Discussion leader's report
Rapport de l'animateur

R. J. MAIR, Geotechnical Consulting Group, London, UK

After consultation with the General Reporter (Dr A Uriel), it was decided that the session should concentrate on cut-and-cover and bored tunnel construction in soils. In the first part of the session Dr Uriel presented those aspects of his General Report dealing with bored tunnels. In order to stimulate discussion on cut-and-cover tunnels, the following invited speakers were asked to present case histories:

Dr Z C Moh	Taiwan
Mr R L L Leite	Brazil

After Dr Uriel's presentation of his General Report dealing with bored tunnels, the Discussion Leader (R J Mair) introduced the topics selected for discussion. The invited speakers then presented their contributions, after which the floor discussion opened.

A summary of the topics introduced by Mair was as follows:

A - <u>Cut-and-cover tunnels</u>

A.1 Undrained or drained?
- selection of design parameters and calculation methods for construction in silts
- temporary works in clays: how best to allow for partial pore pressure dissipation during construction?

A.2 Influence of K_o
- on design earth pressures, strut loads, bending moments and ground movements
- significance of wall installation process: how much are in-situ horizontal stresses reduced?

A.3 Soil Stiffness
- selection of design parameters for design of braced walls
- derivation of 'spring stiffnesses' for Winkler-type design models

B. <u>Bored tunnels</u>

B.1 Ground loss (tunnels in clay)
- how best estimated/calculated?
- best related to stability ratio (overload factor) or load factor?

B.2 Surface settlement profile
- best estimated empirically from error function (Peck, 1969) or from more fundamental analyses as presented by Dr Uriel in his General Report?

B.3 Influence of anisotropy
- significance in design?
- how relevant to design is the cross anisotropic parameter G_{vh} (Rowe and Lee, 1989)?

B.4 Undrained shear strength, s_u
- most appropriate value for bored tunnel design (e.g. from CK_oUC, CK_oUE, DSS laboratory tests, or field vane tests?)

B.5 Long term settlements
- how to evaluate effects of consolidation?
- design parameters required?

In his introduction of the discussion topics, Mair made a number of comments on topic B.1. Geotechnical and tunnelling engineers are often faced with the difficult task of predicting ground loss for a proposed tunnelling operation. If the ground loss can be predicted with a reasonable degree of certainty, the corresponding magnitude and distribution of ground settlements can be fairly easily derived (Peck, 1969; Clough and Schmidt, 1981). Based on an elasto-plastic continuum analysis of an axisymmetric tunnel (Clough and Schmidt, 1981), several workers have presented field data in the form of the stability ratio (overload factor) N plotted aginst ground loss on a logarithmic scale. The General Reporter had also presented such a plot. Most of the case histories to date have been for tunnels constructed either in free air or in compressed air (rather than by the more modern technique of pressurizing the face with slurry). For such tunnels in clay, the critical stability ratio N_c (at which value the tunnel would collapse) varies with the depth, diameter and unsupported length of the tunnel, as shown in Figure 1. This figure is based on centrifuge model tests on shallow tunnels in soft clay (Kimura and Mair, 1981).

For two different tunnels, the value of N during construction may be similar but the value of N_c might differ significantly, depending on the respective geometries. The three-dimensional nature of a tunnel heading, which strongly influences the value of N_c, was extremely important. Mair therefore suggested that it would be more appropriate to relate ground loss to load factor, LF, where

$$LF = \frac{N}{N_c}$$

Experimental data illustrating the relationship between

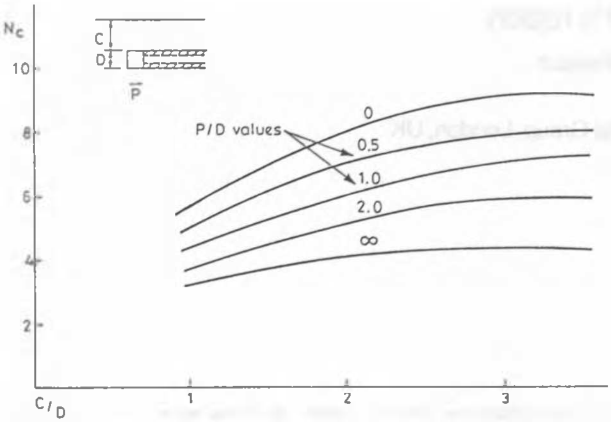

Figure 1 Dependence of critical stability ratio on tunnel geometry

ground loss and load factor were obtained from a series of centrifuge and 1g model tests on plane strain circular tunnels in kaolin clay (Mair, 1983). These are shown on Figure 2. Despite the wide range of cover-to-diameter ratios (C/D) and stress history of the clay, the data fall within a relatively narrow band.

Mair also pointed out that expressions for ground loss in terms of stability ratio derived from an elasto-plastic analysis of a plane strain axisymmetric tunnel (e.g. Clough and Schmidt, 1981) generally overestimated the ground loss observed in the field, often by a considerable margin in the case of soft clays. He believed that this was largely due to the

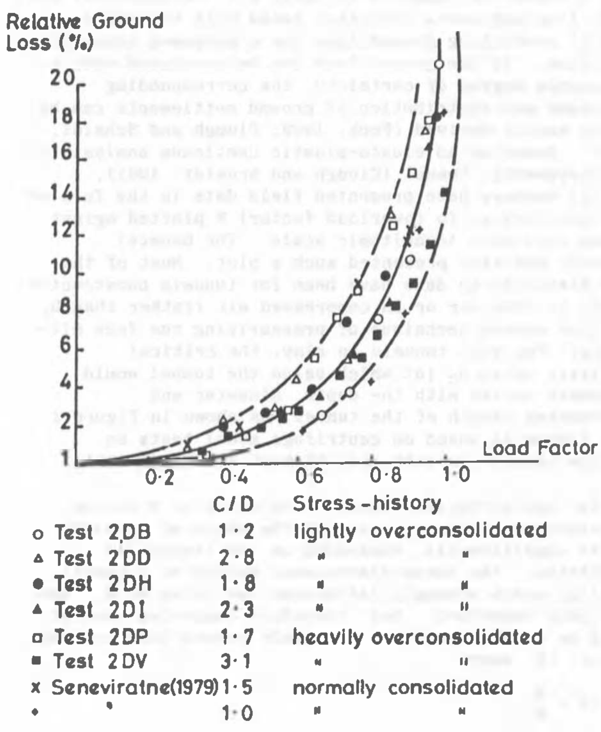

Figure 2 Observed ground loss for centrifuge and 1g model tunnels in soft clay

simplifying assumption of the tunnel geometry, which ignored the three-dimensional nature of the tunnel heading. He questioned whether linear relationships between stability ratio and field measurements of ground loss plotted on a logarithmic scale should be expected, and suggested that such data might be better presented on linear scales.

