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SELECTION OF SOFT CLAY PARAMETERS FOR BANGKOK LOWLAND DEVELOPMENT

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ABSTRACT: The paper stresses the importance of a sound understanding of the soil behaviour in both the limit equilibrium and numerical analysis in soil- structure interaction problems: deep foundation for tall buildings; foundation for elevated expressways, subways, ground improvements works, tunnels for water supply, natural gas supply, sewerage and drainage. In the limit equilibrium analysis the use of Hvorslev strength parameters is suggested while for lightly overconsolidated clays the strength to be referred as a frictional component. In the numerical analysis a realistic stress- strain model for the behaviour of soft clays for stress states below the state boundary surface is recommended.

INTRODUCTION

The infrastructure development in the Bangkok plain has accelerated exponentially in the last twenty-five years. The city of Bangkok has witnessed unprecedented growth in construction activities: deep basements, tall buildings, elevated expressways, subways, tunnels for water supply, natural gas supply and drainage, and even a new airport.

All these activities are taking place in a sedimentary soil deposit with an extensive overlay of soft clay, which has low strength and high compressibility. Underlying the upper clay layer are several aquifers inter-bedded with clay and sand. Extensive ground water pumping from the aquifers has caused large piezometric drawdown and alarming subsidence.

The ground elevation in the Bangkok plain is only 1.0 to 1.5 m above the mean sea level and already many parts of the city are submerged during heavy rainfall and at high tide. Therefore all infrastructure development works need to be elevated above possible flood level and the roads and highways to be built on wide embankments.

The Asian Institute of Technology has fully exploited these developments in terms of research in Geotechnics. The Bangkok plain which is about 300 km (east-west) and 400 km (north-south) is filled with sedimentary soil deposit and is indeed a very large field experimentation centre with all the intense construction activities taking place. Additionally, large-scale field tests are carried out on embankments with and without ground improvements, deep excavations with and without support, piled foundations etc. One of the basic philosophy of research at AIT is to emphasize the role and purpose of understanding the soil behaviour and the selection of appropriate geotechnical parameters and soil models to be used both in limit equilibrium and numerical analysis using finite difference and finite element analysis.

Research in soil mechanics has taken a tremendous turn in its approach in the last two

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decades than that has adopted by Osborne, Coulomb, Rankine, Skempton, Roscoe, Meyerhof, Drucker, Prager and other scientists who carried out research with the traditional meaning of the word "research". The current trend in soil mechanics research is more commercially oriented and seems to blow with the wind in the direction in which the money is there.

There is clearly a category of problems in which the limit equilibrium analysis is applicable and are simple to be used in practice. At the same time there is also a category of problems in which numerical analysis is mandatory and helpful in assessing the potential risk.

Even in the deformation analysis, there are two types of demand. One, when the strain is large and the settlement and movements are also large. Here again the mode of deformation can be under constant volume condition (e.g. the short-term behaviour of a high-reinforced embankment in soft clay) or under consolidation condition (e.g. the improvement of soft clays with preloading and vertical drains). Then comes a class of problem in which the strains are small (e.g. the lateral movement of a deep diaphragm wall when its order of movements about 10 to 50 mm). It appears, currently that there is great confusion in handling the analytical aspects of these varieties of deformations and the safety aspects in soil engineering practice. Various types of computer software packages are indiscriminately used even by reputed individuals and organisations without a proper understanding of the geotechnical parameters or the associated constitutive soil models. It is intended that this paper will illustrate some of the uses and misuses of soil models and parameters in geotechnical analysis.

LIMIT EQUILIBRIUM ANALYSIS

For short-term stability of embankments and excavations in the Bangkok plain, total stress analysis with undrained shear strength under $\phi = 0$ condition is used in practice. Such an approach gives satisfactory results and aspects of anisotropy have been studied extensively but the results have confusing conclusions and are generally ignored. A better estimate of the strength other than the vane strength, using piezocone, dilatometer etc have also been tried as thesis and on special demands, but the old standard vane apparatus seem to do a trick in a more commendable manner.

Refinements to use effective stress analysis demand knowledge of excess pore pressure in the soft clay and such knowledge and evaluation seems an additional burden and factors for concern if not measured or calculated properly. Most of the stability problems of embankments and excavations in soft clay deposit only involve the upper clay layer, as the failure surfaces are shallow and generally less than 10 m or so. Extensive laboratory tests have been carried out in the triaxial apparatus for the strength of soft Bangkok clay and it is perhaps worthwhile to see whether the use of Hvorslev strength parameters in the effective stress analysis yield a better understanding of the performance of these structures. Concurrently for normally and lightly over-consolidated clays, Schofield has expressed the view that the behaviour is predominantly frictional and as such a variable friction angle with zero cohesion to match the appropriate strength envelopes can also be explored.

In the case of piled foundations for tall buildings and elevated expressways, bored piles are now used almost exclusively and driven piles are only confined to large diameter spun piles. The spun piles are founded in the first sand layer around 25 to 30 m depths and the bored piles are often extended to the second sand layer at around depths of 50 m. These piled foundations are fully designed using only the SPT N values and cone resistance and skin friction as estimated from Dutch cone tests. Pile load tests are carried out in almost all the major projects. One control that need to be exercised in entirely relying on the pile load tests data as an in-situ tests is the influence of the extensive drawdown in the Bangkok plain due to deep well pumping. When static pore water pressure is decreases and the effective stress increases. If there is a future possibility for ground water recharge, then the effective stress will again reduced and consequently the strength. Thus, the piezometric drawdown has an effect on all in-situ tests as well as on the samples recovered for laboratory tests. A higher safety factor must be employed to allow for such an increase in the capacity of the piles due to deep well pumping.

