Geotech Geol Eng (2013) 31:1511–1524
DOI 10.1007/s10706-013-9675-x

ORIGINAL PAPER

Undrained Shear Strength of Normally Consolidated and Overconsolidated Clays from Pressuremeter Tests: A Case Study

Ali Soleimanbeigi

Received: 22 March 2013 / Accepted: 3 July 2013 / Published online: 10 July 2013

Abstract Undrained shear strength (s_u) of foundation soil of Marquette interchange near Milwaukee, Wisconsin was evaluated from the results of a number of pressuremeter tests conducted on normally consolidated (NC) organic silts and overconsolidated (OC) silty clay. The s_u-values were interpreted from traditional closed-form methods. The pressuremeter geometry and test sequence as well as response of the soil profiles were also simulated using axisymmetric finite element (FE) method with Cam-Clay soil model. The Cam-Clay model parameters were estimated from laboratory tests on undisturbed soil samples. Results show that the s_u estimated from the rate of cavity pressure change with volumetric strain (referred to as direct traditional method) is almost twice the s_u estimated from an indirect traditional method that estimates s_u from shear modulus, in situ horizontal stress, and ultimate cavity pressure obtained from the cavity pressure curves. The s_u-values predicted from the FE models are lower than those estimated from the traditional methods and shows that the assumption of infinite pressuremeter length in traditional methods results in overprediction of undrained shear strength by a factor of 1.5 for NC clay and 2.2 for OC clay. The results of finite element analysis considering Cam-Clay soil model and finite length for pressuremeters suggest the undrained shear strength of 63 ± 7 kPa for NC organic silt and 259 ± 68 kPa for OC silty clay.

Keywords Pressuremeter test · Undrained shear strength · Finite element · Cam-Clay · Foundation

A. Soleimanbeigi (✉)
Department of Civil and Environmental Engineering, University of Wisconsin-Madison, 2243 Engineering Hall, 1415 Engineering Drive, Madison, WI 53706-1691, USA
e-mail: ali.soleiman@gmail.com

1 Introduction

Over the past 20 years there has been increasing reliance on in situ testing rather than laboratory testing due to increasing availability of more reliable in situ test interpretation methods that make in situ testing less time-consuming and more economical than laboratory testing (Salgado 2008). Different available in situ tests based on their perceived applicability, different ground conditions and relevant geotechnical information obtained from each test were categorized and based on the ratings assigned to each in situ test method, the pre-bored pressuremeter test is the only method applicable to most ground conditions varying from soft soil to hard rock (Robertson 1986). The pressuremeter device is used to estimate shear strength, deformation characteristics, in situ horizontal stress and permeability of the soil (Ménard 1956). However, similar to other in situ test procedures, soil properties estimated from pressuremeter test data are affected by several sources of errors including (1) soil disturbance (collapse, erosion, or softening of borehole walls) associated with boring and equipment installation technique (2) equipment calibration associated with test procedure (e.g., volume change applied pressure), and (3) method of interpretation including models for soil response, initial and boundary conditions (Baquelin et al. 1978; Mair and Wood 1987). The traditional method used to interpret the undrained shear strength (s_u), models the soil as an elastic-perfectly plastic material in plain strain condition (i.e., infinite length for pressuremeter probe) to obtain a simplified closed-form solution. However, the interpreted s_u-values are often considerably larger than s_u-values obtained from other in situ or laboratory test results (Carter et al. 1979; Wroth 1982; Mair and Wood 1987; Yu 1990; Houlsby and Carter 1993; Yu et al. 2005). No studies have been conducted to specify the ratio of the s_u-values interpreted from closed-form solutions to the s_u-values from a method which takes into account the finite pressuremeter length and more precise soil models in the light of field measurements.

In this paper undrained shear strength of foundation soil in Marquette Interchange located near Milwaukee, Wisconsin was evaluated from the results of a number of pressuremeter tests. The s_u-values were estimated using the traditional closed-form solution. The soil profile, pressuremeter test sequence, and geometry conditions were also simulated using axisymmetric finite element (FE) method and Cam-Clay soil model to evaluate the effect of geometry condition and the soil model on the predicted undrained shear strength. The undrained shear strengths interpreted from the traditional method were compared to those estimated from the FE method.

2 Background

The basis of the pressuremeter test is radial expansion of a long cylindrical membrane installed in a borehole as illustrated in Fig. 1a. The pressure is applied inside a cylindrical membrane and volume change of the expanding membrane is measured. The in situ properties of the soil are then interpreted from the pressure versus volumetric or radial strain curve (Fig. 1b). Cavity expansion problem in an elastic-perfectly plastic medium was solved to obtain a relationship between the cavity pressure and volumetric strain for undrained clay (Gibson and Anderson 1961). As the cavity pressure increases, an expanding annulus of plastic soil develops around the cavity. During this phase the cavity pressure is obtained using the following relationship:

$$p_{ca} = \sigma_{h0} + s_u[1 + \ln(G/s_u)] + s_u[\ln(\Delta V/V)] \qquad (1)$$

where p_{ca} is the applied pressure by the pressuremeter; σ_{h0} is the in situ horizontal stress; G is the shear modulus, and $\Delta V/V$ is the volumetric strain. The limiting conditions at which all the surrounding soil deforms plastically is assumed to be reached when $\Delta V/V = 1$ (Gibson and Anderson 1961), therefore the third expression in Eq. (1) is omitted and the limiting cavity pressure (p_L) is obtained by:

$$p_L = \sigma_{ho} + s_u[1 + \ln(G/s_u)] \qquad (2)$$

By substitution of Eq. (2) into Eq. (1), we obtain:

$$p_{ca} = p_L + s_u \ln(\Delta V/V) \qquad (3)$$

From Eq. (3), if p_{ca} is plotted versus $\ln(\Delta V/V)$, the plastic phase of the curve lies on a straight line with gradient s_u as shown in Fig. 1c. In this paper, this approach of estimating s_u proposed by Gibson and Anderson (1961), is referred to as direct traditional method. The limiting pressure p_L is intercepted as p_{ca} corresponding to $\ln(\Delta V/V) = 0$ as illustrated in Fig. 1c.