Regarding topic B.3 concerning anisotropy, Mair drew attention to the paper to the conference by Rowe and Lee, in which it was emphasized that the independent shear modulus G_{vh} was important in prediction of settlement induced by tunnelling (see Figure 3). He questioned the practical significance of this and whether laboratory measurements of G_{vh} were necessary for selection of design parameters for predicting deformations due to tunnelling.

The first invited speaker, Dr Moh (Taiwan), then presented aspects of diaphragm wall design considerations for the Taipei Subway. He emphasized the importance of obtaining reliable estimates of the appropriate strengths of silty clay or clayey silt strata in the passive zone beneath excavation level during construction. This required a detailed consideration of the in-situ pore pressures in that zone and the degree of swelling that was likely to occur. In a contribution with two colleagues in this volume, Moh gives a more detailed account of these problems, and how they were approached.

The second invited speaker, Mr Leite (Brazil), presented a case history from the Sao Paulo Subway, illustrating a method of predicting strut loads and displacements of a braced wall as excavation proceeds. A summary of his presentation is included in this volume.

In the floor discussion Professor G Gudehus (FR Germany) described the problems of tunnel excavations in very soft clays, which could often be underconsolidated. Large ground movements were often the consequence in these soils, and long term settlements due to drainage into the tunnel were almost inevitable.

Dr B Schmidt (USA) addressed the discussion topics concerning prediction of ground loss, the shape of the surface settlement profile above bored tunnels, and long term settlements due to consolidation. He emphasized the importance of the tail void left behind the lining as the shield advances; in the case of slurry or earth pressure balance shield operations, the volume of the tail void might represent an upper bound to the ground loss. Schmidt stated that field evidence showed that Sagaseta's method (presented in the General Report) of predicting the shape of the surface settlement trough was unconservative, and therefore he believed that it could not be recommended. Referring to his own paper to the conference on long term settlements due to consolidation of soft clay around tunnels, he felt that methods of prediction were possible and these should be compared with field measurements.

Dr Som (India) presented a case history of tunnel construction in soft clay for the Calcutta Subway. A pair of tunnels (5.5 m diameter) were constructed using compressed air, such that the estimated stability ratio was about 5. He agreed with the point made in Mair's introduction that elasto-plastic analysis of a plane strain axisymmetric tunnel considerably overestimated the ground loss. The observed figure was about 5%, compared with about 20% predicted by the analysis of Clough and Schmidt (1981).

Dr T B Celestino (Brazil) described a case history of an approximately 10 m diameter tunnel constructed by

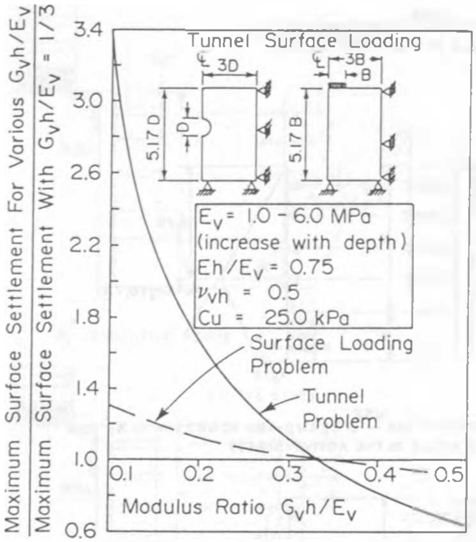

Figure 3 Influence of the ratio G_{vh}/E_v on settlement (from Rowe and Lee, 1989)

NATM in stiff clay for the Sao Paulo Subway. He showed that the elasto-plastic solution for a plane strain axisymmetric tunnel can be used to deduce the undrained strength from measurements of the variation of ground loss with overburden pressure (assuming that the support pressure did not vary significantly with overburden pressure). The value of undrained shear strength back-figured by this method was about 60% of the value measured in laboratory tests; this was considered to be reasonable in view of the fissured nature of the clay.

Mr Bello Maldonada (Mexico) presented an example of settlement calculation for 4 m and 6 m diameter tunnels constructed in soft clay for a deep sewage system in Mexico City. To account for the proximity of the ground surface to the tunnels, he proposed that a notional triangular-shaped stress be applied at the ground surface to calculate the surface settlement additional to the profile predicted by considerations of ground loss from an elasto-plastic axisymmetric plane strain solution. Good agreement was apparently obtained between the calculated and observed maximum surface settlements.

Dr R J Jardine (UK) discussed the question of whether elastic anisotropy was of significance. He pointed out that local strain measurements in laboratory triaxial tests show that the initial stress-strain behaviour of all soils is far stiffer and more non-linear than is usually appreciated. Also, the small strain characteristics are affected by the soil's recent stress history and can be significantly influenced by even perfect sampling techniques. To obtain representative stiffness parameters, he argued that it is necessary to reconsolidate to K_o conditions following a stress path that retraces the most recent stress history of the soil. In his view most of the procedures used in practice to estimate the cross-anisotropic parameter G_{vh} were doubtful. The stress-strain non-linearity was likely to be the principal reason for differences between observations and predictions of surface settlement profiles above tunnels.

