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A MORE FUNDAMENTAL APPROACH TO PREDICT PORE PRESSURE FOR SOFT CLAY

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ABSTRACT: Skempton's (1954) pore pressure coefficient A provides a pragmatic attempt at determining pore pressures during undrained shear, and to use these in settlement computations and stability analysis of embankments in soft clays. Also, the Critical state concept offers a means of acquiring the undrained stress path in normally consolidated clays through using a volumetric yield locus derived from a simple energy balance equation. However, to date there is no novel method by which the undrained stress paths of lightly over-consolidated and heavily overconsolidated clays can be predicted by using fundamental concepts. Based on the work of Handali (1986), Balasubramaniam et al. (1989) presented an alternative pore pressure coefficient that was more generalised than the Skempton's coefficient. However, Pender (1978) proposed a set of parabolas to describe the undrained stress paths of overconsolidated clays, and Lee (1995) considered elliptic paths to be more in agreement with the experimental observations. In this paper, observed and predicted undrained stress paths both under compression and extension, and also from isotropic and K0 pre-shear consolidation states will be presented. Such expressions can then be readily used in computer softwares for stability analysis and settlement computations.

Keywords: Soft clay, undrained stress path, pore pressure coefficient

INTRODUCTION

The classical work of Skempton (1954) and Henkel (1959) on the pore pressure coefficient is still widely used in settlement computations by the Skempton (1964) and Bjerrum (1972) method, and in the simple stability analysis of embankments by effective stress analysis. There are of great practical significance, as excess pore pressure in saturated clays can be derived from a single parameter A (in the axi-symmetric case) relating to the deviator stress increment. However, the parameter A is heavily dependent on the stress history of the clay (whether normally or overconsolidated) and also at the level of the deviator stress. Lo (1976) observed a unique relationship between pore pressure and shear strain at different stages of loading and unloading and emphasised that proper estimation of pore pressure in an undrained loading cannot be carried out without taking shear strain into consideration. The presentation of test data and analysis in this paper will follow the work of Lee (1995).

PROPERTIES OF CLAY SAMPLES

The subsoil in the Bangkok Plain consists of quaternary deposits that originated from sedimentation at the delta of the ancient Chao Phraya River. The Chao Phraya Plain consists of a broad basin filled with sedimentary soil deposits that form alternate layers of clay, sand and gravel. The thickness of the soft to medium stiff clay in the upper layer varies from 12 to 20 m, while the total clay layer (including the first stiff clay layer) is about 15 to 30 m. The engineering properties of the soft clay and other layers were studied in great detail at AIT, and the triaxial behaviour of the undisturbed samples of weathered clay and the soft clay were presented in Balasubramaniam and Uddin (1977), and Balasubramaniam and Chaudhry (1978).

The clay samples used in this study were from Nong Ngoo Hao site at a depth between 5.0m to 6.0 m. The samples were taken using 250 mm thin walled tubes with 0.5 m long. These tubes were sufficiently large to reduce sample disturbance. The samples fall into category of very soft clay. The soil was fairly homogeneous but did

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include very thin horizontal silt layers. Table 1 summarizes the basic properties of the tested clay sample. Table 2 presents the summary of triaxial tests. The samples under strain controlled and static loading tests are marked with symbols "ST". Samples IS-1 and IS−2 were isotropically consolidated. Typical consolidation process of the sample is given in Fig. 1.

Table 1 Physical properties of base clay

Property	Values
Liquid limit (%)	116
Plastic limit (%)	46
Plasticity Index (%)	70
Water content (%)	124
Specific Gravity	2.75
Organic Matter (%)	3-4
pH	8
Grain Size Distribution	
Clay (%)	64
Silt (%)	27-32
Sand (%)	4-9

Table 2 Summary of tests

Test No.	Consolidation Stress		Consolidation Ratio, η_a	OCR
	q_a (kN/m^2)	p_i (kN/m^2)		
ST-1	10	21	0.49	1.5
ST-2	15	31	0.49	1.0
ST-3	23	47	0.49	1.0
ST-4	30	62	0.49	1.0
ST-5	0	79	0	1.0
ST-6	0	53	0	1.6
ST-7	0	44	0	2.0
ST-8	18	70	0.25	1.0
ST-9	40	53	0.75	1.0
ST-10	0	68	0	1.2
IS-1	0	10	0	1.0
IS-2	0	20	0	1.0