DEFORMATION ANALYSIS

In the introduction of this paper, emphasis was made on the large strain analysis as opposed to the need for a small strain analysis on specific problems. It has also mentioned that the large strain can occur as a consequence of undrained deformation or both from undrained and drained deformations. In order to make such decisions, a proper understanding of the soil behaviour is needed. The role of critical state theories and their validity in the solution of the various classes of problems will now be presented.

CRITICAL STATE THEORIES

The critical state theories developed from 1950 to 1975 have the unique potential to model the dilatancy of soil, which was first demonstrated by Osborne in 1856 for granular materials. Most of the elasto-plastic theories used in many versions of the current computer programs tend to offer a method by which the plastic dilatancy ratio can be obtained from Associated and non-Associated flow rules.

The original critical state theories as developed at Cambridge by Roscoe, Schofield, Wroth, Poorooshashb, Thurairajah, and Burland use an Associated flow rule for the volumetric yield locus. Especially the Roscoe and Burland version is found to be successful in predicting the volumetric strain and the pore pressure developed in normally consolidated clays, and the shear strain during anisotropic consolidation. As such, the Roscoe and Burland theory as used in the computer programs are capable of predicting the deformation under constant stress ratio stress paths, i.e. isotropic and anisotropic consolidation. However, the undrained deformation can not be predicted successfully using the dilatancy phenomenon.

Additionally, the Cambridge stress-strain theories are developed for normally consolidated clays, but almost all the soft clay deposits are lightly overconsolidated due to weathering and ageing effects. This is one of the single most limitation that is overlooked by analysts when they use the Cambridge stress-strain theories. Instead of incorporating a theory that will predict the behaviour of lightly overconsolidated clays most analysts tend to adjust the parameters M, λ and κ to best fit their laboratory and field behaviour. Values of M, λ and κ used for Bangkok clay by analysts seem to vary so much from project to project. These aspects will be clear when the case histories are presented. It is here that the work of Pender (1978), and Kim (1991) on the soft Bangkok clay becomes valuable.

STRENGTH AND DEFORMATION BEHAVIOUR

Below the State Boundary Surface (SBS), Balasubramaniam (1969) clearly demonstrated that plastic volumetric and shear strains took place for stress increments applied from stress states within the SBS. The constant q yield locus of Roscoe and Burland offered a means by which distortional strains can be experienced inside the SBS.

Pender (1978) developed a novel stress strain theory, which can contribute both plastic volumetric and dilational strains below the state boundary surface. By carefully analysing the experimental observations available on overconsolidated clays, Pender made a series of hypothesis on his theory.

Pender modelled the undrained stress paths of the normally and overconsolidated clays as a series of parabolas for various values of the overconsolidation ratio and these parabolas are all hooked together at the Critical State for each pre-consolidation pressure. This is indeed a novel modelling technique. Handali (1986) however found that the pore pressures developed under undrained tests on normally consolidated clays are linearly related to the stress ratio η.However for overconsolidated clays, bilinear relations were obtained with the constant of these relations dependent on the OCR values. Handali's (1986) method gave a better fit for the pore pressure development than the parabolic approach adopted by Pender (1978).

Pender (1978) also established a flow rule, which describe the plastic dilatancy ratio inside the State Boundary Surface. Here again the expression was such that the plastic dilatancy ratio varied with the *p*-value and also indicated positive pore pressure development in the lightly overconsolidated state and negative values for the heavily overconsolidated states.

STRESS-STRAIN AND STRENGTH CHARACTERISTICS OF LIGHTLY OVERCONSOLIDATED CLAY

Systematic triaxial testing programs were carried out to monitor and evaluate the properties and characteristics of lightly overconsolidated clays. It consists of many series of tests and the most particular one is shown in Fig. 1. In this series and others the path dependent stress-strain behaviour and strength characteristics of soft Bangkok clay in the normally and overconsolidated states were studied both under isotropic and K_o pre-shear consolidation conditions in compression and in extension. The experimental programs included triaxial

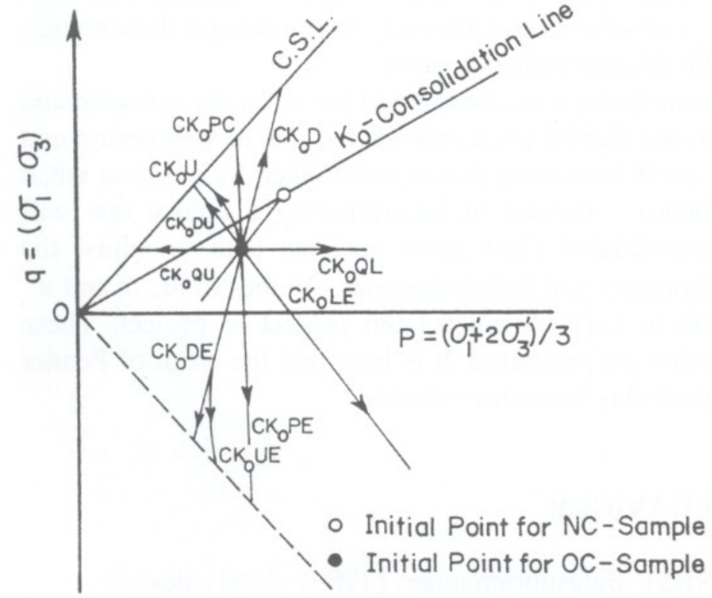

Fig. 1 Typical stress paths followed in this investigation of the behavior of lightly overconsolidated clay (test series, I)