The final contribution was from Professor B Ladanyi (Canada), who expressed concern that it should be more widely appreciated that a delayed response to tunnelling was not necessarily associated with consolidation, but rather due to deviatoric creep. He believed that determination of appropriate parameters characterizing this behaviour was an important part of parameter selection for underground construction.

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Panelist contribution: Proposal of a semi-empirical evolutive method for design of braced walls

Contribution de panelist: Proposition d'une méthode semi-empirique et évolutive pour le dimensionnement des étaiements

R.L.L.LEITE, São Paulo Subway Co., Brazil
F.R.STUCCHI, Maubertec Engenharia e Projetos Ltda, University of São Paulo, Brazil

SYNOPSIS: This paper presents a proposal of an evolutive semiempirical method design of braced walls, which was developed from other semiempirical method, traditionally used, but not evolutive. It is given a description of the method, its use in a stretch of the works and is presented a comparison between predicted and measured values.
The paper also presents a back-analysis of the excavation bracing system and horizontal displacements.

1. INTRODUCTION

Through the semiempirical not evolutive method, the bracing system (walls and struts) is calculated as girders with rigid supports. The values and the distribution of loads are previously imposed. This method, however, does not permit the determination of horizontal deformations due to the evolution of the excavation phases,fig.1.
It can be observed in the fig. 2 that the magnitude of earth pressure depends on the displacement of the wall and since its distribution depends on the same displacement, it is not possible to determine correctly the values of the earth pressure.
Through the method proposed in this paper, fig. 3, it is possible to calculate, with reasonable approximation, the wall's displacements, supposing the bracing system as a girder with elastic supports and introducing evolutive loads into the calculation (fig. 4).
This is an artifice for taking into account the displacements produced by the earlier excavation phase.
In this way, having calculated the displacement of the wall, we can determine, using an interactive procedure, the correct values of the magnitude and distribution of the earth pressure.
As an additional information the method permits the evaluation of soil settlements (fig. 5).

2. USE OF THE METHOD

To check the method, two cross sections of the works, with different soil characteristics, were instrumented.
In this paper it is presented the cross section with the major complexity (fig. 6). The correspondent geologic profile is presented in fig. 7.

3. SOIL CHARACTERISTICS

To determine the soil parameters some laboratory triaxial tests were carried out.
Deep sounding tests were carried out as well.
For interpretation of the figures obtained, the criteria of Jambu, Meyerhof e Schmertmann were used.

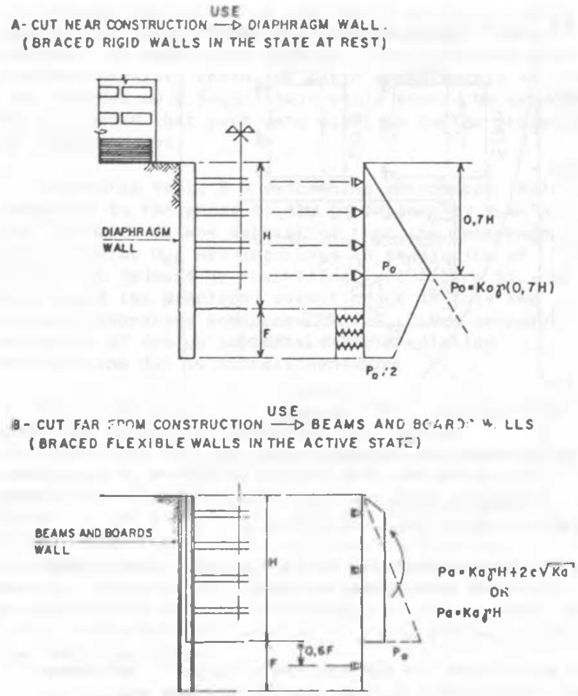


Figure 1. Semiempirical not evolutive method design for braced walls

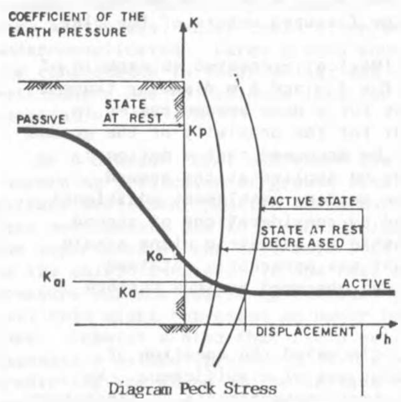


Figure 2. The influence of displacement of the wall on the magnitude of earth pressure

4. PREDICTION DESIGN VALUES

The performance of the works was predicted considering initially the massif in active state. This state was further confirmed through the calculation and measured displacements.
The adopted lateral pressure diagram (fig. 8) is composed by the superposition of earth pressure (considered constant) diagram, superficial uniform loads, and building loads (calculated through Boussinesq equations).
The predicted displacement values and strut loads are presented in fig. 8.