Alternatively, s_u can be obtained from Eq. (2). Rewriting Eq. (2) for s_u gives:

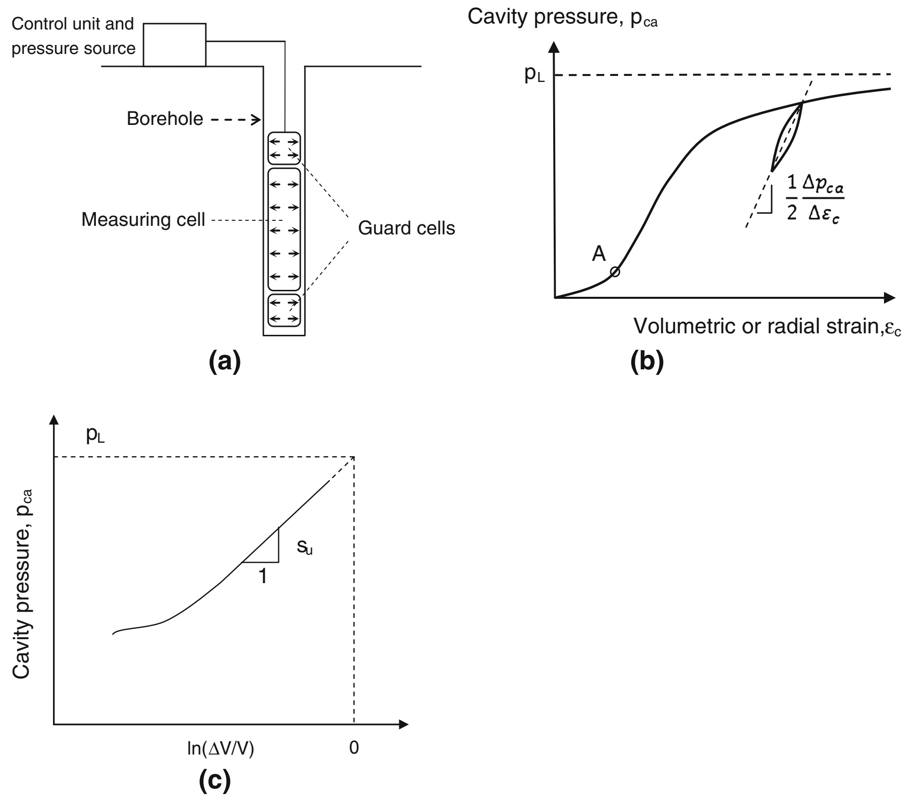
$$s_u = \frac{p_L - \sigma_{ho}}{1 + ln\left(\frac{G}{s_u}\right)} \qquad (4)$$

The s_u-value from Eq. (4) can be obtained from trial and error after estimating values of p_L, G, and σ_{ho}. This alternative method is referred to as indirect traditional method. The p_L-value can be estimated from the y-intercept of horizontal asymptote of p_{ca} in the cavity pressure curve (Fig. 1b) or from the $p_{ca} - \ln(\Delta V/V)$ curve (Fig. 2b). The G-value is obtained from the unload–reload loop in the cavity pressure curve using the following relationship (Palmer 1972; Ladayani 1973):

$$G = \frac{1}{2}\frac{dp_{ca}}{d\varepsilon_c} \qquad (5)$$

where ε_c is the cavity (radial) strain and is obtained from:

Fig. 1 Schematic of pressuremeter test device (**a**), typical cavity pressure curve (**b**), and cavity pressure versus ln(ΔV/V) to estimate undrained shear strength (**c**)



$$\varepsilon_c = (1 - \Delta V/V)^{-1/2} - 1 \tag{6}$$

The G-value is therefore half of the slope of unloading–reloading loop in the p_{ca}–ε_c curve. The σ_{h0} can be estimated from the point on the p_{ca}–ε_c curves where the elastic response of the soil starts after the initial flat part (point A in Fig. 1b). The ratio $\frac{G}{s_u} = I_r$ is termed the rigidity index of the soil. The p_{ca}–ε_c curves were also predicted using finite element method to back-calculate parameters of Cam-Clay soil model. Finite element models will be discussed later in this paper.

3 Pressuremeter Test Reslts

The Menard type pressure-controlled pressuremeter tests were performed in six boreholes in Maquette Interchange site near Milwaukee, Wisconsin. Three boreholes, PB-1, PB-2 and PB-3, were located in normally consolidated (NC) organic clay soil and the other three boreholes, PB-4, PB-5, and PB-6 were located in over-consolidated (OC) silty clay soil. Figure 2a, b show sample soil profile, soil physical properties, depths of water table, and the pressuremeter test spots in PB-3 and PB-6 respectively. The soil profiles at other borehole locations are shown in Fig. 12 in the "Appendix" section. The soil description and depth of the pressuremeter tests at each borehole are summarized in Table 1. Corrections for pressure and volume were applied to the pressuremeter test results (Roscoe and Schofield 1963). Radial cavity strains (ε_c) were calculated from the measured volumetric change of each pressuremeter using Eq. (6). Figure 3 shows the p_{ca}–ε_c curves from the pressuremeter tests in boreholes PB-3 and PB-6. The cavity pressure curves for the rest of the pressuremeter tests are shown in Fig. 13 in the "Appendix" section. The initial flat portion of each curve indicates the soil disturbance during borehole preparation and pressuremeter installation. Therefore, unload-reload cycles are performed to remove the effect of pressuremeter