Table 1 Index properties

Properties	Characteristics
Natural water content (%)	78-85
Liquid limit (%)	98
Plastic limit (%)	37
Plasticity index	61
Liquidity index	0.67-0.79
Average unit weight (t/m^3)	1.51
Specific gravity	2.69
Clay content (%)	70
Silt content (%)	24
Sand content (%)	6

consolidation and swelling tests (both isotropic and K_o-consolidation and swelling) as well as four series of undrained triaxial (three of these in compression and one in extension) and ten series of drained tests. Following the Hvorslev approach, the initial pre-shear void ratio of all the samples is kept constant. Some of the major conclusions are discussed in the following sections.

Index Properties of Clay

The clay used in this study was retrieved from depths of between 3.0 m to 4.0 m (a narrow range) within the Asian Institute of Technology. Thus the clay is expected to have the same initial structure, water content and index properties. The clay used in this study was undisturbed fairly homogeneous and dark gray in color. The physical properties are summarized in Table 1.

Consolidation and Swelling

The λ and κ values of the soft Bangkok clay in the (e, ln p) plot for the stress range covered in the tests program are 0.357 and 0.081, respectively. The idealised K_o-line for the overconsolidated sample is along a stress path (dq/dp)=0.95 and with both dq and dp less than zero.

Undrained Behaviour

A unique SBS is observed in the $(q/p_e, p/p_e)$ plot as shown in Fig. 2 both for isotropically and K_o consolidated samples in compression and in extension. The initial states of the K_o consolidated samples are found to lie inside the SBS as obtained from the isotropic tests, but with progressive shear their states rise to lie on the surface of the isotropically consolidated samples. The initial states of the K_o consolidated samples are thought to have a substantial pre-shear secondary consolidation effect. The critical state line for normally consolidated samples has the same slope both in compression and in extension and the Hvorslev strength envelope for the overconsolidated states also displays symmetry about the p-axis. The peak deviator stress for isotropically consolidated samples reaches a larger strain of about 10%, while the K_o consolidated samples reach such a state within 4% strain. The stress-strain behaviour is influenced by the pre-shear consolidation conditions, and the direction of the major principal stress whether in compression or in extension. The excess pore pressure generation shows a bilinear relationship in the $(u/p_o, \eta)$ plot. Such a relationship can be used to determine the undrained stress path and the excess pore pressure. The M values corresponding to the peak deviator stress and peak stress ratio conditions are 0.83 and 0.88 in compression with the latter value increasing to 0.93 in extension. The corresponding ϕ values are 21° and 23°, respectively in compression and 33° in extension. The corresponding strength ratio S_{uc}/σ_{vo} are 0.235, 0.252 and 0.291, respectively.

The state path of the overconsolidated samples lie within the SBS and exhibit volumetric yielding at higher stress ratio levels. The initial stress paths are sub-parallel to the q-axis confirming the elastic wall concepts of the Cambridge stress strain theories. The constant shear strain contours on the wet side of the critical state (for OCR values less than 2.15) confirm the constant q yield loci concepts. On the dry side the constant shear strain contours form a radial fan when extended converging to a negative p_o value on the p-axis. The overconsolidated samples also display a bi-linear $(u/p_o, \eta)$ relationship, and these relation can be used to describe successfully the undrained stress paths, and the generation of excess pore pressures. The A_f values corresponding to various OCR agree well with the predictions from the Modified theory. The end points of the lightly overconsolidated samples seek the critical state and tend to lie on the Hvorslev type of failure envelope. The strength ratio $(S_{uc}/\sigma_{vo})_{OC}$ / $(S_{uc}/\sigma_{vo})_{NC}$ increases with increasing OCR values. These strength ratios are predicted consistently with the Roscoe and Burland Theory using Λ of 0.77.

For the undrained behaviour of overconsolidated Bangkok clays under K_o pre-shear consolidation conditions, the constant shear strain contours tend to be sub-parallel to the idealised K_o line on the dry side and become parallel to the p- axis on the wet side. The A_f values are found to be different in magnitude for compression and extension tests, but can be predicted with the equation of Mayne and Stewart (1988).