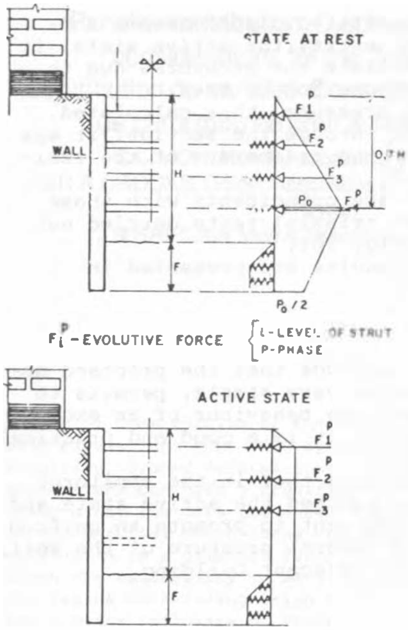


Figure 3. Evolutive empirical method to design for braced walls loads

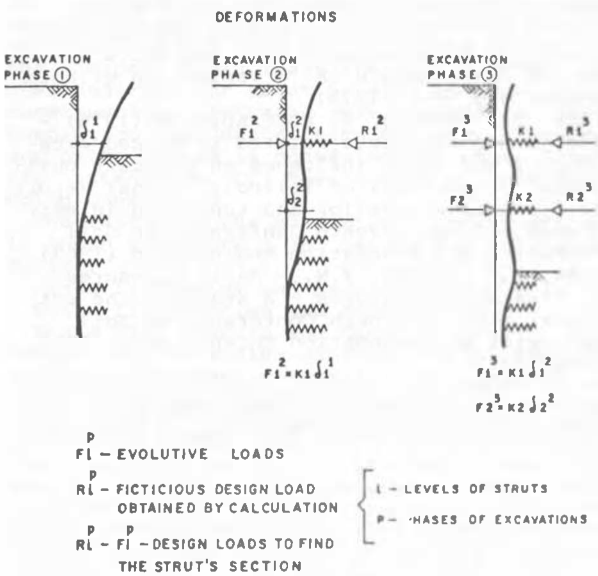


Figure 4. Evolutive empirical method to design for braced walls

This figure shows the semiempirical evolutive method principle relalet to the displacements. The evolutive loads simulate in phase (2) the displacements of phase (1), in phase(3) the displacements of phase (2) and so on.

5. PERFORMANCE OF THE WORKS MEASURED VALUES

The comparison between predicted and measured values is presented in fig. 9. It is verified that the differences were reduced, favouring safety:

PREDICTION OF THE SETTLEMENTS OF A STRUCTURE
(DEFORMATION OF THE MASSIF CONSTANT VOLUME)

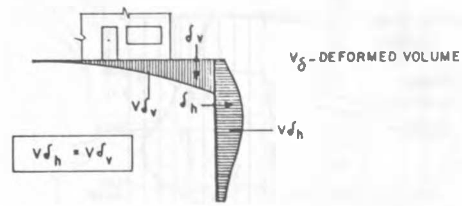


Figure 5. Evolutive semiempirical method to design criteria for braced walls

+ 5,6% in the resultant force of lateral pressure diagram
+ 9,0% in the maximum displacement of the wall

This figures confirm, within a reasonable approximation, the active state of the soil.

6. BACK-ANALYSIS

To confirm the active state of the soil and the type of arching adopted it is necessary to analysi carefully the measured values. Due to the axisymmetry of the loads, fig. 6, it was used a frame model with elastic supports. The hypothesis for this model were the same described in item 2, with the only addition of elastic supports which simulate the passive state in the less loaded side (fig. 10).

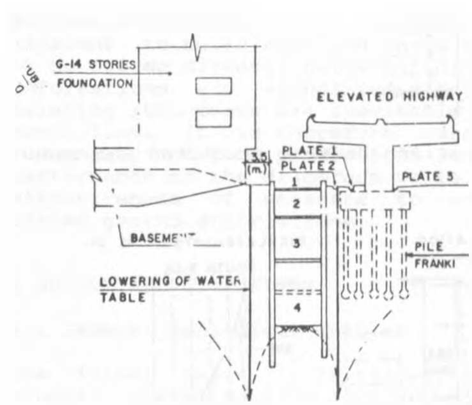


Figure 6. Instrumented cross section

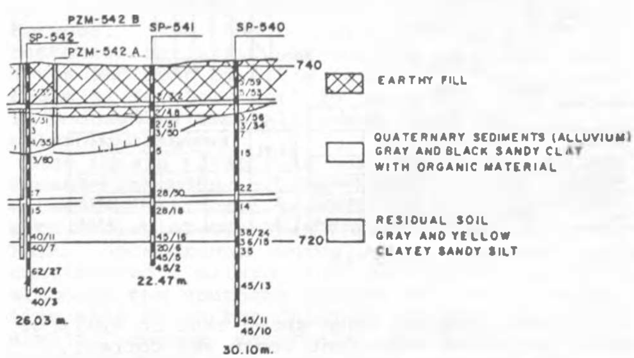
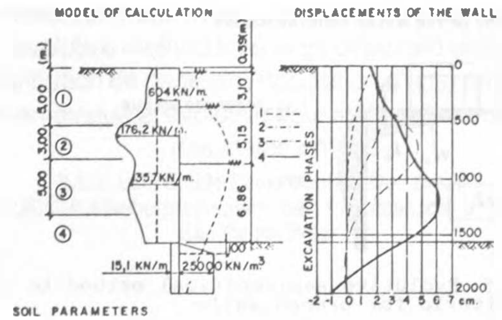


Figure 7. Geological profile



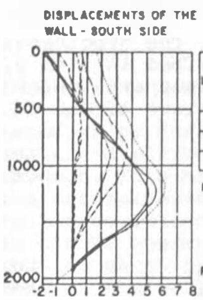
SOIL PARAMETERS

	KN/m³	KN/m²	DEGRE	KN/m²	
LAYERS	Y	C	φ	K_s	E
EARTHY FILL (1)	16,0	0,0	20°	2200,0	–
ALLUVIUM (2)	17,0	10,0	15°	2200,0	10000,0
CLAYEY SANDY (3)	17,0	0,0	40°	8500,0	20000,0
SILT (4)	17,0	0,0	40°	25000,0	60000,0

STRUT LOADS (KN)

LEVEL	EXCAVATION PHASES			
	1	2	3	4
1	–	123,0	0,0	64,0
2	–	–	679,0	506,0
3	–	–	–	957,0

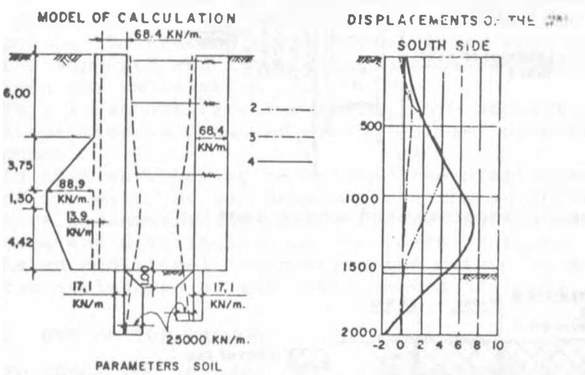
Figure 8. Evolutive semiempirical method for design applied on a history case. Predicted values-south wall



