pressuremeter tests for MRT Blue Line extension in Bangkok

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Abstract. Construction of the extension project of the Bangkok MRT Blue Line underground railway was recently started in 2011. The construction of approximately 5 km long underground tunnel and 4 deep excavations of underground station are considered to be the most important geotechnical works. The pressuremeter was selected as a high-quality in situ testing of the soil to evaluate design parameters for the project. In addition, other field and laboratory tests such as vane shear and CK_0U triaxial tests were included in the investigation programme. This paper aims to present the ground conditions encountered along the MRT Blue Line extension project as well as the site investigation and interpretation techniques with particular focus on the pressuremeter tests. The results are also compared with the pressuremeter investigation from the previous Bangkok MRT project.

Keywords: geotechnical parameters; in situ testing; pressuremeter; mass rapid transit; Bangkok Clay

1. Introduction

The first phase of the Bangkok Mass Rapid Transit (MRT) Underground Railway named the Chaloem Ratchamongkhon (or Blue Line) between Hua Lamphong and Bang Sue was operated since 2004. It comprises approximately 20 km of tunnels, constructed using tunnel boring machines (TBM). Recently, the Blue Line extension project was started the construction in 2011 and is expected to complete in 2015. The extension project from Hua Lamphong to Bang Khae comprises a total length of 14 km (9 km elevated and 5 km underground), including 7 elevated and 4 underground stations. The underground route is to connect the initial MRT route at Hua Lamphong station, then it continues along the underground route along the Rama 4 Road to Charoen Krung Road, Wat Mangkon, Wang Burapha, turning left to Sanam Chai Road passing the Royal Palace, and crossing under the Chao Phraya River at Pak Khlong Talat area.

The important geotechnical works include the design of permanent earth support system for the station and cut-and-cover tunnels. The pressuremeter was selected as a high-quality in situ testing

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of the soil to evaluate selected design parameters for the project. This paper aims to present the ground conditions encountered along the MRT Blue Line extension project as well as the site investigation and interpretation techniques with particular focus on the pressuremeter tests. Moreover, the site investigation for the MRT Blue Line extension project also included the vane shear and the CK_0U triaxial tests. The results of the pressuremeter test could then be compared with the other investigation results as well as the pressuremeter results from the previous studies.

2. Geological condition

Bangkok metropolitan is located on the low flat Chao Praya Delta in the Central Plain region of Thailand. 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 sea levels 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.

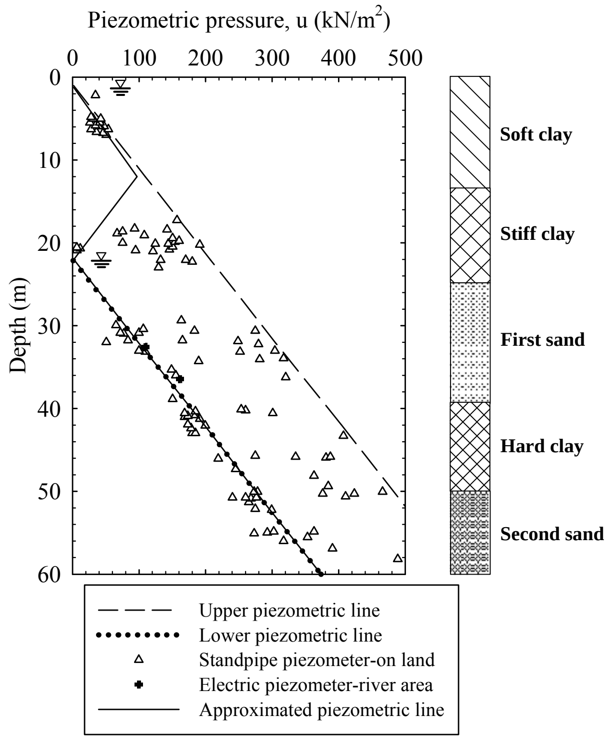


Fig. 1 Piezometric pressure in Bangkok subsoils

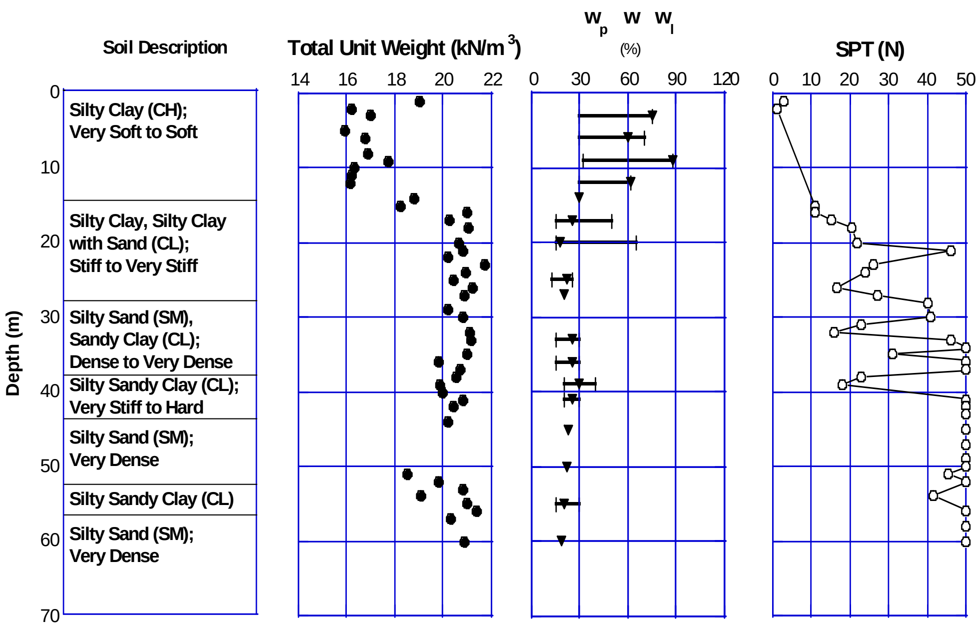


Fig. 2 Soil profile and soil properties for Bangkok MRT Blue Line extension project

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, and clay. While the depth of the bedrock is still undetermined, its level in the Bangkok area is known to vary between 400 m to 1,800 m depth. The aquifer system beneath the city area is very complex and the deep well pumping from the aquifers, over the last fifty years or so, has caused substantial piezometric drawdown in the upper soft and highly compressible clay layer (see Fig. 1).

Field exploration and laboratory tests from both the MRT Blue Line and the MRT Blue Line Extension projects show that the subsoils, down to a maximum drilling depth of approximately 60 to 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. 2.