UNDRAINED STRESS PATHS

A simple equation for the undrained stress path of normally consolidated samples can be derived based on the linear relationship between pore pressure and the stress ratio. This formulation was started with the linear

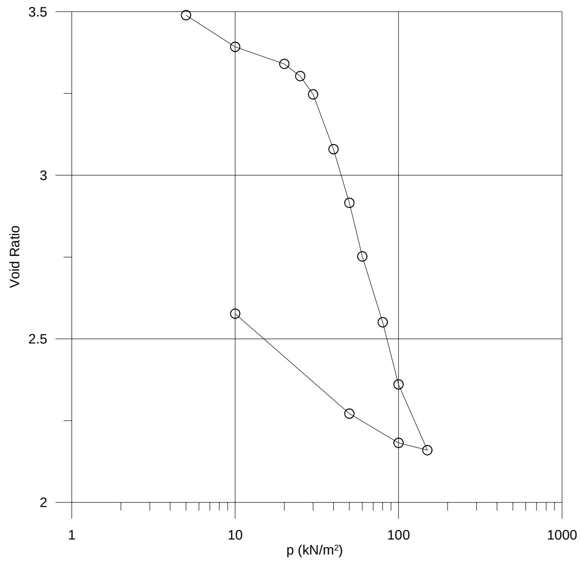

Fig. 1 Consolidation process of clay sample

pore pressure-stress ratio relationship for normally consolidated clays and the bi-linear relations noted for over consolidated clays as interpreted by Handali (1986).

Figure 2a shows the variation of normalised pore pressure (u/p_0) with stress ratio (η) for the isotropically normally consolidated samples, where p_0 is the pre-shear consolidation stress. The variation is linear and the average gradient $(1/p_0)(du/d\eta)$ for Bangkok clay is 0.53. Fig. 2b shows the $(u/p_0, \eta)$ relationship for the anisotropically consolidated samples, which is bi-linear. The gradients $(1/p_0)(du/d\eta)$ for the initial part are 0.34, 0.31 and 0.25 when the anisotropic consolidation ratio η_a are 0.25, 0.49 and 0.75, respectively. The second linear part is parallel to that of the isotropically normally consolidated samples having a gradient $(1/p_0)(du/d\eta)$ of 0.52. In Fig. 3, an interpretation is given wherein the $(u/p_0, \eta)$ plots are shifted upwards in the direction of (u/p_0) by the values $(\Delta p_{0n}/p_0)$. An illustration to describe Δp_{0n} is given in Fig. 4 , where $\Delta p_{0n} = p_d - p_i$, where p_i is the pre-shear mean consolidation stress of the anisotropically consolidated sample; and p_d is the p value on the drained path of the isotropically consolidated to p_0 and drawn at the same level of q_a, the initial pre-shear q value of the anisotropically consolidated sample. Additionally, p_0 is the pre-shear consolidation stress of the sample sheared under undrained condition of the isotropic consolidated sample, having the same pre-shear voids ratio of the anisotropically consolidated sample. Thus,

$$p_{0n} = p_0 - p_i\left(1 - \frac{\eta_a}{3}\right) \qquad (1)$$

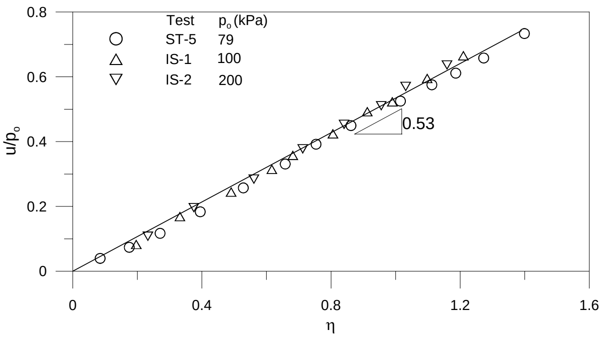

Fig. 2a $\left(u/p_0,\eta\right)$ plot for normally consolidated samples under monotonic loading