STRUT LOADS (Kn)

LEVEL	EXCAVATION PHASES							
	1		2		3		4	
			MEASURED	PREDICTED	MEASURED	PREDICTED	MEASURED	PREDICTED
1	—	—	143,0	123,0	18,0	0,0	10,0	64,0
2	—	—	—	—	417,0	679,0	463,0	506,0
3	—	—	—	—	—	—	982,0	957,0

Figure 9. Comparison between predicted and measured values



PARAMETERS SOIL

	KN/m³	KN/m²	DEGRE	KN/m²	
LAYERS	Y	C	φ	Ks	E
EARTHY FILL (1)	16,0	0,0	20°	2200,0	-
ALLUVIUM (2)	17,0	10,0	15°	2200,0	20000,0
CLAYEY SANDY (3)	17,0	14,0	30°	8500,0	25000,0
SILT (4)	17,0	14,0	30°	25000,0	30000,0

STRUT LOADS (KN)

LEVEL	EXCAVATION PHASES			
	1	2	3	4
1	-	106,0	0,0	94,0
2	-	-	597,0	373,0
3	-	-	-	958,0

Figure 10. Back analysis

The figures obtained have shown that in spite of the value of the resultant force was correct, the earth pressure due to the soil weight was underestimated and the building load was overestimated.

As the massif had suffered displacements sufficiently large to justify the active state, it was decided to calculate the pressures due to building load by the method of Krey.
To equal the total pressure, thus calculated, to the measured one through the section, it was necessary to adjust the parameters of the residual soil.
The new parameters are coincidents with those resultants from the triaxial tests carried out during the works (fig. 10).
The back-analysis results are presented in fig. 10.

7. CONCLUSION

It is possible to conclude that the proposed model, in spite of being very simple, permits to determine reasonably the behaviour of an excavation in cut and cover. It ie a good and practical tool for the design.
The study was conclusive that in the monitored section, the massif reached the active state and the arching was sufficient to promote an uniform distribution of the lateral pressure of the soil, without endanger the adjacent building.

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Panelist contribution: Diaphragm wall design considerations for the Taipei Subway
Contribution de panelist: Considérations sur le projet de paroi moulée pour le Métro de Taipei

Z.C. MOH, Moh & Associates, Taipei, Taiwan
J.H.A.CROOKS, Golder Associates on Secondment to MAA, Taiwan
J.N.SHIRLAW, Moh & Associates, Taipei, Taiwan

SYNOPSIS: Construction of the Taipei subway will involve deep excavations supported by diaphragm walls and internal strutting. Ground deformations associated with these excavations can have a significant effect on adjacent structures. The data presented indicates that the strength of the material which forms the passive zone in the excavation is a critical factor in controlling deformations. Given the variability in material type and stratigraphy, the degree of swelling which occurs during construction has a major influence on strength.

1 INTRODUCTION

The Municipal Government of Taipei, Taiwan, ROC has commissioned construction of a mass rapid transit system to alleviate traffic problems in the city. Planning and design of the Taipei Rapid Transit System (TRTS) has been underway for about 5 years and the first construction contracts were awarded in 1988. Completion of construction of the currently approved system is scheduled for 1994. Possible extensions to the system are presently being considered.

The layout of the currently approved project is shown on Figure 1 and can be sub-divided as follow:

Phase I: Tamshui and Mucha Lines
Phase II: Hsintien and Nankang Lines
Phase III: Panchio and Chungho Lines

Except for the medium capacity Mucha line, the design passenger volume for the system is 50,000 passengers/hour one way.

The currently approved route is 72 km long, with 67 stations. About half of the stations and track will be constructed below grade with the majority of the underground work being concentrated in Phases II and III. Each of the 37 underground stations will typically be 200-300 m long and between 15 to 28 m deep. A further 12.5 km of cut and cover work is planned for cross-overs, sidings, pedestrian shopping malls and the like. Most cut and cover excavations will be supported by internally braced diaphragm walls. About 45 km of diaphragm wall will be required. The remaining 18 km of underground track will be constructed in 5.6 m diameter bored tunnels.

As with any major underground construction project in an urban area, the effect of construction on existing structures is a major concern. In densely populated Taipei, the closely spaced multi-storey buildings are normally supported on floating foundation systems; few structures are piled. Buildings with 4 storeys or less are founded on shallow footings. The layout of the MRT system is such

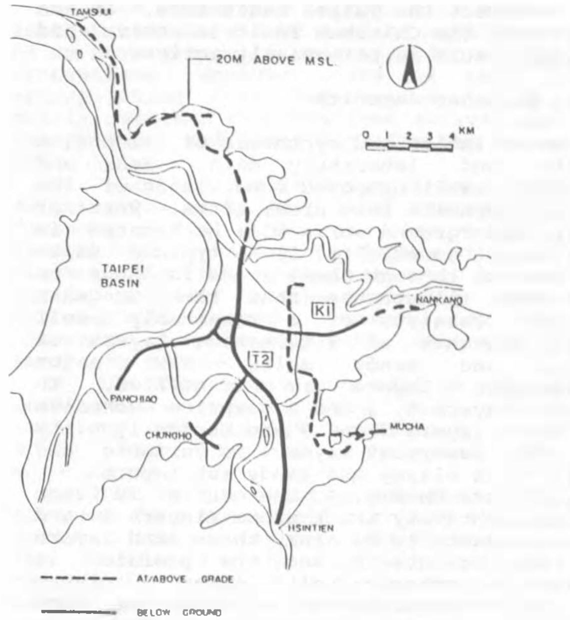

Figure 1. Taipei Rapid Transit System

that there will be numerous cases where deep excavations are to be located immediately adjacent to buildings, and where tunnels are to be driven directly below buildings. Ground deformations and associated effects on pre-existing structures are inevitable and must be controlled. It is therefore essential that reasonably accurate predictions be made of the performance of the diaphragm walls, and of the effectiveness of measures to control associated ground deformations.