3. Pressuremeter methods

An idea of determining the geotechnical parameters at in-situ soil condition has led to the development of the pressuremeter tests. A modern type of pressuremeter known as the “Ménard Pressuremeter” (MPM) was first used in Chicago and has become one of the most widely used types of pressuremeter. Conducting the Ménard Pressuremeter test involves lowering a pressuremeter probe into a test pocket that is slightly larger in diameter (see Fig. 3(a)). As a result, the test is normally categorised as a pre-bored pressuremeter (PBP) type. In 1959, Fukuoka and Utsu (as cited in Ménard (1975)) independently developed a device to determine the horizontal subgrade reaction coefficient, K in laterally loaded piles. This form of pressuremeter has then led to the development of the OYO Corporation device, called “Lateral Load Test”. The lateral load test (LLT) pressuremeter shares a similar basic principle with the Ménard Pressuremeter, except that the LLT uses one monocell cylindrical probe instead the tricell system of Ménard Pressuremeter. The insertion of the pressuremeter probe in a pre-bored hole inevitably causes soil disturbance. To overcome this problem, the self–boring pressuremeters (SBP) were developed in the United Kingdom (UK) (Wroth and Hughes 1973) known as the Cambridge pressuremeter. Fig. 3(b) shows the principle of the self-boring pressuremeter instruments. The Pushed-in Pressuremeter (PIP) was primarily developed for use in offshore drilling; however, the recent version was designed for onshore use with cone trucks. If the pushed-in pressuremeter completely displaced the surrounding soil, it was known as the Cone Pressuremeter. Fig. 3(c) illustrates the probe components used in the Pushed-in Pressuremeter. It is noted that the SBP test was carried out for the first phase of the Bangkok MRT Blue Line project; on the other hand the LLT test was performed for the MRT Blue Line extension project.

3.1 Cavity expansion theory

The pressuremeter test has long been recognised as having well-defined boundary conditions and therefore permits a more rigorous theoretical analysis than any other in-situ test. The mathematical framework of cavity expansion theory, based on the assumption that soil mass is a homogeneous, isotropic and continuous medium, is used in the analysis. The elastic solution of the cylindrical cavity problem is well defined (e.g., Timoshenko and Goodier 1970).

An ideal pressuremeter test as illustrated in Fig. 4, the initial cavity pressure (p_i) would be equal to the in-situ total horizontal stress ($\sigma_{h0} = p_0$). The initial volume (V_0), of the cylindrical cavity can be calculated from the initial cavity radius (a_0) and the height of the pressuremeter cavity (h). In the initial part of loading it is assumed that soil behaves elastically and obeys Hooke’s law until the onset of yielding. Using a small strain theory in cylindrical coordinates, the change of volume

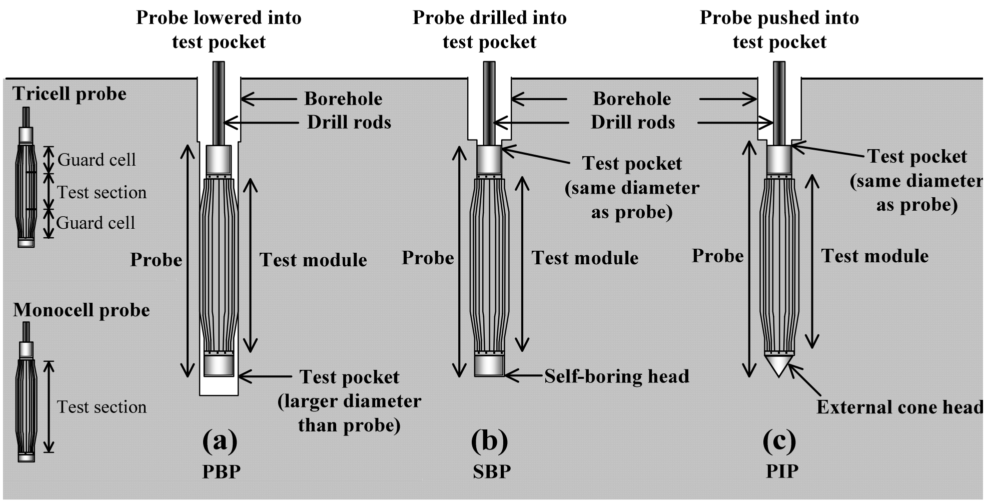


Fig. 3 Pressuremeter probes and test pockets of pre-bored pressuremeter (PBP), self-boring pressuremeter (SBP) and push-in pressuremeter (PIP) (Modified after Clarke, 1995)

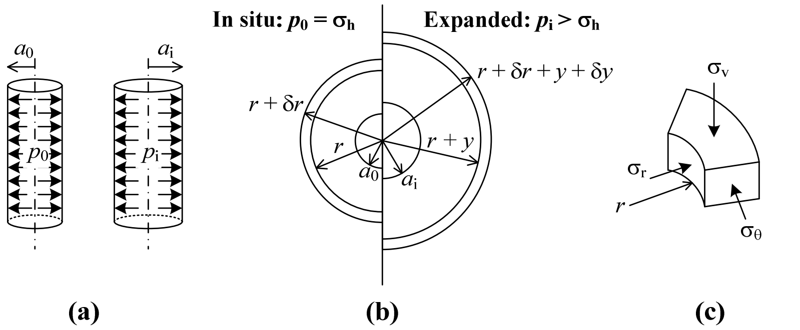


Fig. 4 Definitions used in cavity expansion analysis: (a) expansion of cylindrical cavity; (b) expansion of an element at radius r; and (c) stress action on an element at radius r (Modified after Clarke 1995)

($\Delta V = V - V_0$) can be related to the cavity strain (ε_c) by

$$\frac{\Delta V}{V} = 1 - \frac{1}{(1+\varepsilon_c)^2} \tag{1}$$

The boundary conditions around the cavity are well-defined at the cavity wall in which $r = a$ and $\sigma_r = p$. From the equilibrium equation, the horizontal stress around the cavity can be expressed as

$$p - \sigma_h = 2G\varepsilon \tag{2}$$

where G is the shear modulus of soil. The in situ shear modulus of the soil can be determined by measuring displacement of the cavity wall as the cavity pressure increase above σ_{h0}

$$G = \frac{1}{2}\frac{dp}{d\varepsilon_c} \tag{3}$$

The soil modulus can be also expressed in terms of volumetric strains as

$$G = V_0 \frac{dp}{dV} \tag{4}$$

As pointed out by Mair and Wood (1987), the expression of the cavity, which appears to be a compression process, turns out to be an entirely shearing process. Properties deduced with reference to this analysis concern the shearing and not the compression of the surrounding soil (Schnaid 2009).