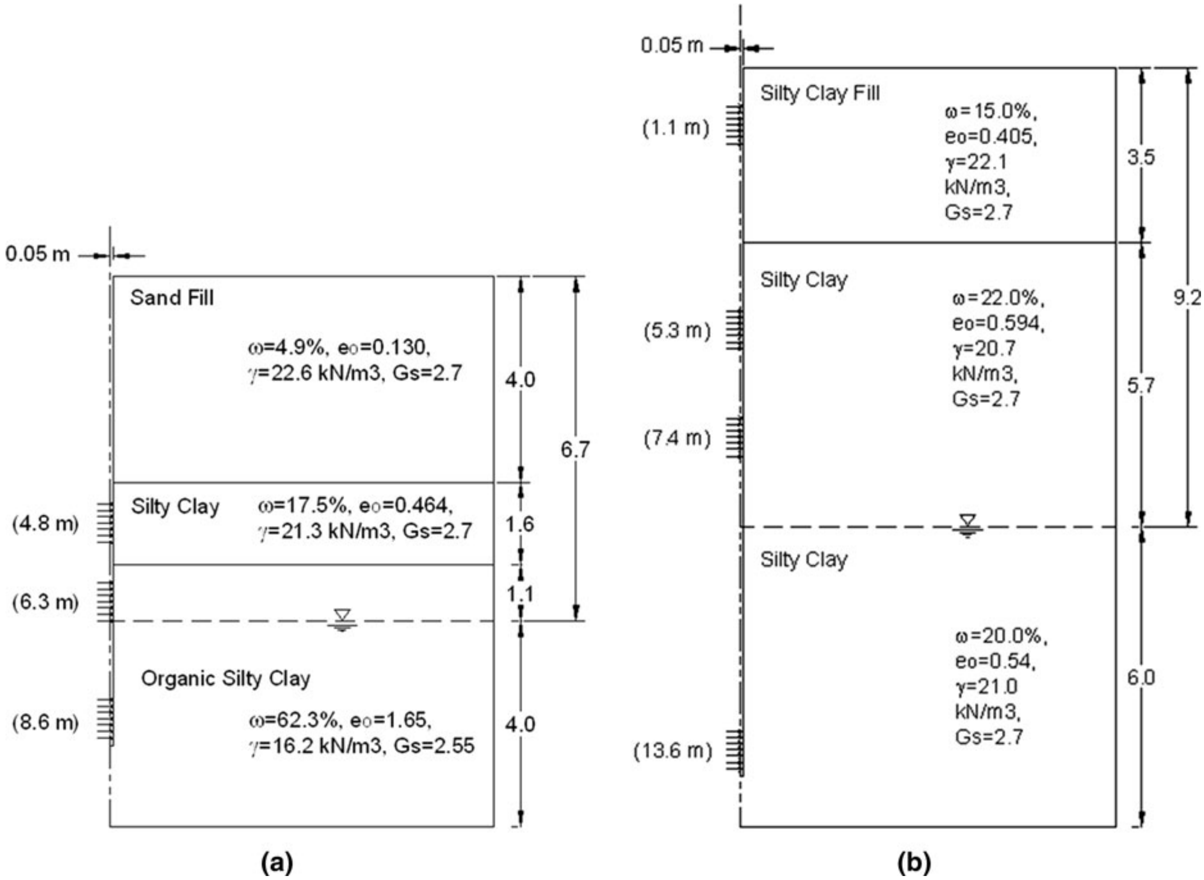


Fig. 2 Boring profiles for pressuremeter tests: **a** PB-3, and **b** PB-6. (ω = in situ water content, e_o = in situ void ratio, G_s = specific gravity, γ = in situ unit weight of soil)

installation disturbance and measure the undrained shear modulus.

To estimate s_u of NC organic silts and OC silty clays using the direct traditional method (Gibson and Anderson 1961), the $p_{ca} - \ln(\Delta V/V)$ curves were generated from cavity pressure curves and plotted in Fig. 4. The unload–reload cycles were omitted from the curves. Initial flat part of the curves reflects the soil disturbance during boring and pressuremeter installation in the boreholes and does not affect the slope of the straight portion of the curves (s_u). The estimated s_u-values obtained from the slope of the best fitting-line over the data points after the initial flat part of the $p_{ca} - \ln(\Delta V/V)$ curves are summarized in column 8 of Table 1. The s_u-value for the NC organic silt is 184 ± 106 kPa and for the OC silty clay is 1,164 ± 406 kPa.

Alternatively, the s_u-values of NC organic silts and OC silty clays were obtained by indirect traditional method using Eq. (4) by trial and error. The best estimate of σ_{h0}, G, and p_L, obtained from the p_{ca}–ε_c curves were used to estimate the s_u-values from Eq. (4). The G-values were calculated from half of the slope of unload-reload loop in each p_{ca}–ε_c curve using Eq. (5). The approximate in situ horizontal stresses were estimated from the point on the p_{ca}–ε_c curves where the elastic response of the soil starts after the initial flat part. The p_L-values were estimated from the y-intercept of horizontal asymptote of p_{ca} in the p_{ca}–ε_c curves. The estimated σ_{h0}, G, and p_L at different pressuremeter test locations are summarized in columns 5–7 of Table 1.

The estimated values of s_u from the indirect traditional method are summarized in column 9 of

Table 1 Soil description and engineering properties from pressuremeter tests

Soil type (1)	Boring name (2)	Average depth (m) (3)	Description (4)	G (kPa) (5)	σ_{ho} (kPa) (6)	p_L (kPa) (7)	s_u (kPa)—direct method (8)	s_u (kPa)—indirect method (9)	λ (10)	κ (11)	e_o (12)	s_u (kPa)—FE method (13)
NC organic silt $\gamma = 20$ kN/m^3	PB1	3.3	OL-very loose	3,360	65	380	67	63	0.20	0.010	1.55	58
		4.8	OL-very loose, LL = 48, PI = 14	1,760	130	460	170	81	0.35	0.035	1.17	62
		6.0	OL-very loose	3,400	95	650	239	130	0.35	0.0175	1.17	71
		11.7	OL-very soft	5,800	205	590	163	71	0.35	0.070	1.29	76
	PB2	4.8	OL-very loose	2,100	95	490	168	97	0.35	0.0175	2.24	60
		6.2	SM-very loose	2,090	105	490	171	94	0.35	0.01	0.46	59
		10.3	OH-very soft, LL = 88, PI = 21	2,580	200	480	83	58	0.35	0.0175	2.86	56
	PB3	4.8	CL-medium stiff	6,120	150	850	452	148	0.03	0.003	0.46	70
		6.3	OH-soft	2,350	140	550	178	98	0.20	0.020	2.24	65
		8.6	OH-soft, LL = 56, PI = 18	2,140	165	600	150	109	0.15	0.025	1.65	56
	PB4	1.1	HF/ML-medium dense to dense	8,170	140	2,400	1,234	636	0.04	0.005	0.32	390
OC silty clay $\gamma = 21$ kN/m^3		3.3	CL-medium stiff to stiff	5,820	105	2,000	1,010	570	0.08	0.01	0.49	270
		5.1	CL-medium stiff to stiff	9,360	165	1,800	807	392	0.08	0.004	0.49	240
		8.5	CL-medium stiff to stiff	1,0340	225	3,050	1,310	791	0.08	0.004	0.49	330
	PB5	3.3	ML-medium dense	9,830	160	3,150	1,610	874	0.03	0.005	0.40	295
		4.8	CL-medium stiff	11,000	250	2,100	1,328	438	0.06	0.004	0.49	143
		6.6	ML-loose	9,000	190	2,600	1,182	670	0.07	0.0035	0.48	230
		12.1	CL-very stiff to hard	8,950	325	3,800	2,042	1,133	0.02	0.007	0.48	313
	PB6	1.1	HF/CL/ML-stiff	6,950	70	1,400	538	328	0.03	0.002	0.41	173
		5.3	CL–ML-soft to medium stiff	1,9850	190	2,400	923	465	0.04	0.003	0.59	249
		7.4	CL–ML-soft to medium stiff	1,1680	215	2,200	739	472	0.12	0.007	0.59	276
		13.6	CL–ML-soft to medium stiff	9,750	280	2,900	1,243	729	0.12	0.01	0.54	258