2 SUBSURFACE CONDITIONS

2.1 General geological setting

The Taipei basin is relatively flat at a general elevation of 5-10 m above sea level. It is approximately square in shape, and measures 15-20 km in the N-S and E-W directions. Within the basin, the Hsintien and Keelung rivers join the Tamshui river (Figure 1) which flows northwest to the Strait of Formosa. The basin which was formed by tectonic activity, is bounded to the east, south and west by hills formed by sedimentary rocks of Tertiary age. To the north, the basin is bounded by mountains consisting of rocks of volcanic origin.

The basin itself is underlain by upper, recent deposits of the Sungshan formation, extensive Chingmei gravels and a lower hard sandy clay of the Hsin Chuang formation. The TRTS underground works will largely be constructed within the Sungshan deposits although the southern portion of the Hsintien line will be in the Chingmei gravels and other similar coarse materials deposited by the Hsintien river.

The island of Taiwan is located in an area of moderate seismicity. Of the major faults which transect the Taipei basin area, it is thought that the Chinshan fault is active and the Taipei fault is potentially active.

2.2 The Sungshan deposits

Based on collation and synthesis of extensive borehole and laboratory data, Moh and Associates (1987) proposed subdivision of the Sungshan deposits into areal zones. Most of the TRTS underground works will be located in the T2 and K1 zones (Fig. 1). A typical east-west section through these areas is shown on Fig. 2 and illustrates that the Sungshan formation consists of a reasonably well defined sequence of alternating layers of cohesive and sandy soils. Six major stratigraphic layers are identified. In summary, layers 6, 4 and 2 comprise cohesive soils while layers 5 and 3 are basically silty sands. The lowermost layer 1 is variable and contains both clayey and sandy sub-layers.

As indicated on Fig. 2, the central T2 zone contains relatively thick sands (layers 3 and 5). In the easterly K1 area, these sand layers are thin or absent and the profile is dominated by cohesive soils. Layer 4 clayey soils occur frequently at and below the TRTS excavation invert levels and these materials are therefore very important in design of the underground works. A particular feature of the layer 4 soils is that the material is silty and exhibits low plasticity in the central T2 zone, but becomes much more clayey and plastic in the K1 zone. The combined effect of varying plasticity of layer 4 and change in thickness in sandy layers from east to west has a significant influence on performance of excavations and tunnels.

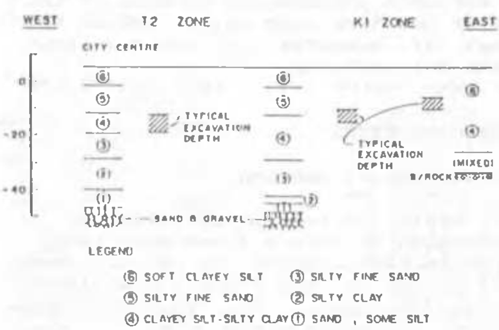

Figure 2. Typical east-west section showing simplified stratigraphy

2.3 Groundwater conditions

For about 20-30 years up to the 1970's there was extensive pumping from the Chingmei gravels for water supply. Ou et al (1983) report that this practice caused as such as 2 m of ground surface subsidence in the central basin area and affected the entire basin area to some degree (Fig. 3). Pumping was prohibited in the 1970's. The current piezometric conditions (Fig.4) reflect this history. Initially, conditions were presumably hydrostatic with a groundwater level at about sea level. Pumping caused a

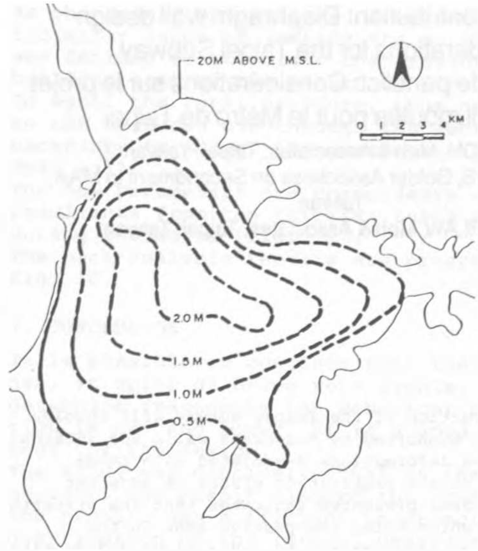

Figure 3. Contours of ground subsidence (metres) between 1955 - 1986

maximum of about 40 m head loss in the gravels which was reflected in layers 1, 2 and 3. However, layer 4 appears to have acted as an aquitard because conditions in the upper layers 5 and 6 were not significantly affected (i.e. they remained hydrostatic with respect to the previous water table). Pumping from the Chingmei gravels has had some effect on the layer 4 soils. However this effect is variable across the basin depending both on the local stratigraphy together with the variable compressibility and permeability characteristics of layer 4. Following cessation of pumping, the piezometric pressure in the Chingmei gravel and overlying Sungshan units 1-3 has recovered rapidly as shown on Fig. 4. In 1988, conditions in these lower layers were still sub-hydrostatic with respect to sea level but recovery is continuing. Recovery of water pressures will continue during the TRTS construction period and must be taken into account in the design of the works.

3 DIAPHRAGM WALL EXPERIENCE IN TAIPEI

Diaphragm walls have been widely used in Taipei primarily for the construction of deep basements (Moh and Ou, 1979a; Moh and Song, 1984). Typically these excavations have been 15 m to 17 m in depth, similar to that for some of the TRTS stations. Good quality field monitoring data from these excavations is available and forms a valuable basis for evaluating wall performance.