3.2 Analysis methods

Here we adopt only undrained analysis applied to pressuremeter tests in Bangkok Clays. Undrained analysis assumes no volume change in plane strain shearing with no strain in the direction parallel to the axis of the cylindrical cavity. All soil elements around cavity are subjected to deformations which are similar in mode but of different magnitudes. The exact solution for the shear stress (τ) at the cavity strain is (Palmer 1972)

$$\tau = \frac{1}{2}\varepsilon_c(1+\varepsilon_c)(2+\varepsilon_c)\frac{dp}{d\varepsilon_c} \tag{5}$$

For small strain values, it can be approximated as

$$\tau \approx \varepsilon_c \frac{dp}{d\varepsilon_c} \tag{6}$$

Eq. (6) can be re-written in terms of the volumetric strain as

$$\tau = \frac{dp}{d\left[\ln\left(\Delta V / V\right)\right]} \tag{7}$$

These expressions allow for construction of the sub-tangent pressuremeter curve as referred to Palmer method. Based on this method, the shear stress can be found from the slope of the pressure-volumetric strain curve.

The undrained shear strength (s_u) determination, based on the elastic-perfectly plastic soil assumption, was developed by Gibson and Anderson (1961). Fig. 5(a) shows the stress-strain relationship of the elastic-perfectly plastic ground, which soil responds elastically until the undrained shear strength of the soil is reached. If the probe is inserted without any disturbance, then the initial cavity pressure should be the same as the initial total horizontal stress (σ_{ho}). At this stage, the small elastic volumetric strain at the onset of yield is given as

$$\frac{\Delta V}{V} \approx \frac{\Delta V}{V_0} = \frac{s_u}{G} \tag{8}$$

As cavity pressure increases, a plastic region develops around the cavity reaching the elastic-plastic boundary formed around the expanding probe, In this region the change in pressure can be obtained by integrating Eq. (7) with respect to ln($\Delta V/V$) to give

$$p = \sigma_{h0} + s_u\left[1 + \ln\left(\frac{G}{s_u}\right) + \ln\left(\frac{\Delta V}{V}\right)\right] \tag{9}$$

When the soil deforms plastically, the cavity pressure does not increase indefinitely and a limit pressure is gradually approached for $\Delta V/V = 1$. By substituting this value in Eq. (9), a limit pressure (p_L) is written as

$$p_L = \sigma_{h0} + s_u\left[1 + \ln\left(\frac{G}{s_u}\right)\right] \tag{10}$$

This expression, proposed by Ménard (1975), demonstrates that the cavity limit pressure depends strongly on the undrained shear strength and the shear modulus of the soil. For the response of the pressuremeter test in the plastic phase, where $\sigma_{h0} + s_u \le p \le p_L$, Eq. (10) can be conveniently written as

$$p = p_L + s_u \ln\left(\frac{\Delta V}{V}\right) \tag{11}$$

This solution indicates that if the pressuremeter test results are plotted in terms of cavity pressure (p) against the logarithm of the volumetric strain ($\ln(\Delta V/V)$), the results of the plastic portion should lie on a straight line with a slope equal to the undrained shear strength of the soil (s_u) as illustrated in Fig. 5(b). The cavity pressure at $\Delta V/V = 1$ or $\ln(\Delta V/V) = 0$ is then equal to the limit pressure (p_L).

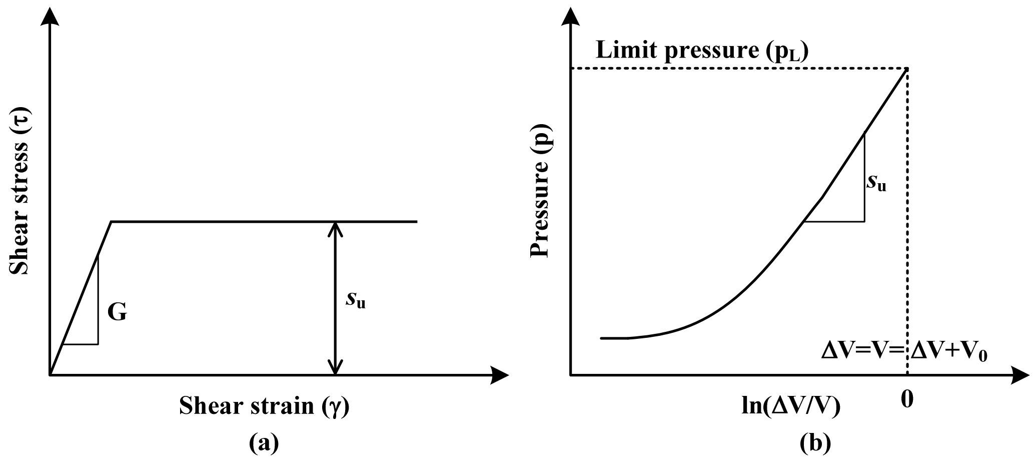


Fig. 5 Undrained shear strength determination from pressuremeter test in clay (Redraw from Schnaid, 2009)

Marsland and Randolph (1977) proposed methods for the shear strength determination from the limit pressure (p_L). In this method, the total horizontal stress (σ_{h0}), shear modulus (G), and limit pressure (p_L) are estimated from an iteration process of fitting the values of σ_{h0} and s_u with the yield point of the test curve. Eq. (10) can be rearranged as

$$s_u = \frac{p_L - \sigma_{h0}}{1 + \ln\left(\frac{G}{s_u}\right)} = \frac{p_L^*}{N_p} \tag{12}$$

where $p_L^* = p_L - \sigma_{h0}$ = net limit pressure and N_p = pressuremeter constant.

Hawkins *et al*. (1990) modified the Marsland and Randolph method to determine the coefficient of earth pressure at rest. This method still uses the iteration process similar to the Marsland & Randolph method; however, instead of using Palmer's peak strength the undrained shear strength is obtained by plotting the p vs ε_c and p vs $\ln(\Delta V/V)$ curves. The undrained shear strength is defined as the slope of p vs $\ln(\Delta V/V)$ curve at the yield pressure (p_y) that is

$$p_y - \sigma_{h0} = \frac{dp}{d[\ln(\Delta V/V)]} \tag{13}$$

The interpretation from the methods based on the infinitely long cylindrical cavity expansion theory leads to the overestimation of the shear strength parameters. Several researchers (e.g., Houlsby and Carter 1993, Shuttle and Jefferies 1995, Yu *et al*. 2005) have conducted parametric studies with the aid of the finite element analysis. The effects of the length to the diameter ratio (L/D), the depth of penetration to the diameter ratio (H/D), the rigidity index ($I_r = G/s_u$), the types of soil models, the overconsolidation ratio (OCR), the strain range over which shear strength is deduced, and the soil initial state of stress are included in the existing studies. The results were presented in terms of the correction factor (β) to be multiplied with the undrained shear strength derived from the infinitely long cylindrical cavity expansion. The results of this multiplication are called the true values of the undrained shear strength, which would have been measured if the pressuremeter was infinitely long. Parametric studies using the Mohr-Coulomb model (Houlsby and Carter 1993, Shuttle and Jefferies 1995) revealed that the length to the diameter ratio (L/D) and the soil rigidity index (G/s_u) are both significant factors affecting the correction factor (β). The effect of the depth of penetration to the diameter ratio (H/D) is minor and negligible. Similar parametric studies, using critical state soil models were conducted by Yu *et al.* (2005). In the critical state models used in their analyses, the shear modulus (G) is assumed to be a linear function of the mean effective stress (p'); a constant Poisson's ratio is also assumed. When the shear modulus (G) changes with the mean effective stress (p'), the values of the shear modulus vary with the applied pressure (as the process of pressuremeter testing proceeds further in the finite element model). This variation is not applied when the shear modulus is assumed constant in the Mohr-Coulomb model. Hence, the effects of the rigidity index (G/s_u) on the correction factor (β), when the critical state models are used, are not meaningful. As a consequence, Yu *et. al.* (2005) focused on the effect of the OCR, K_o and the used constitutive soil models instead. They concluded that only the OCR had a significant effect on the overestimation of the undrained shear strength, while K_o and the used constitutive soil models were insignificant.