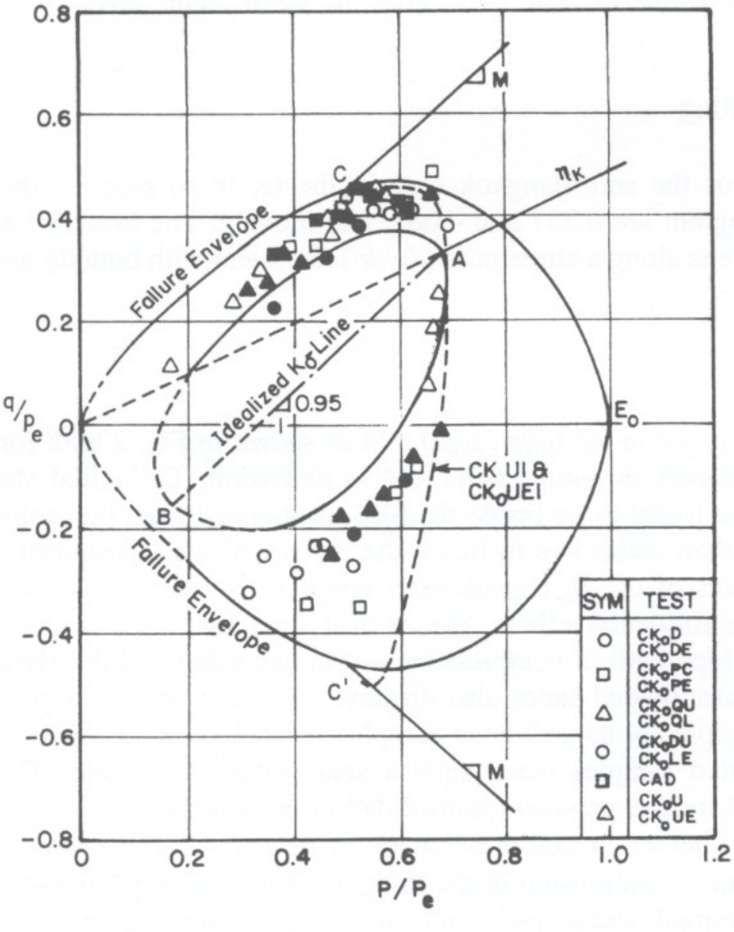

Fig. 2 State boundary surface and elastic zone for
K_o consolidated samples in $(q/p_e, p/p_e)$ plot

The drained stress path of the CID specimen approached the SBS at higher stress levels and the samples reach the normally consolidated states. When the stress paths lies on the SBS, the strains are of higher magnitude due to larger plastic volumetric strains. For all the drained specimens, when the stress paths cross from the overconsolidated to the normally consolidated state, the (ε_v, p) relation is used to determine the volumetric yield points. Such volumetric yield points constitute the volumetric yield locus on the SBS. For all the drained tests, the (q, ε_{vp}) or the (ε_{vp}, p) values are used to define the zones in which only elastic volumetric strain takes place and the zones in which plastic volumetric strains both compressive and dilation takes place inside the SBS. From the $(\varepsilon_{vp}, \varepsilon_{sp})$ relationship, the plastic dilatancy ratio $d\varepsilon_{vp}/d\varepsilon_{sP}$ is calculated for each stress path and this ratio is estimated such that it does not include the undrained shear strain component $d\varepsilon_s$ both on the wet and dry sides.

FORMULATION OF STRESS-STRAIN BEHAVIOR INSIDE THE SBS

Plastic Volumetric Strain inside the SBS

The qualitative description of the volumetric yielding inside the SBS to accommodate the plastic volumetric strain for the stress state near the isotropic consolidation close to the maximum past pressure, for the lightly overconsolidated state close to the volumetric yield loci and for the stress state close to the failure envelope on the heavily overconsolidated side, modeling is done considering the variation of λ. The variation of λ within the SBS is dependent both on the stress ratio (η) and p/p_e. To simulate the variation of λ inside SBS, two parabolic equations are proposed as follows:

$$\lambda_{OC} = \lambda_{NC}\left(p/(p_e)_{\eta o}\right)^{\frac{1}{2}} \tag{1}$$

and

$$\frac{d\lambda}{d\eta} = \kappa\left(1 - p/(p_e)_{\eta o}\right)^{\frac{1}{2}} \tag{2}$$

where λ_{NC} and λ_{OC} is the slope of the virgin consolidation line and the recompression line in e-ln p plot, and $(p_e)_{\eta o}$ is the equivalent stress corresponding to the initial stress ratio.

Constant shear strain contours within the State Boundary Surface

The constant shear strain contours as obtained from the CIU tests are similar to those presented by Wroth and Loudon (1967) and Balasubramaniam (1975) among others. An important feature of these contours is that they are sub-parallel to the p-axis in the lightly overconsolidated state and are radial in nature on the heavily overconsolidated states. Thus, they show distinct characteristics of the undrained shear behavior in the lightly and heavily overconsolidated states. Based on the typical behavior of these contours a relation for the incremental undrained shear strain (%) is proposed as:

$$d\varepsilon_S = \frac{\lambda_{OC}}{\kappa}(1+\kappa)M\,/\,n\,d\eta\exp\left[\frac{\lambda_{OC}}{\kappa}(1+\kappa)\eta - 2\Lambda\right] \tag{3}$$

where n is the overconsolidation ratio and Λ is $(1-\kappa/\lambda)$.

Plastic Strain Increment Ratio

The plastic strain increment ratio within the SBS is not uniquely dependent on the stress ratio but decreases as the mean normal stress decreases for a particular stress ratio. The plastic strain increment ratio is however, uniquely dependent on the stress ratio for stress state on the SBS. The variation of $d\varepsilon_v^P/d\varepsilon_s^P$ with respect to the stress ratio within the SBS can be simplified as a variation in which the plastic dilatancy ratio is only dependent on the value of p/p_e for any one particular stress ratio as shown in Fig. 3. Further, the trend is found to be same for both compression and extension conditions.