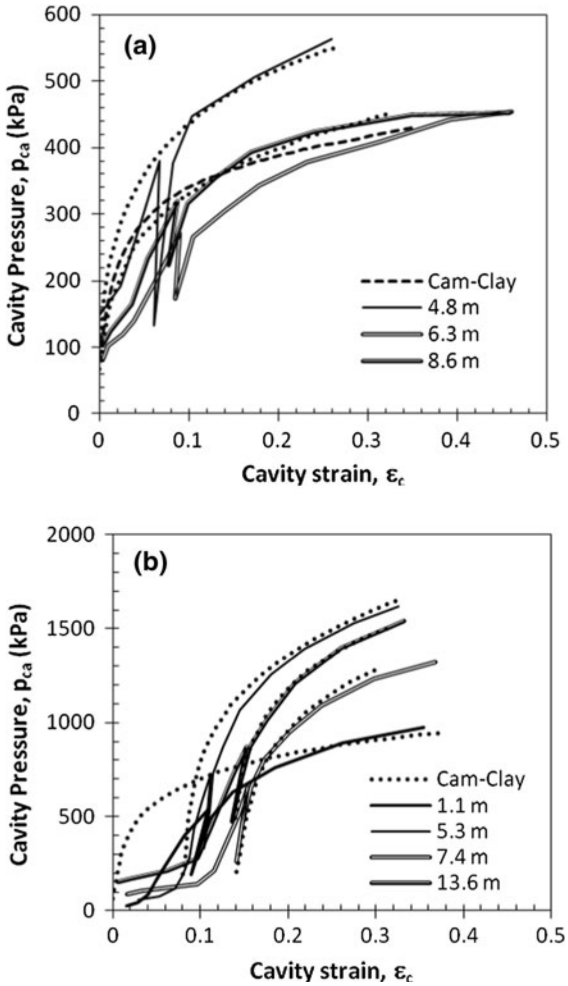


Fig. 3 Measure and predicted cavity pressure *curves* from pressuremeter tests: borehole PB-3 (**a**) and borehole PB-6 (**b**)

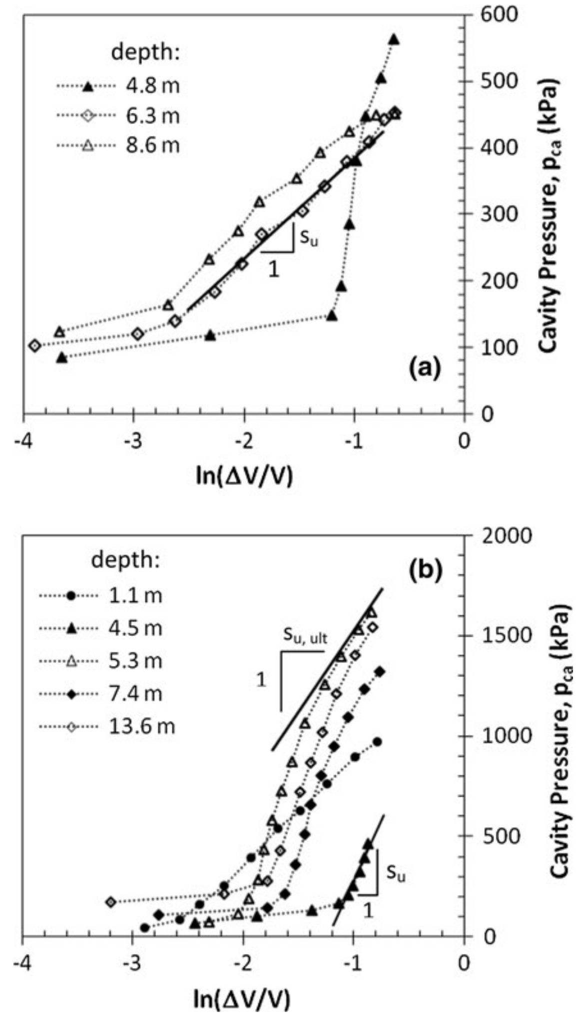


Fig. 4 Cavity pressure versus natural log of volume increment ratio for pressuremeter tests: borehole PB-3 (**a**), and borehole PB-6 (**b**)

Table 1. The interpreted s_u-value for the NC organic silt is 95 ± 29 kPa and for the OC silty clay is 579 ± 176 kPa.The average s_u-values from the indirect traditional method are lower than those estimated from the direct traditional method. Additionally, the indirect traditional method results in lower standard deviation about the average s_u. Figure 5 compares the s_u-values obtained from the two methods. The data points show a linear trend with coefficient of determination (R^2) of 0.95. The s_u-values from the indirect traditional method are approximately half of the s_u-values obtained from the direct traditional method. The s_u-value from the direct traditional method determined in this case study represents the slope of the best fitting-line over the data points after the initial flat part in $p_{ca} - \ln(\Delta V/V)$ plane (Fig. 4b). This method is commonly followed in design practice. However, the slope of the majority of the $p_{ca} - \ln(\Delta V/V)$ curves decreases with increasing the cavity strain which shows lower shear strength at larger deformations. Less uncertainty surrounds the determination of the apparent large strain shear strength [or ultimate strength, $s_{u,ult}$ in Fig. 4b given by the slope of the $p_{ca} - \ln(\Delta V/V)$ curve at larger deformations as the influence of possible disturbance during pressuremeter installation and uncertainty concerning the true reference condition is smaller (Mair and Wood 1987). The average s_u-values obtained from the slope of the

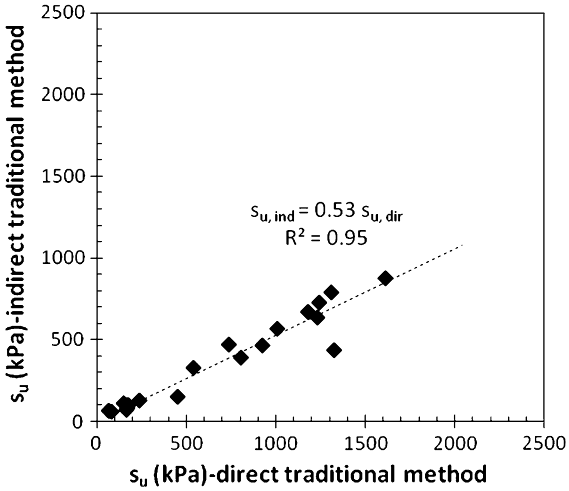


Fig. 5 Comparison of undrained shear strength between traditional methods

best fitting-line is 28 % higher than the average $s_{u,ult}$-values obtained in this study. Previous studies also reported that the interpreted s_u-values from the direct traditional method (Gibson and Anderson 1961) are often considerably larger than s_u-values obtained from other field and laboratory test results (Carter et al. 1979; Wroth 1982; Mair and Wood 1987; Yu 1990; Houlsby and Carter 1993; Yu et al. 2005). Mair and Wood (1987) recommended the s_u-values be determined from either the slope of the $p_{ca} - \ln(\Delta V/V)$ curves at large strain ($s_{u,ult}$) or from the indirect traditional method. With the incorporation of three additional soil properties to estimate the s_u, (i.e., shear modulus, in situ horizontal stress, and ultimate cavity pressure), the indirect traditional method is considered more reliable and therefore suggested for estimation of undrained shear strength from the pressuremeter test results.