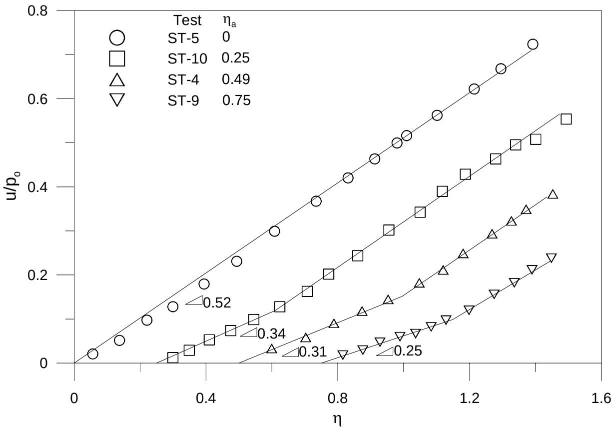

Fig. 2b $\left(u/p_0,\eta\right)$ relationship for anisotropically consolidated samples

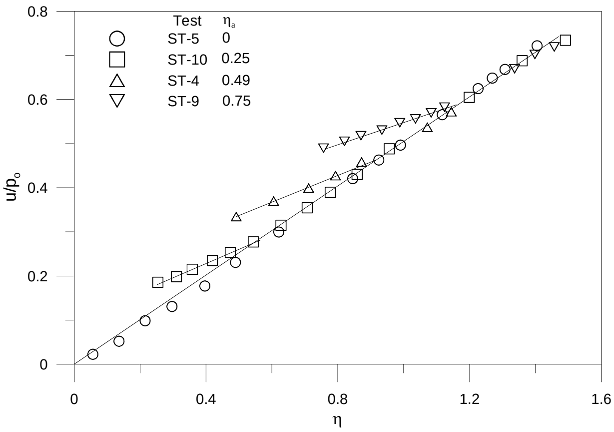

Fig. 3 Shifted $\left(u/p_0,\eta\right)$ paths for anisotropically consolidated samples

Figure 5 shows the $\left(u/p_0\,,\eta\right)$ plot for overconsolidated samples. Here again, the $\left(u/p_0\,,\eta\right)$ relationships are bi-linear. The measured gradients $\left(1/p_0\right)\left(du/d\eta\right)$ of the first linear section for samples having OCR values of 1.2, 1.6 and 2.0 (all the samples have the same void ratio of 2.55) are 0.35, 0.22 and 0.18 respectively. The gradient of the second linear section (0.52) is the same as that of the normally consolidated sample. In Fig. 6, the bi-linear relationships of the over-consolidated samples are shifted in the direction of $\left(u/p_0\right)$ by the corresponding value $\left(\Delta p_{0n}/p_0\right)$, where Δp_{0n} is defined as the difference between the pre shear consolidation stress of the normally consolidated sample and the corresponding overconsolidated sample. The adjustment made and shown in Fig. 7 is identical to the thinking that the point of reference for measuring pore pressure of the over-consolidated sample is the same as that of the normally consolidated sample. Such a shift results in the merging of the second linear path of the over-consolidated samples to the path of the normally consolidated sample as shown in Fig. 6. For normally consolidated clays the relationship between q, p and u can be given by:

$$u = p_0 + \frac{1}{3}q - p \tag{2}$$

$$\text{Also,}\ \ u = Cp_0\eta \tag{3}$$

Combining (2) and (3), it can be shown that

$$\frac{p}{p_0} = \frac{\left(1 - C\eta\right)}{\left(1 - \frac{\eta}{3}\right)} \tag{4}$$