4. Pressuremeter investigation for the MRT Blue Line project

The pressuremeter investigation was adopted for the MRT Blue Line project to provide high-quality in situ testing with minimum sample disturbance. The ground conditions of soft to stiff Bangkok Clays were ideal for this test. The design objectives for these investigations were to estimate the in situ total horizontal stress, the undrained shear strength, and the soil stiffness and its variation with strain. The self-boring pressuremeter (SBP) tests were carried out for the first MRT Blue Line project and their results were well-documented by Prust *et al*. (2005). On the other hand, the recent pressuremeter investigation for the extension project was performed using the OYO type of pressurmeter called lateral load test (LLT).

4.1 Previous study

The previous studies on pressuremeter in Bangkok subsoils were conducted at the Asian Institute of Technology (AIT) (e.g., Huang 1980, Surya 1981, Bergado *et al*. 1986) and the Bangkok Mass Rapid Transit (MRT) Blue Line project (Prust *et al*. 2005). These involve both the pre-bored pressuremeter (PBP) of the LLT type (Fig. 3(a)) and the self-boring pressuremeter (SBP) of the Cambridge type (Fig. 3(b)). Details of both types of pressuremeter tests are presented in Table 1.

Table 1 Pressuremeter tests conducted in Bangkok subsoils

Type	Probe			Test location	Subsoils tested	Reference
	Diameter (mm)	Length (mm)	*L/D*			
LLT	70	600	8.57	AIT campus	Soft and stiff clays	Huang (1980) Surya (1981) Begado *et al.* (1986)
SBP	83.1	500	6.02	Bangkok MRT Blue Line project	Soft and stiff clays and dense sand	Prust *et al.* (2005)
LLT	70	600	8.57	Bangkok MRT Blue Line extension project	Soft and stiff clays	The current study

Table 2 Comparisons of soil parameters from SBP, conventional investigations and back-analysis results (Prust *et al*. 2005)

Soil layer	Conventional investigation	SBP	Back-analyses
	Earth pressure coefficient, K_0 (-)		
Soft clay	0.75^*	0.1 to 0.3	0.75
Stiff clay	0.4 to 1.0^*	0.2 to 1.4	0.65
Dens sand	0.4^*	0.2 to 1.3	-
	Shear strength, s_u (kN/m^2)		
Soft clay	$s_u/\sigma'_{vo} = 0.35$	$s_u/\sigma'_{vo} = 0.45$	$s_u/\sigma'_{vo} = 0.45$
Stiff clay	$s_u = 50 + 7.8z^+$	$s_u = 100 + 15.6z^+$	$s_u = 100 + 15.6z^+$
Dens sand	$\phi' = 36°$	$\phi' = 35 - 37°$	$\phi' = 36°$
	Stiffness ratio (E_u/s_u)		
Soft clay	$E_u = 400$ to $500s_u$	$E_u = 500s_u$ ($\varepsilon = 0.1 - 0.2\%$)	$E_u = 700s_u$
Stiff clay	$E_u = 500s_u$ (above 18 m) $E_u = 1000s_u$ (below 18 m)	$E_u = 500s_u$ (above 18 m) $E_u = 1000s_u$ (below 18 m) ($\varepsilon = 0.05$ to 0.1%)	$E_u = 1000s_u$
Dens sand	$E'/N^\# = 0.8$ to 4.0	$E'/N^\# = 1.9$ to 7.9	-

*Based on Jaky's formula: $K_0 = 1 - \sin\phi'$, ^+z is the depth below ground surface, $^\#N$ is SPT N value

In the early 1980s, the LLT tests were conducted, in the main, on weathered clay, Bangkok Soft and Stiff Clays up 15 m deep at the AIT campus. The classical cavity expansion theories, such as those of Gibson and Anderson (1961), Palmer (1972), were utilised in the soil parameter interpretations. The results were compared and correlated with the in situ vane shear test and the Cone Penetration Test (CPT). Bergado *et al*. (1986) summarized the LLT tests and related works undertaken at the AIT. They concluded that the undrained shear strength from the LLT tests ($s_{u\mathrm{PMT}}$) was 10 to 25 percent higher than the undrained shear strength from the vane shear ($s_{u\mathrm{FV}}$).

The SBP test was first engineering practice in Thailand for the design of underground station diaphragm walls in the Bangkok MRT Blue Line project (Prust *et al*. 2005). A total of six SBP tests were conducted in Bangkok Soft Clay, Stiff Clay and Dense Sand layers up to 40 m deep. Conventional site investigation programs were also employed, including wash-boring, vane shear test, standard penetration test (SPT), and triaxial tests. The results of the SBP test interpretation were compared with those obtained from conventional methods (i.e., vane shear, triaxial tests, and empirical correlations), and the back-analysis of wall deflection. The results are summarised in Table 2.

From the previous study of the SBP test for the first phase of MRT Blue Line project, the stiffness ratio (G/s_u) and shear strain relationship of Bangkok clays can be summarised as shown in Fig. 6. While the stiffness ratios of both Soft and Stiff Clay layers are not constant, the G/s_u ratio degrades with increasing shear strain level. Indeed, the typical range of the shear strain, resulting from the diaphragm wall movement, is from 0.1 to 0.2% and 0.05 to 0.1% for the Bangkok Soft and Stiff Clays, respectively (Teparaksa 1999, Prust *et al.* 2005). In the case of the tunnel excavation, Mair (1993) suggested that the range of shear strain, induced by the tunnelling, should be in the order of 0.1 to 1%. The shear strain ranges together with the ratios G/s_u according to the SBP in the Bangkok Soft and Stiff Clays for deep excavation and tunnelling, are also shown in Fig. 6.