4 Cam-Clay Soil Model

The elastic-perfectly plastic soil model used in traditional method does not take into account the effect of stress history on the soil behavior. Critical state models are capable of considering effects of stress history and therefore more appropriate for modeling behavior of overconsolidated soils (Roscoe and Schofield 1963; Roscoe and Burland 1968; Wood 1990; Yu 2000). The modified Cam-Clay model is a volumetric hardening–softening model based on the critical state concept and has been widely used to simulate behavior of NC and OC clay (Roscoe and Burland 1968; Wood 1990). Figure 6 shows the schematic of the modified Cam-Clay model and the model parameters. The yield surface of the model in

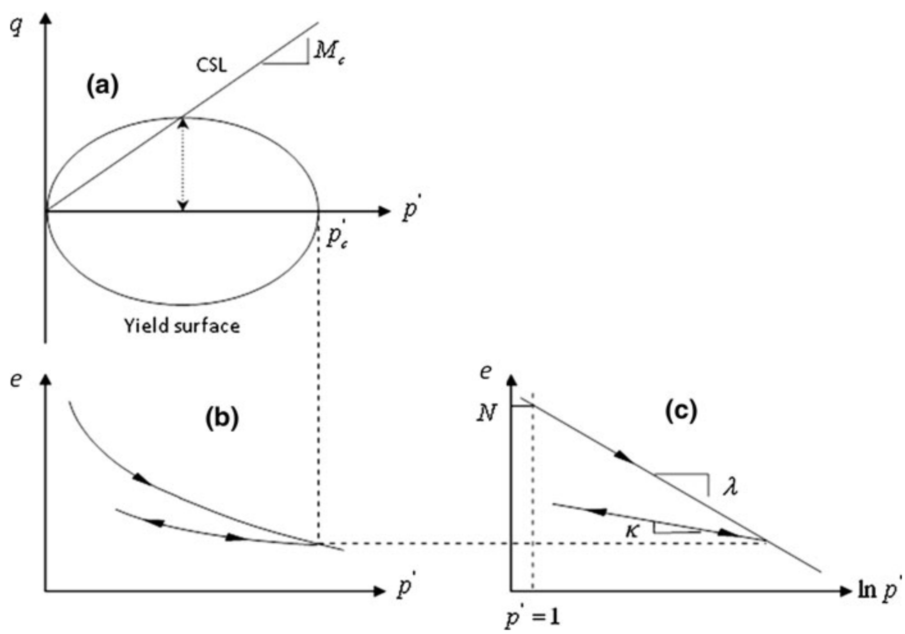


Fig. 6 Elliptical yield surface for Cam-Clay model in p′–q plane (**a**) normal compression line and unloading–reloading line in compression plane (**b**, **c**) (After Wood 1990)

Table 2 Range of Cam-Clay model parameters from laboratory tests

	NC organic clay	OC silty clay
λ	0.069–0.630	0.004–0.083
κ	0.003–0.074	0.0004–0.007
M_c	1.2	1.2
OCR	1–2	2–15
e_o	0.81–3.78	0.28–0.72

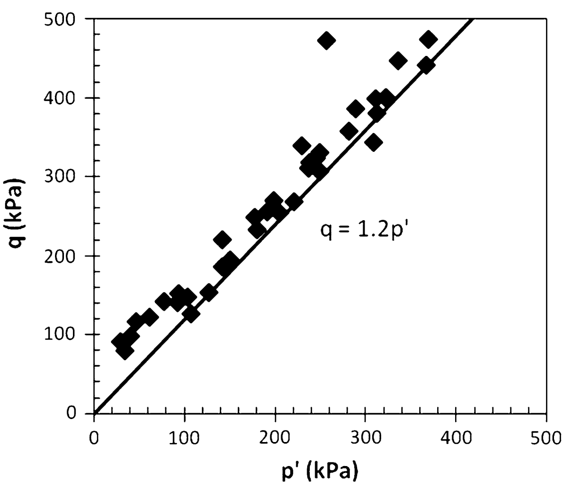


Fig. 7 Stress points at failure measured from triaxial compression tests

p′–q (deviator stress-mean effective stress) plane has an elliptical shape. After soil yielding, the yield surface expands to model stress hardening (increase of deviator stress after yielding) behavior of NC soil but shrinks to model stress softening (reduction of deviator stress after yielding) of OC soil. The basic model parameters include size of the elliptical yield surface, p'_c (i.e., the effective overconsolidation pressure); slope of the virgin compression line, λ; slope of the swelling line, κ; and the slope of the critical state line (CSL), M_c, in p′–q plane. Poisson's ratio (μ) and shear modulus (G) are used to predict elastic response of soil in the modified Cam-Clay model. Details of the model is described in (Wood 1990).

The model parameters were obtained from consolidation and triaxial compression tests on undisturbed

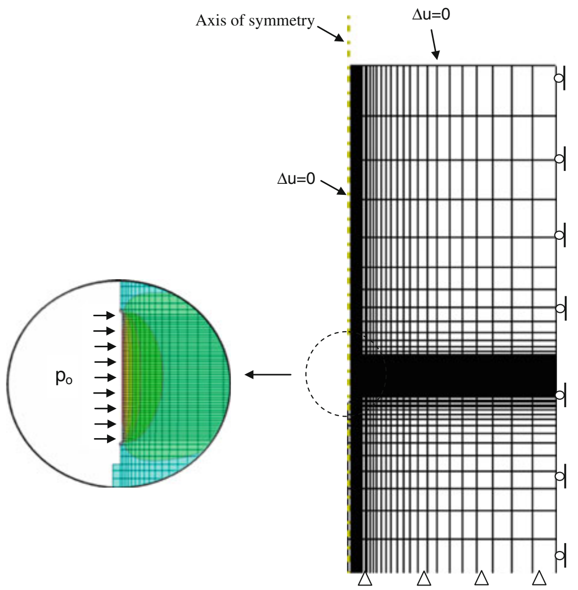


Fig. 8 Typical finite element model of borehole PB-6 for each pressuremeter test

NC organic silt and OC silty clay specimens taken from the foundation of the Marquette Interchange site. One-dimensional consolidation tests following ASTM 2435 were conducted to obtain λ and κ. The initial void ratios (e_o) were obtained following ASTM D 4959. Table 2 summarizes the range of λ and κ for NC organic silts and OC silty clays. The critical state friction angle (ϕ_c) of the soil was estimated from consolidated undrained (CU) triaxial compression tests on undisturbed soil samples with pore-water pressure measurement following ASTM D 4767. The deviator stress versus mean effective stress values were plotted in Fig. 7. The slope of the best-fitting line over the p′–q points is $M_c = 1.2$. The ϕ'_c of both NC organic silts and OC silty clays is 31° as obtained from:

$$M_c = 6sin\phi'_c/(3 - sin\phi'_c) \tag{7}$$

The initial void ratios were determined from measurement of in situ water content, specific gravity and total unit weight at each borehole and summarized in Table 2. The modified Cam-Clay model parameters were further adjusted to match the predicted cavity pressure curves to those measured form pressuremeter test data.