The results of monitoring from some of these excavations are summarized on Fig. 5 and indicate that for excavations in the T2 area, where conditions are most favorable, maximum wall deflections equal to about 0.5% of the excavation depth can be expected. For the K1 area, maximum wall deformations are typically greater at 0.5-0.75% of the excavation depths. This range of deformation is reasonably consistent with published data for other similar

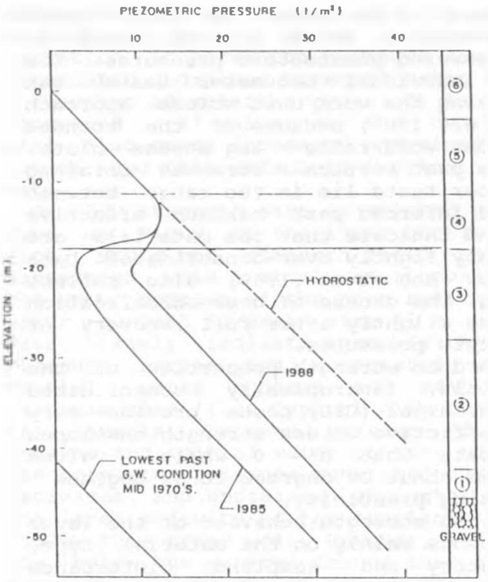

Figure 4. Changes in ground water conditions - city center area

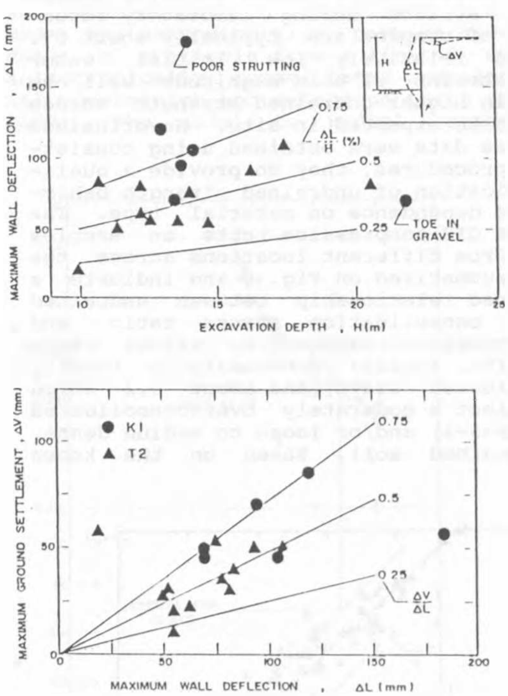

Figure 5. Deformations of diaphragm wall in Taipei

structures. On the lower portion of Fig. 5, maximum ground surface settlement is related to maximum wall deflection. For excavations in the more sandy T2 area, maximum ground surface settlement is about 50% of the maximum wall deflection. As expected, the ratio is higher for excavations in the more plastic K1 area.

The data summarized on Fig. 5 is encouraging in that maximum ground surface settlements of 50 mm have generally been recorded adjacent to excavations. However, care is required in extrapolating this information, which was mainly obtained from basement excavations, to the TRTS cut and cover work for the following reasons:

(1) Some of the TRTS excavations will be deeper (i.e. up to 28 m deep) whereas the performance data shown on Fig. 5 are for excavations with maximum depths of 15-17 m. Further, the TRTS excavations will be long and narrow compared to the typically square or rectangular shaped basement excavations; therefore, beneficial three dimensional effects are reduced.

(2) The TRTS excavations will be open for a much longer period, which will allow more swelling and strength reduction in clayey soils than is the case for building excavations.

(3) Most of the existing experience has been obtained from construction in the stronger T2 soils, with relatively limited experience in the K1 area. For the depths associated with the TRTS structures in the weaker K1 soils, significant deformations can be expected. Kao et al (1987) report cases which indicate the dangers associated with extrapolating experience from the T2 area to the K1 area.

(4) The relatively encouraging performance of excavations in the T2 area shown on Fig. 5 was largely achieved in the past 5-10 years when groundwater conditions in the basin were favorable. Referring to Fig. 4, it is clear that the water pressure in the material forming the critical passive zone of excavations, was in fact much lower than it is currently; during construction of the TRTS works, further increases in water pressures will occur. Unless adequate groundwater control is implemented to restore the previously prevailing beneficial groundwater conditions, significantly greater wall deflections and associated ground surface settlement can be expected.

While the previous experience forms a valuable and encouraging data base, good analytical techniques are needed to take into account the differing conditions which will prevail during the TRTS work. For many of the typical station excavations, cohesive soils of layer 4 will occur at and below the base of the excavations. The stiffness and strength of this layer are the controlling factors in wall design. Therefore the properties of this material are discussed in more detail below.

4 ENGINEERING PROPERTIES OF THE LAYER 4 DEPOSITS

As noted previously, the nature of the layer 4 materials varies significantly across the basin. Typical ranges for various index and engineering properties together with gradation limits are summarized on Fig. 6. From these data it is evident that despite similarity in terms of depositional history, the layer 4 soils will exhibit a range of behavior from that associated with plastic clay to coarse silt/fine sand. This wide variation in engineering behavior together with variations in stratigraphy, result in very different excavation design and ground movement control

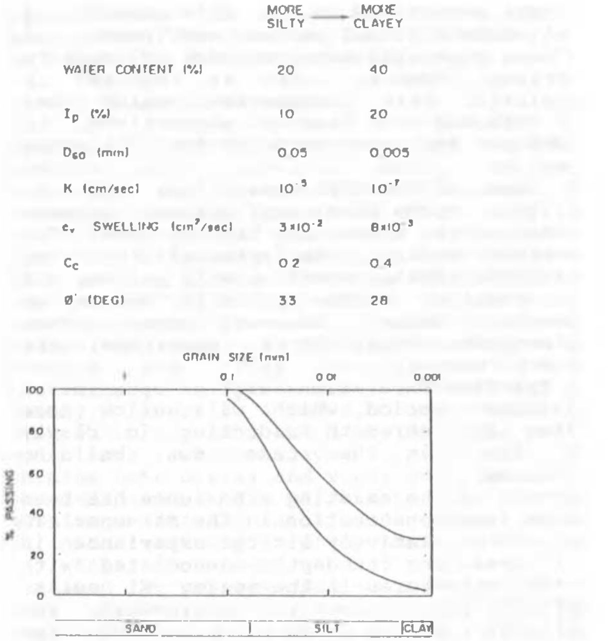

Figure 6. Typical engineering properties - layer 4

philosophies between the central and eastern portions of the underground work.