Also, for the peak stress ratio condition coinciding with the critical state

$$C = \frac{1}{\left[\left(2 - \frac{M}{3}\right)M\right]} \tag{5}$$

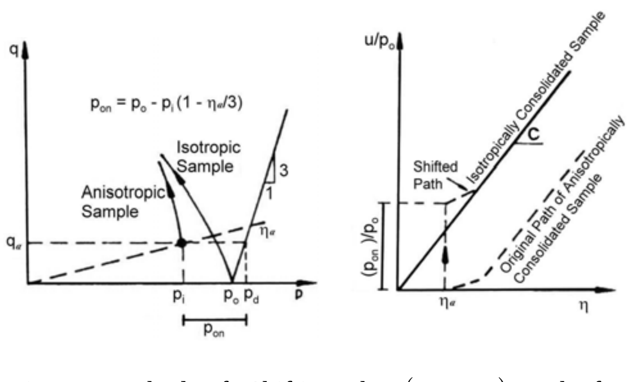

Fig. 4 Method of Shifting the $\left(u/p_0,\eta\right)$ path for anisotropically consolidated samples

Therefore the undrained stress path for normally consolidated samples can be expressed as

$$\frac{p}{p_0}=\left[\frac{1-\dfrac{1}{\left(2-\dfrac{M}{3}\right)M}\eta}{\left(1-\dfrac{\eta}{3}\right)}\right] \tag{6}$$

Kim (1991) also extended this relationship to over-consolidated clays following the work of Handali (1986) by introducing two parameters "C_1"and "C_2"for the normally consolidated and overconsolidated regions. These parameters indicate the slopes of the first and second linear sections of the $\left(u/p_0,\eta\right)$ relationships. The stress ratio at the intersection of these segments was defined as η_t. The pore pressure developed in these two segments is determined as,

$$\frac{u}{p_0}=C_1\eta \quad \text{for } \left(\eta\le\eta_t\right) \tag{7a}$$

and

$$\frac{u}{p_0}=\left(C_1-C_2\right)\eta_t+C_2\eta \quad \text{for } \left(\eta>\eta_t\right) \tag{7b}$$

where, p_0 is pre-shear mean normal stress. These equations are used for the generation of undrained stress paths

$$\frac{p}{p_0}=\frac{1-C_1\eta}{1-\dfrac{\eta}{3}} \quad \text{for } \left(\eta\le\eta_t\right) \tag{8a}$$

and $$\frac{p}{p_0}=\frac{1+\left(C_2-C_1\right)\eta_t-C_2\eta}{1-\dfrac{\eta}{3}} \tag{8b}$$

for $\left(\eta>\eta_t\right)$

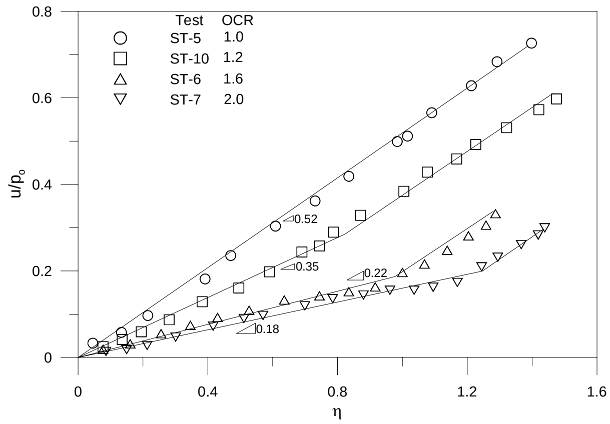

Fig. 5 $\left(u/p_0,\eta\right)$ plots for over-consolidated samples under monotonic loading