5 Development of Finite Element Models

Axisymmetric finite element (FE) analyses were conducted using ABAQUS software (ABAQUS^{TM} 2008) to model the boreholes and pressuremeter tests in NC organic silts and OC silty clays. Figure 8 shows a sample FE model. Only one half of each borehole section was modeled due to symmetry. The radius of each borehole is 0.05 m. The length of the expanding pressuremeter probe is 0.75 m. The width of each FE model is 3.0 m after trials to minimize the end effects on the stresses induced by the probe pressure. The height of each model is equal to depth of each pressuremeter test conducted in the borehole plus an appropriate extra distance to minimize end effects on the induced stresses. The initial conditions are in situ vertical and horizontal stresses. To estimate the in situ horizontal stress, the coefficients of lateral earth pressure at rest (K_0) for NC organic silt and OC silty clay at each borehole were obtained from the following equation (Mayne and Kulhawy 1982):

$$K_0 = (1 - \sin\phi')OCR^{\sin\phi'} \quad (8)$$

Boundary conditions applied to each FE model include: (1) zero rotation about vertical axis and about the vector normal to the plane on the axis of symmetry; zero horizontal displacement on the axis of symmetry and on the right side of the model; zero horizontal and vertical displacement on the bottom side; (2) no displacement constraints along wall of the borehole; (3) zero pore water pressure on the ground surface and on the borehole wall except the depth interval over which the pressuremeter test is conducted, and (4) the time-dependent pressure of each pressuremeter test as illustrated in Fig. 9. The probe pressures at each step were normalized with respect to the maximum applied pressure during the test. The pressures were corrected to account for the volume change and pressure loss within the pressuremeter system (Ménard 1956; Baquelin et al. 1978; Mair and Wood 1987). The duration time for each pressure level is 60 s after which the volume change of the probe is recorded. The duration time to increase the pressure from a previous level is 10 s. The type of the finite element is 8-noded quadrilateral axisymmetric with pore pressure. The element size was refined near the pressuremeter test location and becomes coarser with increasing distance. The size of the elements along the test intervals was adjusted based on the convergence of the numerical predictions.

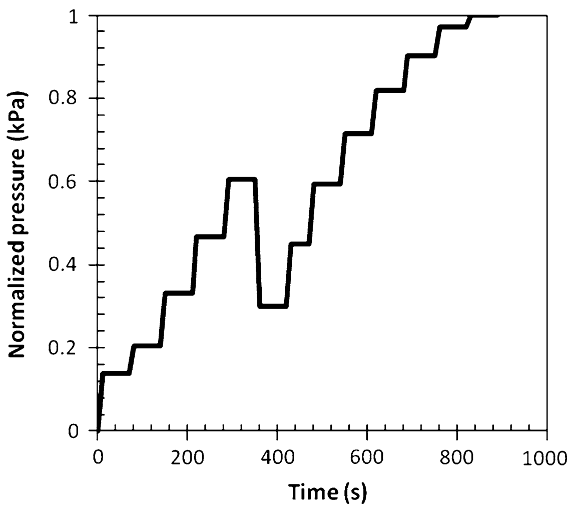


Fig. 9 Typical time-dependent pressure boundary applied to soil by pressuremeter

The Cam-Clay model parameters measured from the laboratory test results (Table 2) were assigned to soil profile in each FE model. For each FE model, the time-dependent pressure of each pressuremeter test was applied to the cavity wall at the test location and the cavity strains during the test were predicted.

6 Finite Element Analysis Results

The predicted cavity pressure curves from the FE models were compared to those measured from pressuremeter tests. The Cam-Clay model parameters were slightly adjusted to calibrate the FE models such that for each pressuremeter test, the predicted cavity pressure curves match the measured ones. Columns

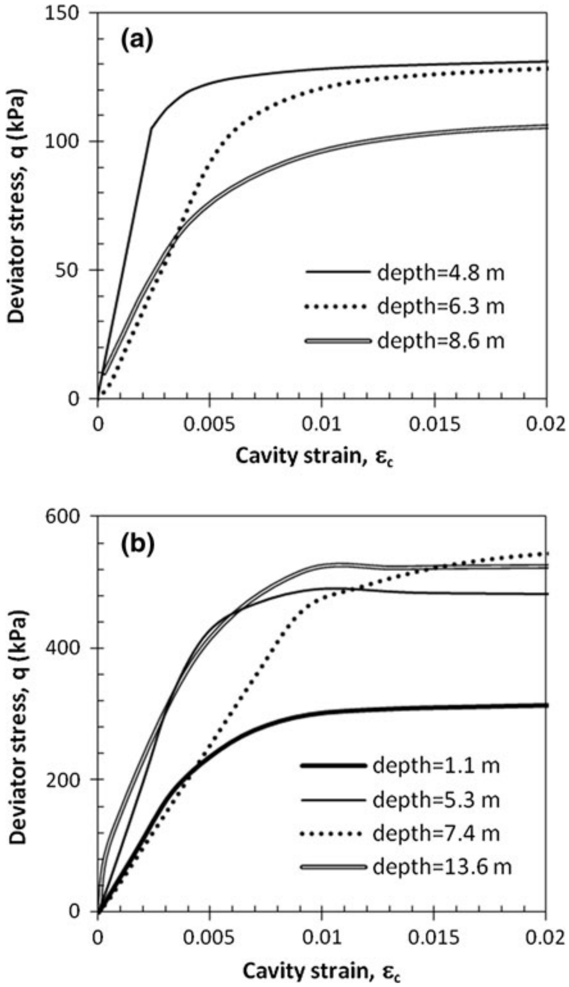


Fig. 10 Deviator stress versus cavity strain at the cavity wall in borehole PB-3 (**a**) and PB-6 (**b**)

10–12 in Table 2 contain λ, κ and e_o, assigned to soil profile in FE models. In the cavity expansion problem, the radial stress (σ_r) and circumferential stress (σ_θ) at the cavity wall are the maximum and minimum principal stresses respectively. Values of σ_r and σ_θ during each simulated pressuremeter test were predicted from the FE analyses. The deviator stress is defined as:

$$q = \sigma_r - \sigma_\theta \tag{9}$$

The deviator stress versus cavity strain (q–ε_c) curves during each test were plotted and the s_u-values were obtained from:

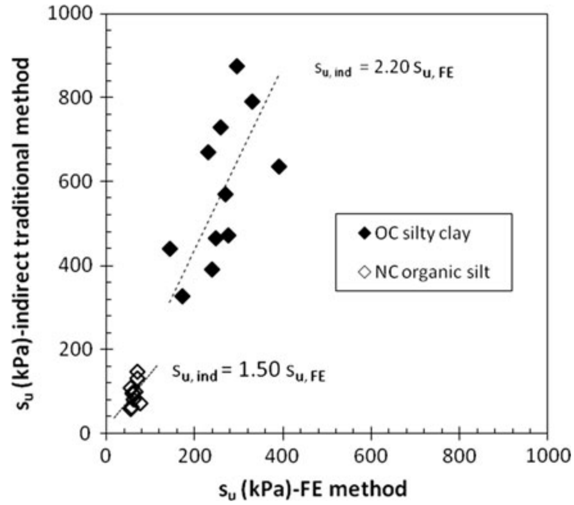


Fig. 11 Comparison between traditional method and FE method

$$s_u = \frac{q_f}{2} \tag{10}$$

where q_f is the maximum deviator stress determined from the q–ε_c curves. The predicted cavity pressure curves for the pressuremeter tests conducted in boreholes PB-3 and PB-6 are shown in Fig. 3a, b by the dotted curves. The predicted cavity pressure curves approximately match the measured ones. Similar cavity pressure curves were predicted for the remainder of the pressuremeter tests (PB-1, PB-2, PB-4, PB-5, and PB-5) as shown in Fig. 13 in the "Appendix" section. The deviator stress at failure and thus s_u-values were determined from the deviator stress-cavity strain (q–ε_c) curves as shown in Fig. 10. The predicted s_u-values for NC organic silt and OC silty clays from the pressuremeter tests in six boreholes are summarized in column 13 of Table 1. The predicted s_u-value for the NC organic silt is 63 ± 7 kPa and for the OC silty clay is 259 ± 68 kPa.