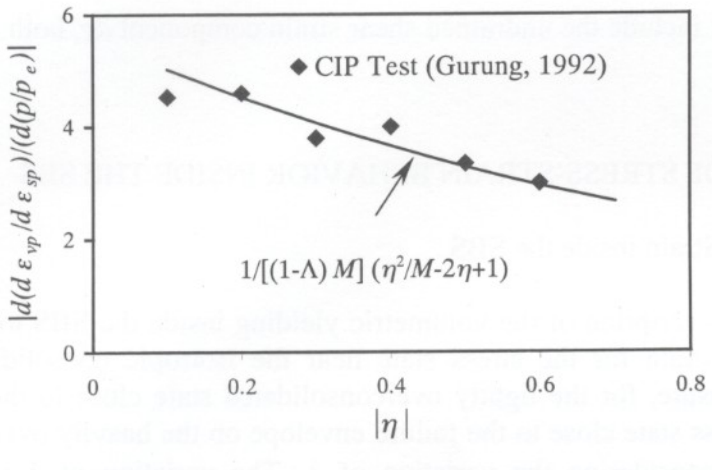

Fig. 3 Variation of $(d\varepsilon_v^p / d\varepsilon_s^p)/(p/p_e)$ versus the stress ratio within SBS

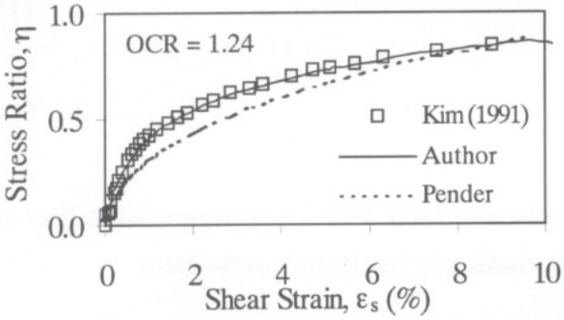

Fig. 4 (η, ε_s) Plot for CIU sample compared with model prediction

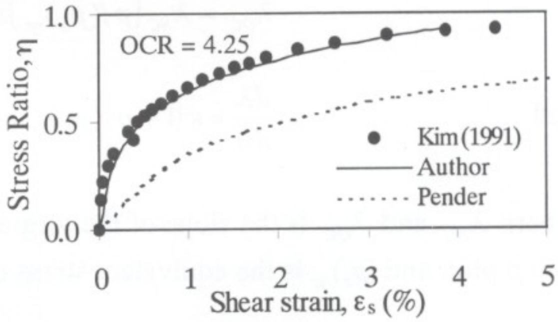

Fig. 5 (η, ε_s) Plot for CIU sample compared with model prediction

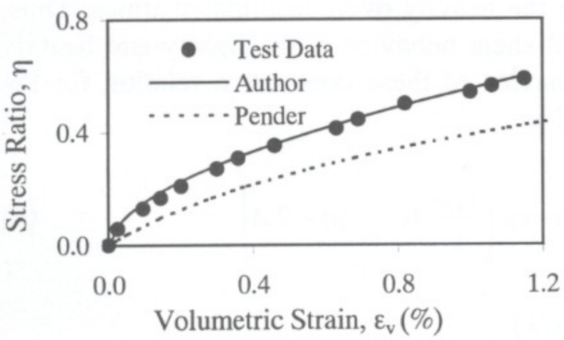

Fig. 6 (η, ε_v) Plot for CID test compared with model prediction (OCR=2.15)

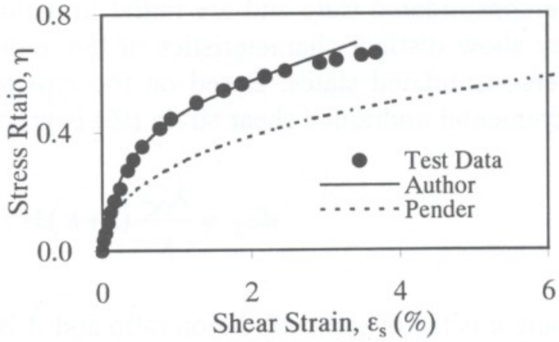

Fig. 7 (η, ε_s) Plot for CID test compared with model prediction (OCR=2.15)

Incremental Plastic Stress Strain Relation

$$d\varepsilon_s^p = \left(d\varepsilon_s^p\right)_{undrained} + \left(d\varepsilon_s^p / d\varepsilon_v^p\right) d\varepsilon_v^p \tag{4}$$

The incremental plastic volumetric strain $d\varepsilon_v^p$ is obtained considering the variation of λ inside SBS (Eqs. (1) and (2)) for the known undrained stress paths. The plastic dilatancy ratio $(d\varepsilon_v^p / d\varepsilon_s^p)$ is obtained from the relation with η and p/p_e in Fig. 3. The plastic shear strain for undrained condition $(d\varepsilon_s^p)_{\text{undrained}}$ is obtained from the Eq. (3).

Elastic Strains

The elastic volumetric strain is related to

$$d\varepsilon_v^e = \kappa/(1+e)(dp/p) \quad \text{and} \quad d\varepsilon_s^e = 0 \tag{5}$$

The prediction of the model thus developed is shown in Figs. 4, 5, 6 and 7. This model of course needs extensive refinement with additional detailed study.

CASE HISTORIES

Large Strain Analysis

Three test embankments (TS1, TS2 and TS3) with 40m x 40m plan dimension and side slopes 3:1 were built at the airport site in Nong Ngu Hao. These embankments were built on subsoil with PVD installed at spacing of 1.0, 1.2 and 1.5 m, respectively in a square pattern down to a depth of 12 m. A sand blanket of 1.0 m height was laid on the excavated ground (-0.3 m MSL) prior to the installation of PVD. After the PVD installation, the sand blanket was increased to 1.5 m. Then, clayey sand was used to raise the embankment to 4.2 m (i.e. 75 kPa of surcharge) in stages. During construction, stage I loading was up to 18 kPa. Stage II was taken to 45 kPa, followed by stage III to 54 kPa and stage IV to 75 kPa (4.2 m fill height). For TS1 embankment with 1.5 m spacing, a 5 m wide and 1.5 m high berm was installed, when the surcharge increased from 45 to 54 kPa. The berm width was increased to 7 m when the surcharge increased from 54 to 75 kPa. For TS2 and TS3, a berm width of 5 m and 1.5 m high was included when the surcharge increased from 54 to 75 kPa. The waiting period was 45 days for TS1 and TS2 with 54 kPa surcharge and this was reduced to 30 days for TS3 which has the closest spacing of PVD. The finite element analysis using the CRISP program was used for the prediction of parameters. The predicted and the observed values are shown in Figs. 8, 9 and 10.