4.2 Current study

In the current study, the pre-bored pressuremeter tests of the LLT type were performed along the alignment of the Bangkok MRT Blue Line project as presented earlier in Fig. 2. The OYO LLT pressuremeter model 4165 (Type M), with a cell diameter of 70 mm and a length of 600 mm, was used. Four LLT tests were conducted at each station location up to a depth of 24 m. The subsoils tested were Very Soft to Soft Clays and Medium Stiff to Stiff Clays. Table 3 summarizes the LLT tests locations in this study.

During the test, the corrected pressure (p), which is defined as the cell pressure corrected with the reaction of cell rubber and the hydrostatic pressure, is measured. This pressure is calculated by

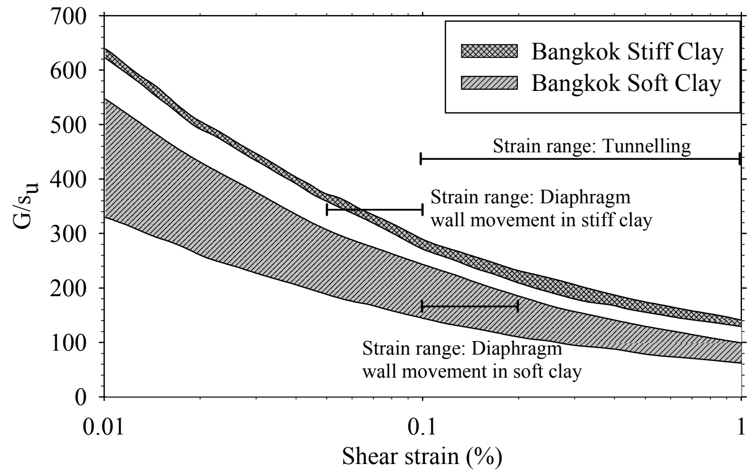


Fig. 6 Variations of G/s_u with shear strain from self-boring pressuremeter tests in Bangkok Soft and Stiff Clays (Modified after Prust *et al.*, 2005)

$$p = p_m + p_s - p_g \tag{14}$$

where p_m is the cell pressure, p_s is the maximum value of all $p_g - p_m$ values (in this case it is ranged from 0–6 kN/m^2), p_g is the gas pressure which is obtained from calibrated reading curve.

Equal step of pressure increments are applied and maintained for two minutes each. The change of membrane volume (ΔV) is recorded after 15, 30, 60 and 120 seconds. The creep volume is defined as the change of injected volume between the 30 and 120 second readings (i.e., V_{120s}–V_{30s}). The results of the pressuremeter tests are generally plotted as corrected pressure (p) versus the radius of probe (r) as shown in Fig. 7. In this plot, the limit pressure (p_L) can be defined as the maximum corrected pressure. Furthermore, a creep curve can be constructed by plotting the corrected pressure (p) against the creep volume ($V_{120s} - V_{30s}$) as also shown with a dashed line in Fig. 7. The benefit of the creep curve is to aid in locating the initial pressure (p_i) and the yield

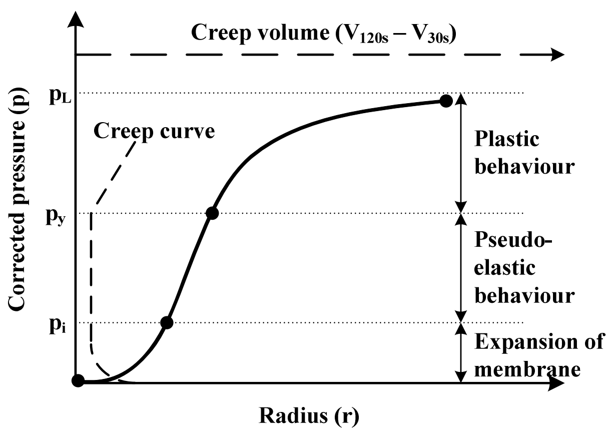


Fig. 7 Typical result for the LLT pressuremeter test

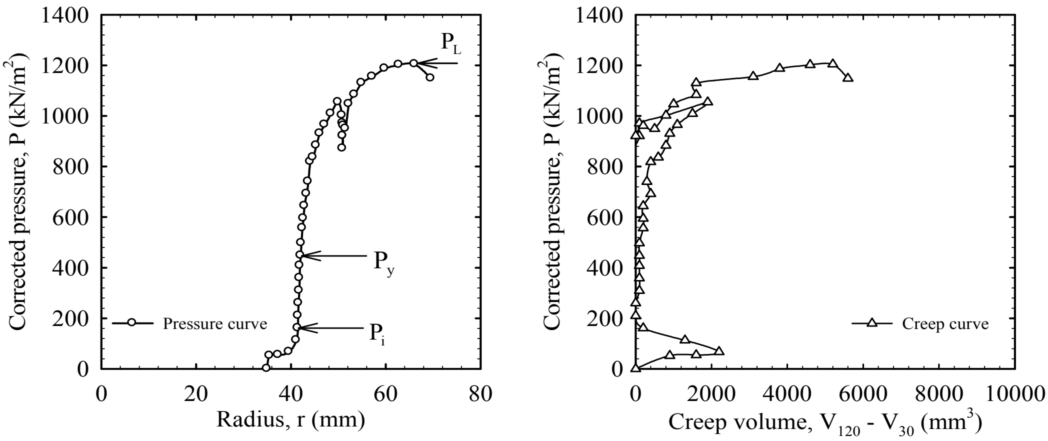


Fig. 8 Pressuremeter test result at Wang Burapha Sation in very stiff clay layer at depth of 17 m

Table 3 Summary of the LLT testing results along the Bangkok MRT Blue Line extension

Station	Depth (m)	Soil types	p_i (kN/m^2)	p_y (kN/m^2)	p_L (kN/m^2)
Wat Mangkon Station	9	Soft clay	-	-	-
	13	Medium stiff clay	154	247	367
	16	Stiff clay	-	-	-
	21	Very stiff clay	-	-	-
Wang Burapha Station	6	Soft clay	57	77	156
	12	Medium stiff clay	145	188	367
	17	Very stiff clay	180	448	1247
	22	Very stiff clay	175	543	1370
Sanam Chai Station	10	Soft clay	89	126	250
	14	Medium stiff clay	189	260	463
	19	Very stiff clay	225	701	1967
	24	Very stiff clay	298	679	1483
Itsaraphap Station	8	Soft clay	-	-	-
	11	Soft clay	-	-	-
	16	Stiff clay	250	354	707
	20	Very stiff clay	321	516	1208
Bang Wa Station	10	Soft clay	-	155	408
	15	Stiff clay	190	345	682
	19	Very stiff clay	210	505	1461
	24	Very stiff clay	349	555	1260
Phet Kasem 48 Station	7	Soft clay	65	80	146
	11	Medium stiff clay	152	188	298
	16	Very stiff clay	200	490	1259
	20	Very stiff clay	365	660	1599

pressure (p_y). The initial pressure (p_i) is the pressure necessary to achieve the initial contact between the probe cell and the borehole wall, in which it can be used to determine the in situ horizontal total stress (σ_{h0}). On the other hand, the yield pressure (p_y) is the pressure where the plastic strain occurs in the soil and it could be employed to calculate the undrained shear strength (s_u).