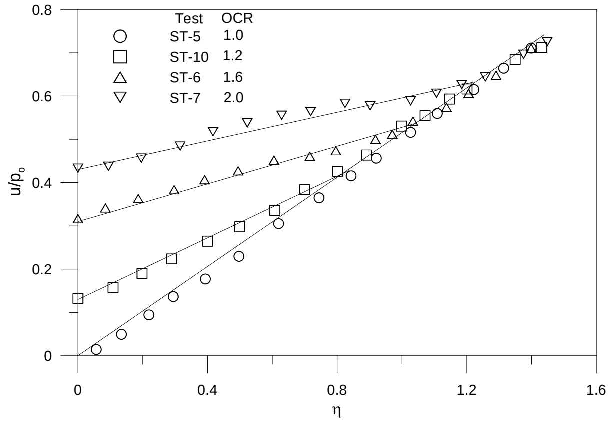

Fig. 6 Shifted $\left(u/p_0,\eta\right)$ plots for over-consolidated samples

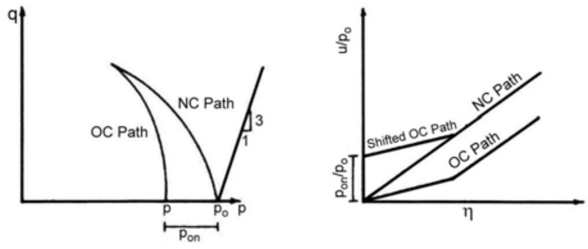

Fig. 7 Shifting of $\left(u/p_0,\eta\right)$ plot for overconsolidated samples

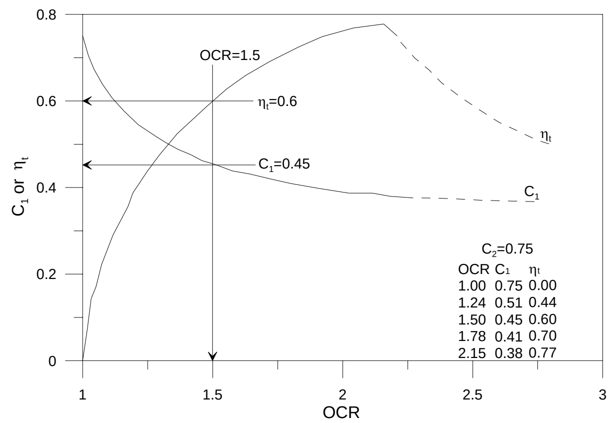

Fig. 8 OCR $-C_1-\eta$ relationship from CID samples

The pore pressure parameter C_1 decreased as OCR values increased, while the corresponding value of η_t increased. These relationships for CIU tests are simplified and presented in Fig. 8. Thus, C_1 and η_t values are determined directly from this figure if the OCR is known. The C_2 value is constant and can be determined by running an undrained test on normally consolidated clay.

PARABOLIC UNDRAINED STRESS PATHS

By carefully studying the undrained stress paths, Lee (1995) concluded elliptical shapes were found to be closer to the actual shapes rather than the parabolic shapes adopted by Pender (1978). Thus the undrained stress paths are modelled as,

$$\left(\frac{p-[p_{cs}]_{oc}}{p_0-[p_{cs}]_{oc}}\right)^2+\left(\frac{q}{M_{oc}[p_{cs}]_{oc}}\right)^2=1 \qquad (9)$$

The meaning of M_{oc} and $[p_{cs}]_{oc}$ are shown in Fig. 9. Eq. (8) can be written as,

$$q=\frac{M_{oc}[p_{cs}]_{oc}}{\left|p_0-[p_{cs}]_{oc}\right|}\sqrt{(p_0-p)\left(p_0+p-2[p_{cs}]_{oc}\right)} \qquad (10)$$

To extend the model to a more general expression, the effect of the rotation of the direction of principal stress can be included as

$$q=\eta_0 p+\frac{(AM_{oc}-\eta_0)\times[p_{cs}]_{oc}}{\left|p_0-[p_{cs}]_{oc}\right|}\times\sqrt{(p_0-p)\left(p_0+p-2[p_{cs}]_{oc}\right)} \qquad (11)$$

It is noted that Eq. (10) gives the same stress path in normalised (p, q) plot for normally consolidated clays sheared from isotropic pre-shear conditions as the yield locus in the modified theory when $p_0=2p_{cs}$.