The s_u-values predicted from FE models are smaller than those interpreted from the traditional methods. Figure 11 compares the s_u-values for NC organic silt and OC silty clay interpreted from indirect traditional method to those predicted from the FE method. The s_u-values of NC organic silt interpreted from the traditional method is 50 % higher than those predicted using FE method. Similar results were reported by Houlsby and Carter (1993). They performed finite element analysis of pressuremeter tests with finite length in the soil following elastic-perfectly plastic Tresca criterion and concluded that the assumption of infinite pressuremeter length results in overestimation of the s_u by a factor of approximately 1.25–1.43 for typical clays.

The overprediction of s_u-values interpreted from the traditional method is higher for OC silty clay. Figure 11 shows that the overprediction of s_u-values for OC silty clay due to assumption of infinite length for the pressuremeter is about 70 % higher than that for NC organic silt. The results obtained from analysis of the pressuremeter test results in this case study agree with the previous understanding that the interpreted shear strength using conventional elastic-perfectly plastic methods is higher than the shear strength measured from other in situ or lab test results due to simplifying assumption of infinite pressuremeter length in traditional method (Mair and Wood 1987; Wroth 1982; Yu 1990; Houlsby and Carter 1993; Yu et al. 2005; Yeung and Carter 1990; Shuttle and Jefferies 1995).

The ratio of s_u interpreted from the traditional direct method to s_u predicted from FE method is 2.2 for NC organic silt and 3.7 for OC silty clay. The ratio from the traditional indirect method is 1.5 for NC organic silt and 2.2 for OC silty clay. The results illustrate the significance of pressuremeter length and soil behavior in prediction of undrained shear strength of clay from pressuremeter test data. The results obtained herein are specific to the experimental data in this case study. To obtain more robust reduction factors and to generalize the findings in this study, a comprehensive investigation which involves a larger data base is suggested.

7 Conclusions

In this study, data from pressuremeter tests conducted on normally consolidated organic silt and overconsolidated silty clay were analyzed to estimate the undrained shear strength of the foundation soil in Marquette Interchange site. Traditional closed-form solution was used to interpret the undrained shear strength from the test results. The pressuremeter test procedure and soil profiles were also simulated using axisymmetric finite element method with Cam-Clay model for soil response. The following findings were concluded from the results of this study:

1. The undrained shear strength interpreted from the slope of cavity pressure versus logarithm of volumetric strain rate (referred to as direct traditional method), is almost twice the undrained shear strength interpreted from the indirect traditional method, where in situ horizontal stress, shear modulus and ultimate cavity pressure measured from the cavity pressure curve are taken into account.
2. The s_u-value interpreted from the direct traditional method is 184 ± 106 kPa for NC organic silt and 1,164 ± 406 kPa for the OC silty clay. The s_u-value interpreted from the indirect traditional method is 95 ± 29 kPa for NC organic silt and 579 ± 176 kPa for the OC silty clay.
3. The undrained shear strength predicted from finite element method is lower than that from the traditional methods and shows that the assumption of infinite pressuremeter length in traditional methods results in overprediction of undrained shear strength by a factor of 1.5 for normally consolidated clay and 2.2 for overconsolidated clay.

The results obtained herein are specific to this case study. More comprehensive study is recommended to obtain a range of overprediction for different types of soils.

Appendix

See Figs. 12 and 13.