The soil parameters used in the FEM analysis are given in Table 2. In the Roscoe and Burland theory as used in the CRISP program, the values of λ, κ and M are used. For the same clay deposits λ, κ and M need to be same for the normally consolidated states. However in Table 2, λ ranges from 0.10 to 0.90; κ ranges from 0.02 to 0.18 and M ranges from 0.90 to 1.20.

For a second case study at the same site with reduced pre-loading and vacuum induced consolidation the soil parameters used in the FEM analysis are given in Table 3. Now for the same clay λ ranges from 0.1 to 0.73, and κ ranges from 0.01 to 0.08 and M ranges from 1.0 to 1.4.

In a similar analysis for the Muar clay embankment in Malaysia that the soil parameters used are given in Table 4. Here λ ranges from 0.09 to 0.16, κ ranges from 0.04 to 0.06 and M range from 1.07 to 1.19. The Muar clay is vary similar to the Bangkok clay. At the same Muar clay site the analysis conducted on a reinforced embankment has the soil parameters listed in

Table 5. For the same Muar clay, κ ranges from 0.03 to 0.10, λ ranges from 0.10 to 0.61 and M from 1.07 to 1.20.

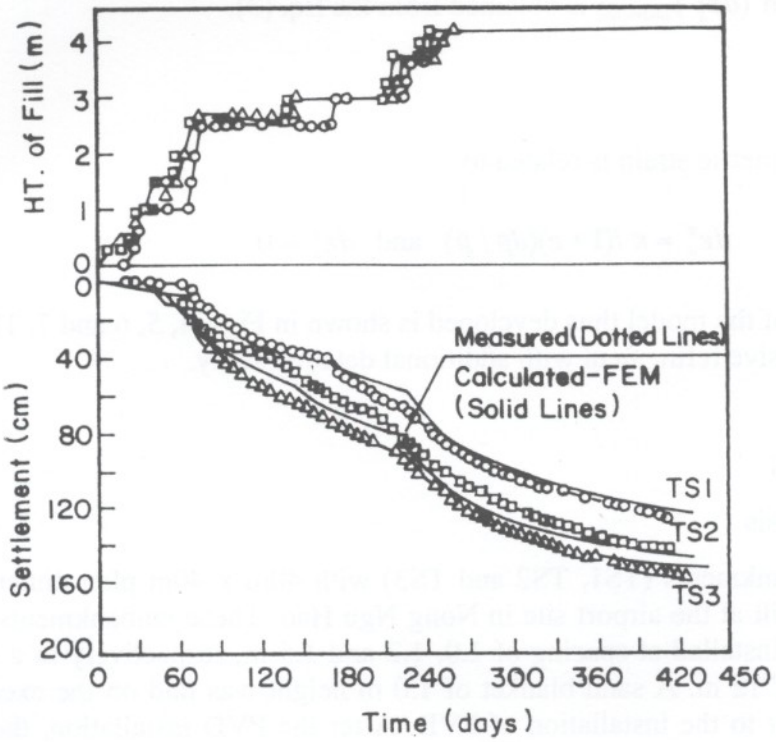

Fig. 8 Comparison of computed FEM and measured settlements with time of embankments TS1, TS2 and TS3