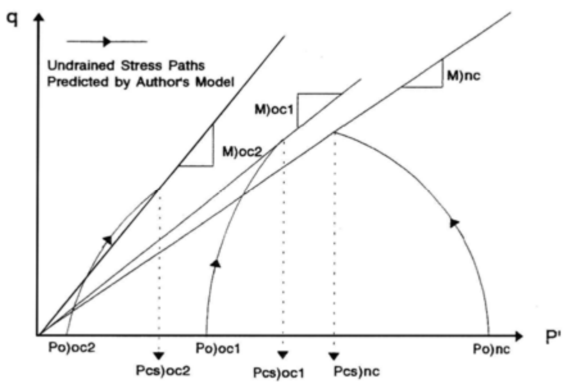

Fig. 9 Definition of clay properties used in Lee (1995)

Figure 10 and Fig. 11 showed the undrained stress paths as modelled by the Modified Theory and the Pender Model respectively. For normally consolidated clay, the undrained stress paths predicted by the Modified Theory are elliptical in nature, while the Pender (1978) Model assumes parabolic shape. For the over-consolidated clays, the Modified Theory predicts that the undrained stress paths will rise vertically, while the Pender Model assumes Critical State seeking parabolic paths. However, the model proposed in this paper shows the undrained stress paths are of elliptical shape for the normally consolidated region and wet side of the critical state, while the undrained stress paths on the dry side are elliptic but terminate in the Hvorslev (1960) type failure envelope.

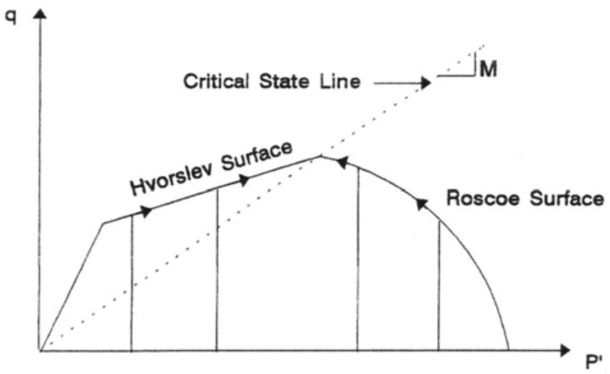

Fig. 10 Expected undrained stress paths (Modified Cam Clay)

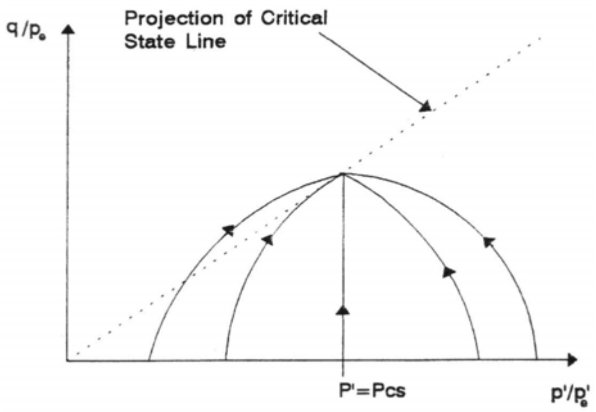

Fig. 11 Parabolic undrained stress path (Pender, 1978)

The undrained stress paths of normally consolidated and over-consolidated clays predicted by Pender's model and the authors are superimposed with the experimental data for CIU tests on normally and over-consolidated clay from Wroth and Loudon (1967) in (Fig. 12). The corresponding data for Bangkok clay are presented in Fig. 13 and Fig. 14. The CK_0U test data are presented in Fig. 15 together with the predictions. Similar data for extension tests on Bangkok clay are presented in Fig. 16 for normally and overconsolidated Bangkok clay. The extension data under K_0 conditions are presented in Fig.

17. It is found that the models developed by Handali (1986) and Lee (1995) give excellent prediction of undrained stress paths in triaxial compression and extension for both normally and over-consolidated states and under isotropic and K_0 consolidation states.

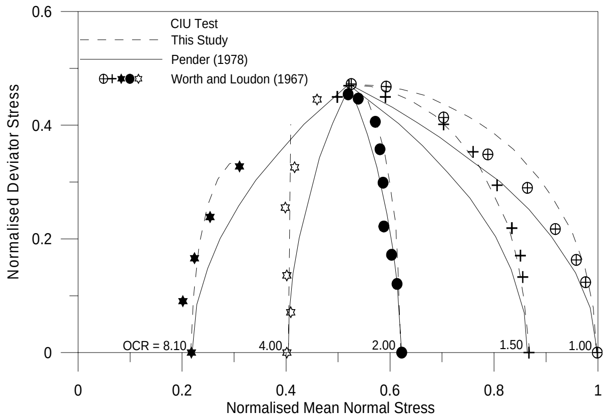

Fig. 12 Normalised (q, p) plot compared with the model predictions (CIU test)