An example of the LLT testing result in the stiff clay layer at 17 m of the Wang Burapha Station is presented in Fig. 8. The initial pressure (p_i) and the yield pressure (p_y) can be determined using creep cure; however, the limit pressure (p_L) is estimated from the maximum applied corrected pressure as illustrated in Fig. 8. Other testing results are summarised in Table 3.

5. Geotechnical parameters interpreted from pressuremeter tests

The interpretation of geotechnical parameters obtained from pressuremeter tests on Bangkok subsoil is addressed here with emphasis on the application in the underground construction for the MRT project. The total horizontal stress (σ_{h0}) or coefficient of earth pressure (K_0), the undrained shear strength (s_u), and the soil moduli are discussed.

5.1 Total horizontal stress (σ_{h0}) and coefficient of earth pressure at rest (K_0)

The determination of the horizontal stress (σ_{h0}) was conducted using three available methods: the creep curve method, the Marsland & Randolph method (Marsland and Randolph 1977), and the Hawkins method (Hawkins *et al*. 1990). After the values of the horizontal stress were obtained, the coefficient of the earth pressure at rest (K_0) is calculated by

$$K_0 = \frac{\sigma'_{h0}}{\sigma'_{v0}} = \frac{\sigma_{h0} - u}{\sigma_{v0} - u} \tag{15}$$

where σ'_{v0}, σ'_{h0} are the effective vertical and horizontal stresses respectively, σ_{v0}, σ_{h0} are the total vertical and horizontal stresses respectively and u is the piezometric pressure of groundwater.

The piezometric head of Bangkok groundwater pressure was not hydrostatic, due to extensive deep well pumping undertaken in the 1970s. Accordingly, the standpipe piezometer (on the land area) and the electric piezometer (under the river area) were employed to measure the piezometric pressure of Bangkok subsoils. The results of the measured piezometric pressure along the Bangkok MRT Blue Line Extension project area were shown earlier in Fig. 1. The groundwater level (GWL) on the project site ranged from 1.0 to 1.5 m deep. This GWL was heavily influenced by the fluctuation of water in the canal system, especially in the rainy season. At the top of the first sand layer (approximately 21 to 25 m depth), the piezometric pressure approached a value of zero, indicating that there was a drawdown of water flow from the Bangkok soft clay and the first stiff clay layers to the first sand layer. Based on the above information on piezometric pressure and the drawdown water flow, an approximated piezometric line can be drawn as shown in Fig. 1. This approximated line is used in Eq. 15 for the calculation of piezometric pressure, u and K_0 value.

The total horizontal stress and the coefficient of earth pressure at rest obtained from three different methods are presented in Fig. 9. In soft clay layer, the interpretation using creep curve method for the total horizontal stress was not included. This is because the curves do not clearly exhibit the pseudo-elastic behaviour in soft clays. Both the Marsland and Randolph, and Hawkins methods gave very low values of the total horizontal stresses in the soft clay layer. Indeed, the total horizontal stresses obtained from the Marsland and Randolph method were lower than the piezometric pressure which, in turn, gave negative values for the earth pressure coefficient. Similarly, the total horizontal stress from the Hawkins method was slightly higher than the piezometric pressure which gave an earth pressure coefficient of close to zero.

Furthermore, the earth pressure coefficient values in the stiff clay layer, calculated using the above three methods, fell in the range of 0.35 to 0.75. The creep curve, and Marsland & Randolph methods yielded similar results, with the average K_0 of 0.46 ± 0.18 and 0.45 ± 0.19, respectively. The Hawkins method yielded an average value of K_0 of 0.68 ± 0.14. Prust *et al*. (2005), in their interpretation of the K_0 values resulting from self-boring pressuremeter tests, empirical correlations and finite element back-analysis (see Table 2), concluded that the values of K_0 were 0.75 and 0.65 for the Bangkok Soft Clay and Stiff Clay, respectively. Their K_0 in stiff clay of 0.65 was somewhat close to the average value calculated by Hawkins method of 0.68.

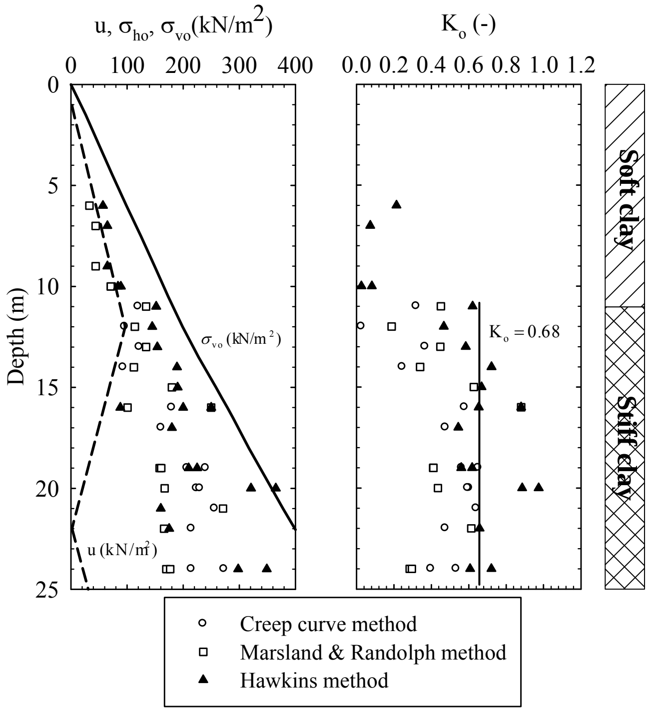


Fig. 9 Total horizontal stresses and coefficient of earth pressure at rest from LLT Tests

5.2 Undrained shear strength (s_u)

The undrained shear strength (s_u) can be obtained from various methods such as the peak shear strength by Palmer method (Palmer 1972) and the perfectly plastic analysis by Gibson & Anderson method (Gibson and Anderson 1961). The undrained shear strength can also determined from the limit pressure (p_L) and the modulus of rigidity (G/s_u) as well as with the pressuremeter constant (N_p) by Marsland and Randolph (1977). The interpretation of s_u from the LLT tests as presented in Table 4 consisted of 5 different methods: (i) Palmer method, (ii) Gibson and Anderson method, (iii) Eq. 12 with input of the limit pressure (p_L) and the initial shear modulus (G_i), (iv) Eq. 12 with input of the limit pressure (p_L) and the unload/reload shear modulus (G_{ur}) and (v) Eq. 12 with input of the limit pressure (p_L) and the pressuremeter constant (N_p). The N_p values adopted in this study are 6 for soft clay (at 0–15 m depth) and 6.7 for stiff clay (at 15–25 m depth), respectively.