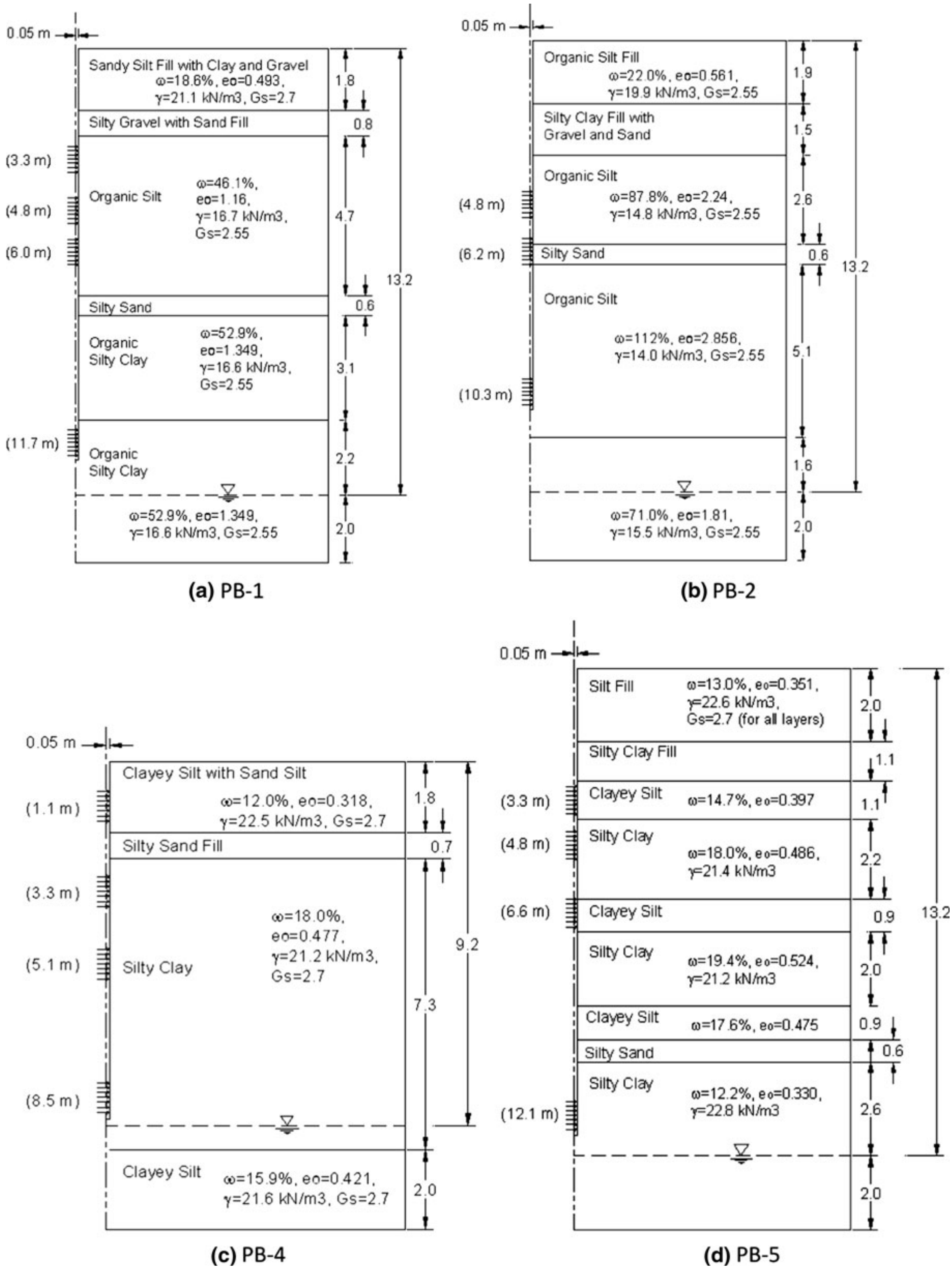


Fig. 12 Boring profiles for pressuremeter tests: PB-1 (**a**), PB-2 (**b**), PB-4 (**c**), and PB-5 (**d**)

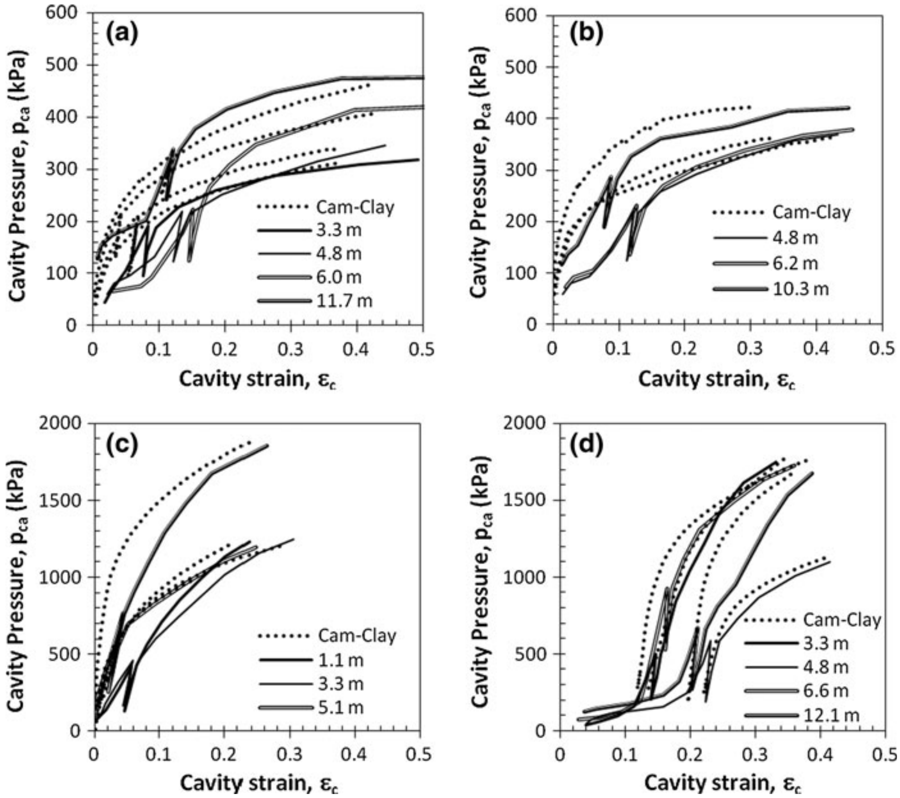


Fig. 13 Measure and predicted cavity pressure curves from pressuremeter tests: borehole PB-1 (**a**), PB-2 (**b**), PB-4 (**c**), and PB-5 (**d**)

References

ABAQUS Theory manual, version 6.8 (2008) Hibbit, Karlsson and Sorensson, Inc

Baquelin F, Jézéquel JF, Shields DH (1978) The pressuremeter and foundation engineering. Trans Tech Publications, Clausthual

Carter JP, Randolph MF, Wroth CP (1979) Stress and pore pressure changes in clay during and after the expansion of a cylindrical cavity. Int J Numer Anal Meth Geomech 3:305–322

Gibson RE, Anderson WF (1961) In situ measurement of soil properties with the pressuremeter. Civil Eng Public Works Rev 56(658):615–618

Houlsby GT, Carter JP (1993) The effects of pressuremeter geometry on the results of tests in clay. Géotechnique 43(4):567–576

Ladayani B (1973) Expansion of a cavity in a saturated clay medium. J Soil Mech Found Eng Div ASCE 89(SM4):127–161

Mair RJ, Wood DM (1987) Pressuremeter testing: methods and interpretation. CIRIA Ground Engineering Report: In-Situ Testing. Butterworths, London

Mayne PW, Kulhawy FH (1982) Ko-OCR relationships in soils. J Geotech Eng ASCE 108(6):851–872

Ménard L (1956) An apparatus for measuring the strength of soils in place. Master of Science Thesis, University of Illinois

Palmer AC (1972) Undrained plane-strain expansion of a cylindrical cavity in clay: a simple interpretation of the pressuremeter test. Geotechnique 22(3):451–457

Robertson PK (1986) In-situ testing and its application to foundation engineering. Can Geotech J 23(4):573–584

Roscoe KH, Burland JB (1968) On the generalised stress-strain behavior of wet clay. In: Heyman J, Leckie FA (eds) Engineering plasticity. Cambridge University Press, Cambridge, pp 535–609

Roscoe KH, Schofield AN (1963) Mechanical behavior of an idealized "wet clay". In: Proceedings of European conference on soil mechanics and foundation engineering. Wiesbaden, pp 47–54

Salgado R (2008) The engineering of foundations. McGraw-Hill, New York

Shuttle DA, Jefferies MG (1995) A practical geometry correction for interpreting pressuremeter tests in clay. Géotechnique 45(3):549–553

Wood DM (1990) Soil behavior and critical state soil mechanics. Cambridge University Press, Cambridge

Wroth CP (1982) British experience with the self boring pressuremeter. In: Proceedings of symposium on the pressuremeter and its marine applications, pp 143–164

Yeung SK, Carter JP (1990). Interpretation of the pressuremeter test in clay allowing for membrane end effects and material non-homogeneity. In: Proceedings of 3rd international symposium on pressuremeters, Oxford, pp 199–208

Yu HS (1990) Cavity expansion theory and its application to the analysis of pressuremeters. PhD Thesis, University of Oxford

Yu HS (2000) Cavity expansion methods in geomechanics. Kluwer Academic Publishers, London

Yu HS, Charles MT, Khong CD (2005) Analysis of presuremeter geometry effects in clay using critical state models. Int J Numer Anal Meth Geomech 29:845–859