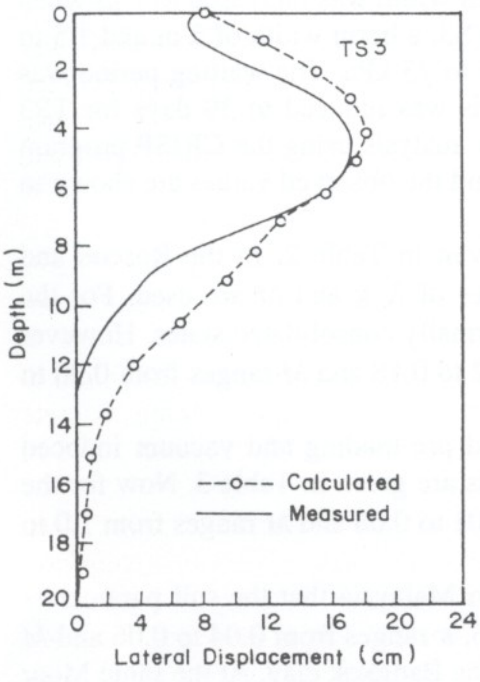

Fig. 9 Comparison of computed FEM and measured lateral deformations

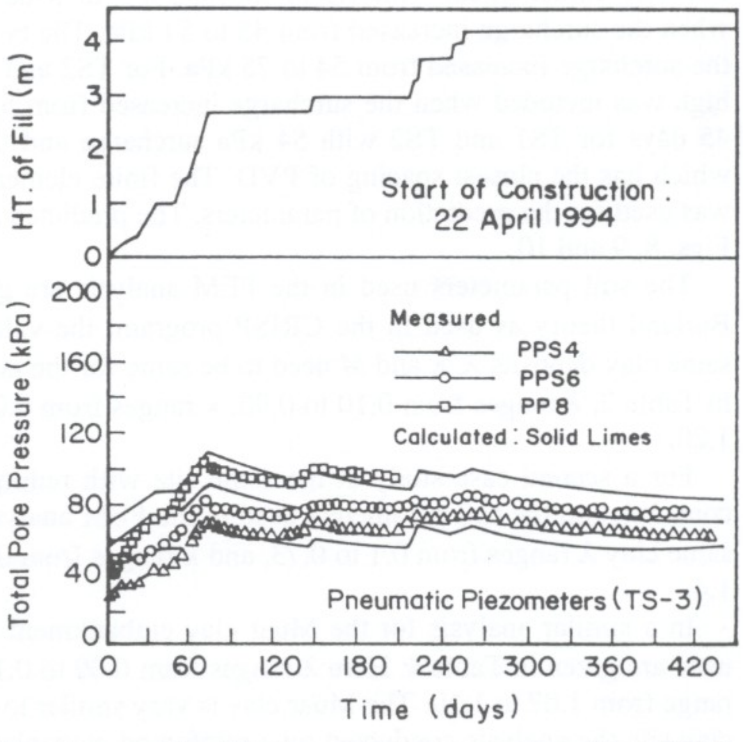

Fig. 10 Computed pore pressure from FEM and measured values- TS3

Table 2 Soil parameters used for FEM analysis for PVD improved ground with embankment surcharge

Depth (m)	κ	λ	M
0-2	0.07	0.34	1.20
2-7	0.18	0.90	0.90
7-12	0.10	0.50	1.00
12-15	0.07	0.34	1.20
15-18	0.02	0.10	1.20

Table 3 Soil parameters used for FEM analysis for PVD improved ground with vacuum preloading

Depth (m)	κ	λ	M
0-1	0.03	0.30	1.20
1-8.5	0.08	0.73	1.00
8.5-10.5	0.05	0.50	1.20
10.5-13	0.03	0.30	1.40
13-18	0.01	0.10	1.40

Table 4 Soil parameters used for Muar Clay in Malaysia (embankment)

Depth (m)	κ	λ	M
0-2	0.06	0.16	1.19
2-6	0.06	0.16	1.19
6-8	0.05	0.15	1.12
8-18	0.04	0.09	1.07

Table 5 Soil parameters used for Muar Clay in Malaysia (reinforced embankment)

Depth (m)	κ	λ	M
0-2	0.06	0.35	1.20
2-7	0.10	0.61	1.07
7-12	0.06	0.28	1.07
12-18	0.04	0.22	1.07
18-22	0.03	0.10	1.20

SMALL STRAIN CASE STUDIES

In this section the deformation analysis of deep excavations in the Bangkok subsoil is presented. The field data comes from four sites named as A, B, C and D.

In site A, the excavation was 9.8 m deep with three level basements. The excavation was supported by 0.82 m thick diaphragm wall up to 17 m depth. The barrette legs were installed at the bottom of the well to a depth of 44 m with a horizontal spacing of 5.4 m. Top-down construction method was adopted and two floor slabs were constructed at depths of 2.0 m and 7.0 m, which functioned as bracing during excavation (see Fig. 11).

In site B in Fig. 12 the excavation depth was 15.5 m for 4 level basements. The excavation was also supported by diaphragm wall of 0.82 m thickness with embankment at 20.0 m. Three floor slabs formed as internal bracing at 0.0, 2.0 and 9.5 m depths, which were formed by top-down method.

In site C a 26.0 m deep, 0.82 m.thick diaphragm wall was used to support an excavation of 18.5m depth. Barrette piles were installed below the wall to a depth of 50.0 m. Three concrete slabs were constructed by top-down method as supporting system at 2.75, 8.25 and 13.25 m depths (refer Fig. 13).

In site D, a 1.0 m thick and 20.0 m deep diaphragm wall was used to support a 16.0 m deep excavation. The bracing system consisted of strut-wales and king posts. The struts were pre-loaded as well in Fig. 14.

Here again the CRISP program was used and FEM analysis was carried out. The soil parameters used in the FEM analysis are given in Table 6 and the observed and predicted wall deflections are shown in Figs. 11, 12, 13 and 14 for sites A, B, C and D, respectively. There is good comparison between the predicted and measured values. Now the soil parameters used in FEM analysis are different from the previous values, κ ranges from 0.026 to 0.090, λ ranges from 0.111 to 0.358 and M ranges from 0.88 to 1.05.

It appears that for stress strain analysis, the use of the CRISP program with the Roscoe and Burland theory seems rather not precise. All what the Authors have done possibly is to select λ, κ & M in such a way the undrained stress strain curve is predicted well in this case.