The interpretation of s_u from the Gibson & Anderson method, the Eq. 12 with p_L and G_i and the Eq. 12 with p_L and N_p are also compared with the results from the vane shear test in soft to medium stiff clay layer and the CK_0U triaxial test in stiff to very stiff clay layer. It is noted that the shear strength investigation for the MRT Blue Line extension project includes the vane shear test and the CK_0U triaxial test in soft to medium stiff clay and stiff to very stiff clay layers, respectively. A plot showing the s_u values from various methods with depth is presented in Fig. 10. It is also noted that the s_u values from the vane shear tests were adjusted by Bjerrum's correction factor.

Table 4 Undrained shear strength (s_u) interpretation from LLT Tests

Station	Depth (m)	s_u (kN/m^2)				
		Palmer	Gibson & Anderson	$\dfrac{p_L-\sigma_{h0}}{1+\ln\left(\dfrac{G_i}{s_u}\right)}$	$\dfrac{p_L-\sigma_{h0}}{1+\ln\left(\dfrac{G_{ur}}{s_u}\right)}$	$\dfrac{p_L-\sigma_{h0}}{N_p}$
Wat Mangkon Station	9	-	-	-	-	-
	13	-	-	84.0	-	35.5
	16	-	-	-	-	-
	21	-	-	-	-	-
Wang Burapha Station	6	47.1	19.8	22.1	18.5	16.5
	12	70.5	43.1	47.7	45.6	37.1
	17	-	254.7	246.9	240.0	174.1
	22	427.5	367.2	337.6	231.3	178.3
Sanam Chai Station	10	68.3	36.8	40.9	34.7	26.8
	14	148.8	67.3	59.0	58.3	45.7
	19	515.0	476.0	436.5	356.0	260.0
	24	504.0	381.1	355.9	281.7	176.9
Itsaraphap Station	8	-	-	-	-	-
	11	-	-	-	-	-
	16	97.3	103.7	93.0	78.0	68.3
	20	349.0	193.7	227.2	153.5	132.4
Bang Wa Station	10	-	106.6	-	-	-
	15	143.3	133.7	117.9	76.4	81.9
	19	481.0	297.5	330.0	216.1	186.7
	24	450.0	206.4	204.5	149.8	136.0
Phet Kasem 48 Station	7	36.1	15.6	17.1	14.3	13.5
	11	55.6	35.3	33.9	26.0	24.3
	16	-	290.6	294.0	239.7	158.0
	20	-	286.4	415.2	268.4	184.1

The undrained shear strength in the Bangkok soft clay indicated a reasonable agreement among Gibson & Anderson method, the Eq. 12 with G_i and G_{ur}, the Eq. 12 with N_p methods and field vane shear strength. The trend of the undrained shear strength increased with depth, having the values of 15 to 60 kN/m^2 from depths of 5 to 13 m. The analysis of the Palmer method generally yielded higher values of the undrained shear strength compared to all the other methods. In the stiff clay layer, all the interpretation methods showed a relatively high degree of scatter. However, the tendencies of the undrained shear strength increased with depth, as shown by the solid lines in Fig. 10. In addition, the undrained shear strength from the Gibson & Anderson method, the Eq. 12 with G_i and G_{ur}, the Eq. 12 with N_p methods are two, three and four times higher than the CK_0U triaxial undrained shear strength.

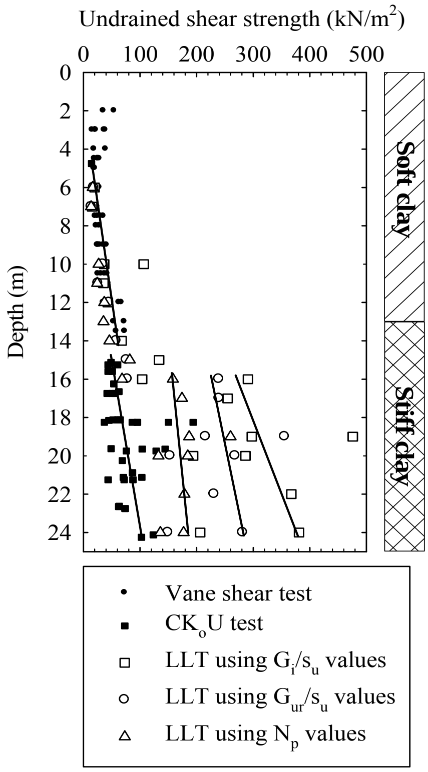


Fig. 10 Comparison of undrained shear strength from field vane shear test, CK_0U triaxial test and LLT tests

Table 5 Correction factor due to pressuremeter geometry

Pressuemeter type*	L/D	Soil model+	Correction factor, $\beta^{\#}$ at 2 - 5% strain		References
			Soft clay	Stiff clay	
LLT	8.57	MC	0.858	0.813	Houlsby and Carter (1993)
SBP	6.02	MC	0.815	0.764	
SBP	6.02	MC	0.910	0.875	Shuttle and Jefferies (1995)
LLT	8.57	MCC	0.891	0.896	Yu *et al.* (2005)
SBP	6.02	MCC	0.867	0.872	

* LLT = Later Load Test, SBP = Self-boring Pressuremeter
+ MC = Mohr-Coulomb, MCC = Modified Cam Clay
\# For Bangkok Clay, G/s_u = 150 and 300 are assumed for soft clay and stiff clay in the MC analysis, and OCR = 1.5 and 1.6 are assumed for soft clay and stiff clay in the MCC analysis

However, these interpretations of the s_u values should be reduced by correction factor (β) due to pressuremeter geometry as suggested by Houlsby and Carter (1993), Shuttle and Jefferies (1995) and Yu *et al.* (2005). The correction factors for these cases are summarised in Table 5. It can be seen from the table that all the analyses resulted in a 10–25% reduction of the undrained shear strength for both the LLT and SBP. In the case of soft clay layer, 10–15% reduction factor would have insignificant effect to the prediction compared to the result from the vane shear tests. However, in the stiff clay layer, β = 0.813 and 0.896 from the Mohr-Coulomb and the Modified Cam Clay analyses would not bring the s_u from the LLT close to the results from the CK_0U tests.

As a result, the empirical approaches are selected to correlate the undrained shear strength from the limit pressure (p_L) or the net limit pressure (p_L^*). An empirical equation for prediction the undrained shear strength of Bangkok soft clay was suggested by Bergado *et al*. (1986)

$$s_{u,FV} = \frac{p_L}{5.9} \tag{16}$$

where $s_{u,FV}$ = undrained shear strength from the field vane shear test.