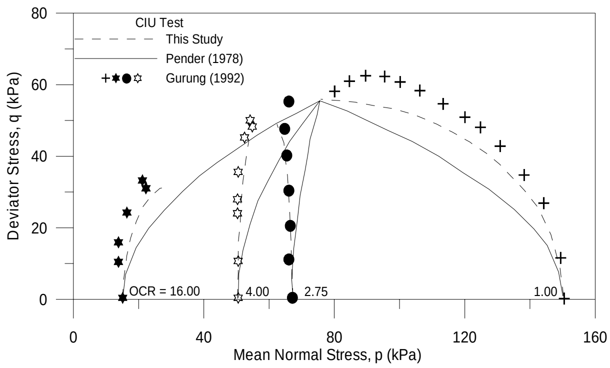

Fig. 13 Undrained stress paths compared with the model predictions (CIU test)

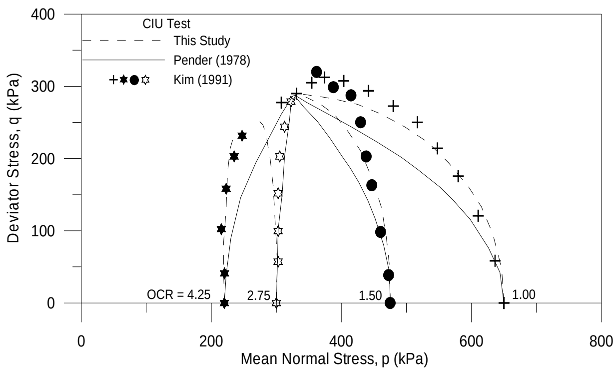

Fig. 14 Undrained stress paths compared with the model predictions (CIU test)

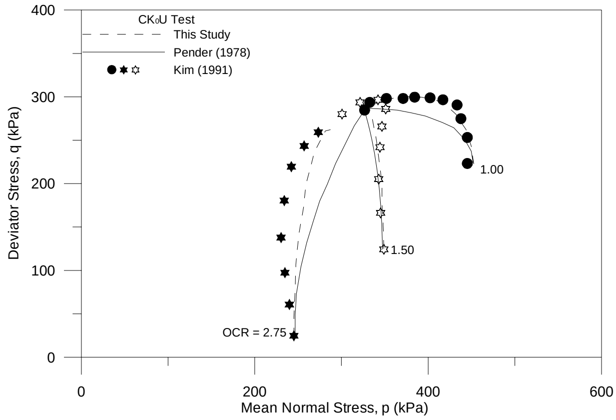

Fig. 15 Undrained stress paths compared with the model predictions (CK_0U test)

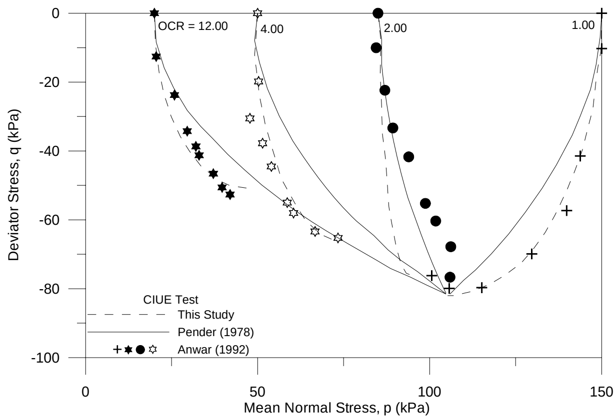

Fig. 16 Undrained stress paths compared with the model predictions (CIUE test)