Table 6 Soil parameters used in Bangkok Clay (deep excavation)

Depth (m)	κ	λ	M
0-2	0.053	0.182	1.05
2-15	0.090	0.358	0.93
15-22	0.026	0.111	0.88

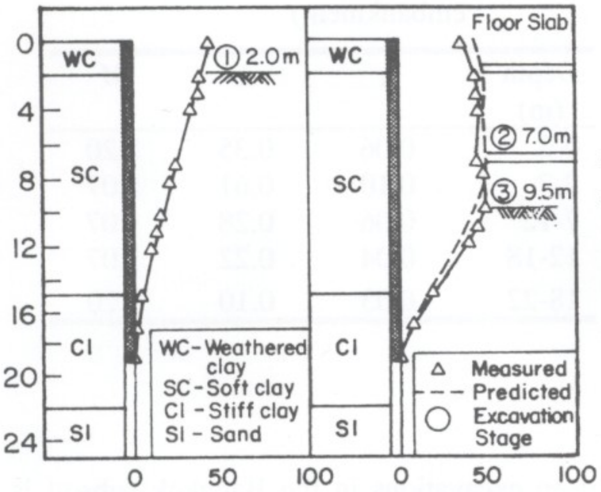

Fig. 11 Diaphragm wall deflection profile at site A

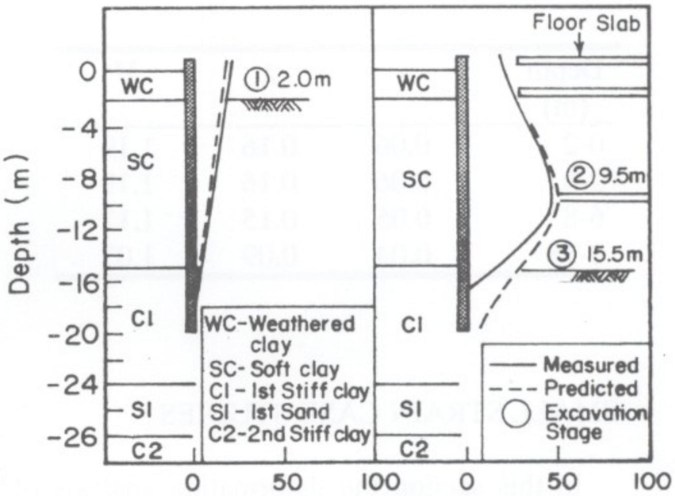

Fig. 12 Diaphragm wall deflection profile at site B

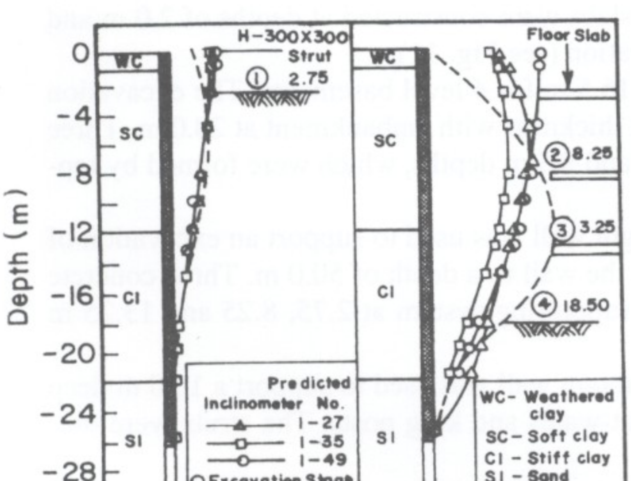

Fig. 13 Diaphragm wall deflection profile at site C

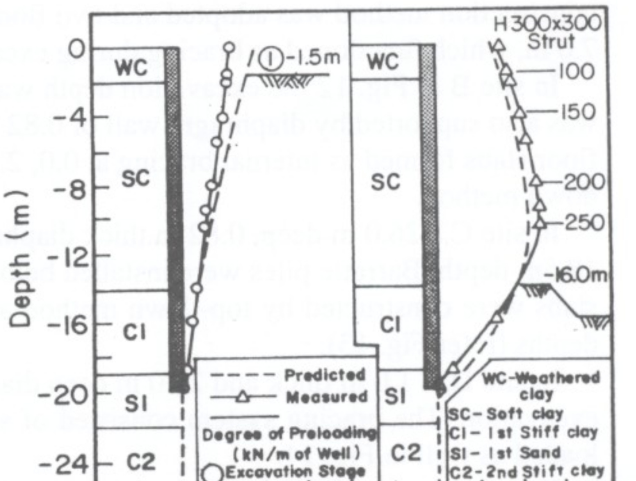

Fig. 14 Diaphragm wall deflection profile at site D

REFINEMENTS IN FEM ANALYSIS

From the material presented in this paper it appears that in the case when the strain is large and is of the consolidated type a relative stress strain model can be used in the FEM analysis which model the elasto-plastic behaviour below the SBS. However for small strain analysis possibly a simple non-linear elastic analysis is sufficient or the soil parameters can be back calculated from field observations. In the test data extensive research work has gone into small strain behaviour in the laboratory tests. It seems that at this strain level the test methods are far more complicated and the measured soil parameters need to be carefully reviewed for their corrections.

Additionally, the current computer programs do not contain a good soil model to evaluate undrained and drained creep. Especially in the use of PVD with high surcharge substantial time dependent lateral movements are takes place. A limited success is achieved in doing a field deformation analysis to evaluate such back figured parameters to be used in full-scale analysis.

CONCLUSIONS

The paper stresses the importance of a sound understanding of the soil behaviour in both the Limit Equilibrium and numerical analysis in soil- structure interaction problems: Deep foundation for tall buildings; foundation for elevated expressways, Subways, Ground improvement works, tunnels for water supply, natural gas, sewerage and drainage.

In the limit equilibrium analysis the use of Hvorslev strength parameters is suggested while for lightly overconsolidated clays the strength to be referred as a frictional component. In the numerical analysis a realistic stress- strain model for the behaviour of soft clays for stress states below the state boundary surface is recommended.

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