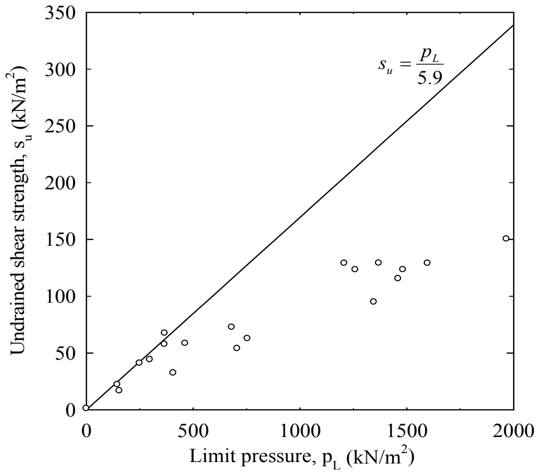
Fig. 11(a) shows the data of the undrained shear strength versus the limit pressure of both the soft and stiff clays from LLT tests including the plot of Eq. 16. It is noted that the undrained shear strength in the soft clay (0–15 m depth) and in the stiff clay (15–25 m) was taken from the vane shear and the CK_0U triaxial undrained tests, respectively. It can be seen that Eq. 16 agrees well with the plot of p_L versus s_u up to the undrained shear strength of 70 kN/m^2. This agreement is reasonable as Eq. 16 was calibrated with the vane shear strength in the soft clay layer only.

Additionally, the Eq. 12 with N_p = 5.5 as suggested by Ménard (1975) can be used to make a correlation between the undrained shear strength with the net limit pressure (p_L^*). Another non-linear correlation of the undrained shear strength with the p_L^* value was also suggested by Briaud (1992)

$$s_u = 0.67 p_L^{*0.75} \tag{17}$$

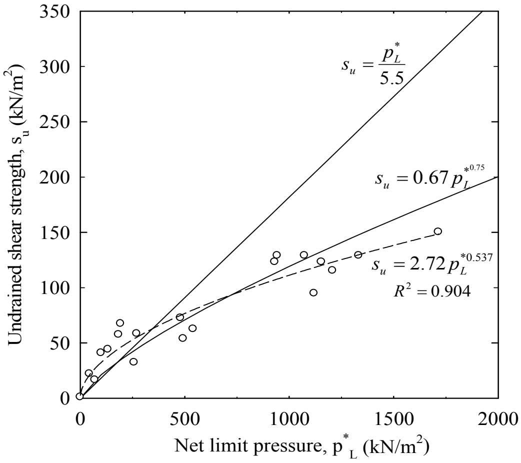
where both s_u and p_L^* are in kN/m^2.

These correlations are plotted to compare with the data of s_u versus p_L^* from LLT tests as shown in Fig. 11(b). The linear function of Eq. 12 with N_p = 5.5 does not show a similar trend to that of the plot of p_L^* versus s_u. Eq. 17, however, gives a much better prediction when compared with Eq.



(a) Correlation between undrained shear strength (s_u) and limit pressure (p_L)

Fig. 11 Continued



(b) Correlations between undrained shear strength (s_u) and net limit pressure (p^*_L)

Fig. 11 Correlations of the undrained shear strength with limit pressure and net limit pressure from LLT results

Table 6 Undrained moduli from LLT Tests

Station	Depth (m)	E_i (MN/m^2)	E_{ur} (MN/m^2)	E_{ur} / E_i
Wat Mangkon Station	9	0.3	-	-
	13	1.2	-	-
	16	3.7	-	-
	21	18.7	141.3	7.54
Wang Burapha Station	6	2.1	4.2	2.02
	12	5.6	6.6	1.18
	17	30.7	34.2	1.11
	22	12.8	44.7	3.48
Sanam Chai Station	10	2.3	4.0	1.72
	14	6.8	7.1	1.04
	19	26.1	52.5	2.01
	24	11.0	20.9	1.90
Itsaraphap Station	8	1.2	-	-
	11	0.6	-	11.63
	16	14.1	30.3	2.15
	20	12.4	54.7	4.40
Bang Wa Station	10	2.4	8.3	3.44
	15	8.4	52.7	6.25
	19	16.1	77.8	4.82
	24	19.4	72.4	3.73
Phet Kasem 48 Station	7	2.1	4.5	2.11
	11	2.7	7.7	2.81
	16	11.9	21.9	1.84
	20	8.9	29.3	3.28

12. Nevertheless, it seems to overestimate the undrained shear strength at a higher range of the net limit pressure. The best fit (dashed) line resulting from the regression analysis is also plotted in Fig. 11(b). A reasonably high value of $R^2 = 0.904$ was obtained by the following equation

$$s_u = 2.72 p_L^{*0.537} \tag{18}$$

where both s_u and p_L^* are in kN/m^2.

5.3 Soil moduli

Following the procedures as described above, the initial and unloading/reloading shear moduli could be obtained from the pressuremeter curves. These two shear moduli were then converted to the initial and unloading/reloading pressuremeter moduli (E_i and E_{ur}), using the elastic theory with the undrained Poisson's ratio (ν_u = 0.5) i.e., $E_i = 3G_i$ and $E_{ur} = 3G_{ur}$. The values of the pressuremeter moduli were determined from the initial and unloading/reloading curves (see Table 6), and were also plotted with depth (see Fig. 12).

The tendencies of both the E_i and E_{ur} show an approximate linear increase with depth. However, the linear relationships are clearly separated between the soft and stiff clay layers. In general, the E_{ur} values are 2 and 3.5 times higher than the E_i in soft and stiff clays, respectively. When the E_i values are compared with the E_{50} obtained from the CK_0U triaxial tests, they seem to locate on the upper values of E_{50}. This outcome is possibly due to the lesser degree of soil disturbance caused by the LLT tests.

6. Conclusions

This research study focuses on the interpretation of the geotechnical parameters from the LLT pressuremeter test from the Bangkok MRT Blue Line extension project. The study deals with three main geotechnical parameters: coefficient of earth pressure at rest (K_o), undrained shear strength (s_u) and pressuremeter moduli (E_i and E_{ur}). The following conclusions are drawn.

(i) An average value of K_0 of 0.68 ± 0.14 from Hawkins method (Hawkins *et al.*, 1990) can be used as an input parameter for stiff clay initial stresses calculation. None of the methods, studied here, give reasonable value of K_0 for soft clay layer.

(ii) According to the results in this study, the correlations from the net limit pressure (p_L^*) provides reasonable values of the field undrained shear strength. As a result, the non-linear correction such as Eq. 18 is suggested for both cases of Bangkok soft and stiff clays.

(iii) The values of the pressuremeter initial tangent modulus (E_i) are comparable with the secant modulus at 50% of undrained shear strength (E_{50}) from CK_0U test. Therefore, this might be a better choice when it comes to the selection of stiffness modulus. However, care must be taken in terms of the determination of initial pressure, where E_i is obtained.

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