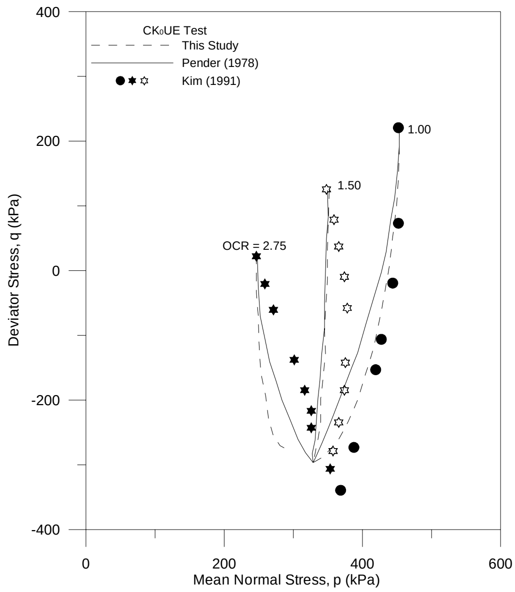

Fig. 17 Undrained stress paths compared with the model predictions (CK_0UE test)

CONCLUSIONS

In this paper alternate expressions have been derived for a pore pressure coefficient for normally and over-consolidated clays (dependent on the stress ratio) and valid for all pre-shear consolidation conditions. Also, parabolic expressions are found to model the undrained stress paths of normally and over-consolidated clays more effectively than Pender's elliptic formulations. The prediction of the pore pressure coefficients is important in settlement and stability analysis of embankments and excavations in soft clays. These coefficients have been found to be superior to those of Skempton and Henkel, as they are independent of the stress history and the level of deviator stress during shear.

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REFERENCES

Anwar, M. S. (1992). Extension Behaviour of Bangkok Soft Clay below State Boundary Surface. MEng Thesis, Asian Institute of Technology, Thailand.

Balasubramaniam A. S. and Chaudhry A. R. (1978). Deformation and strength characteristics of soft Bangkok clay. J. Geotech Eng. Div., ASCE., 106(GT6): 716-720.

Balasubramaniam, A.S., Handali, S., Phien-wej, N., and Kuwano, J. (1989). Pore pressure stress ratio relationship. Proceedings of 12th International Conference on Soil Mechanics and Foundation Engineering, Rio de Janeiro, Brazil, 1: 11-14.

Balasubramaniam A. S. and Uddin W. (1977). Deformation characteristics of weathered Bangkok clay in extension. Geotechnique, 27(1) : 75-92.

Bjerrum, L. (1972). Embankment on soft ground. Proceedings of the ASCE Specialty Conference on Performance of Earth and Earth-Supported Structures. Predue University, USA, 2: 1.54.

Gurung, S. B. (1992). Yielding of Soft Bangkok Clay below the State Boundary Surface under Compression Conditions. MEng Thesis, Asian Institute of Technology, Thailand.

Handali, S. (1986). Cyclic Behaviour of Clays for Offshore Type of Loading. PhD Thesis, Asian Institute of Technology, Thailand.

Henkel, D. J. (1959). The relationships between the strength pore water pressure and volume change characteristics of saturated clays. Geotechnique, 9: 119-135.

Hvorslev, M. J. (1960). Physical components of the shear strength of saturated clays. Proceedings of the ASCE Research Conference on the Shear Strength of Cohesive Soils. Boulder, USA: 169-173.

Kim, S. R. (1991). Stress Strain Behaviour and Strength Characteristics of Lightly Overconsolidated Clays. PhD Thesis, Asian Institute of Technology, Thailand.

Lee, C. J. (1995). Modelling of Stress-Strain Behaviour Below The State Boundary Surface. MEng Thesis, Asian Institute of Technology, Thailand.

Lo, W. B. (1976). Repeated Load Test on Bangkok Subsoils. MEng Thesis, Asian Institute of Technology, Thailand.

Pender, M. J. (1978). A Unified Model for Soil Stress-Strain Behaviours. Proc. 10th ICSMFE : 213-222.

Skempton, A. W. (1954). The Pore Pressure Coefficients A and B. Geotechnique, 4: 143-147.

Skempton, A. W. (1964). Long term stability of clay slopes. Geotechnique, 14: 77-101.

Wroth, C. P. and Loudon, P. A. (1967). The Correlation of Strains within a Family of Triaxial Tests on Overconsolidated Samples of Kaolin. Proc. of the Geotech. Conf., Oslo, 1: 159-163.