The stress history of the layer 4 soils can be expected to be largely dependent on the changes in piezometric pressures which have occurred due to past groundwater pumping. A typical profile of maximum past vertical stress determined from oedometer tests is shown on Fig. 7 together with the past maximum vertical stress inferred from past groundwater

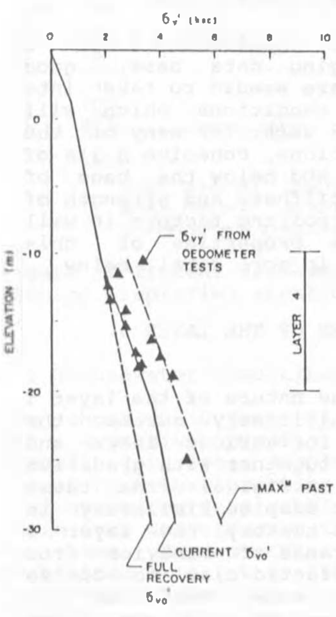

Figure 7. Typical stress history profile - layer 4

level changes. Also shown is the current vertical effective stress profile which is based on measured piezometric pressures. The data from individual oedometer tests was obtained using the work/unit volume approach (Becker et al, 1987) because of the rounded nature of the void/ratio - log stress plots. The maximum past vertical stresses obtained from oedometer tests lie in the range between current and inferred past maximum effective stresses and indicate that the materials are currently only lightly over-consolidated. Data presented by Moh et al (1989) also reflect this finding. The degree of over-consolidation will increase slightly after full recovery of the piezometric pressures.

With regard to strength properties of the layer 4 soils, isotropically consolidated undrained triaxial (CIU) tests provide very consistent effective stress strength envelopes which indicate that c'= 0 while values increase from about 28 degrees to 33 degrees with decreasing plasticity.

The undrained strength behavior of the layer 4 soils depends mainly on the material type, stress history and sampling disturbance effects. Material type effects can be explored based on the results of CIU compression tests on samples reconsolidated approximately to the in situ vertical effective stress. It should be noted that because of sampling disturbance which is difficult to avoid in silty soils, volumetric strains during laboratory reconsolidation of samples are typically about 5%. Given the relatively low initial water content, strains of this magnitude will be reflected in higher undrained strength values than could be expected in situ. Nevertheless since these data were obtained using consistent test procedures, they do provide a qualitative indication of undrained strength behavior and its dependence on material type. The results of CIU compression tests on samples obtained from different locations across the basin are summarized on Fig. 8 and indicate a well defined relationship between undrained strength: consolidation stress ratio and porewater pressure response. A_f values range between unity, typical of normally or lightly overconsolidated clays, and about 0.2 which would reflect a moderately over-consolidated clay (OCR=3-4) and/or loose to medium dense, coarser grained soil. Based on the known

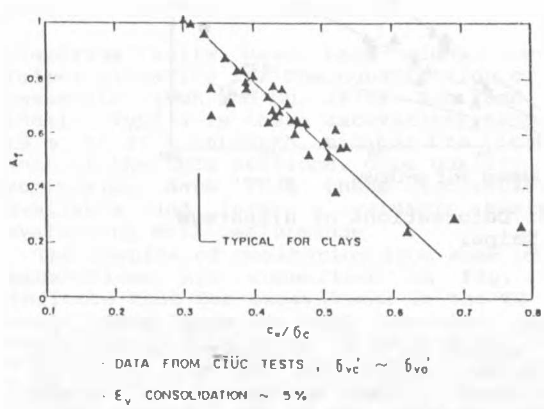

Figure 8. Undrained strength and PWP behaviour - layer 4

stress history of the deposit (i.e. only light over-consolidation), it is considered that the variation in porewater pressure response and associated undrained strength ratio is largely a function of material type. It is noted that increased ϕ' values for the coarser soils will also cause increased undrained strength ratios and this effect is implicit in these data.

The CIU strength data is related to material type on Fig.9. On the lower portion of the plot, undrained strength ratio and A_f are correlated with initial water content while the upper portion of the plot relates initial water content to grain size (i.e. D_{60}). The data clearly indicate that high A_f and low strength ratios are associated with high initial water content and small average particle size (i.e. typical clay behavior). As the material becomes coarser (i.e. D_{60} increases and initial water content decreases), the porewater pressure generated during shear decreases and undrained strength increases. The large majority of the available test data follow this trend well. However, a few points lie below the general trend; this may be due to a greater degree of over consolidation of these samples.

Better definition of in situ undrained strength for diaphragm wall design is obtained from tests on samples which are anisotropically consolidated to their in situ stresses and sheared both in compression (i.e. active mode) and extension (i.e. passive mode). Such data is available from a limited number of anisotropically consolidated undrained triaxial compression and extension tests (CAUC and CAUE). The axial to radial consolidation

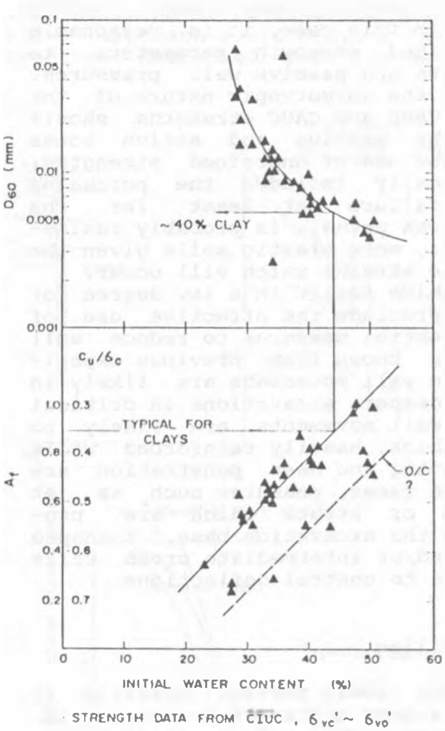

Figure 9. Dependence of strength and P.W.P. behaviour on material type - layer 4

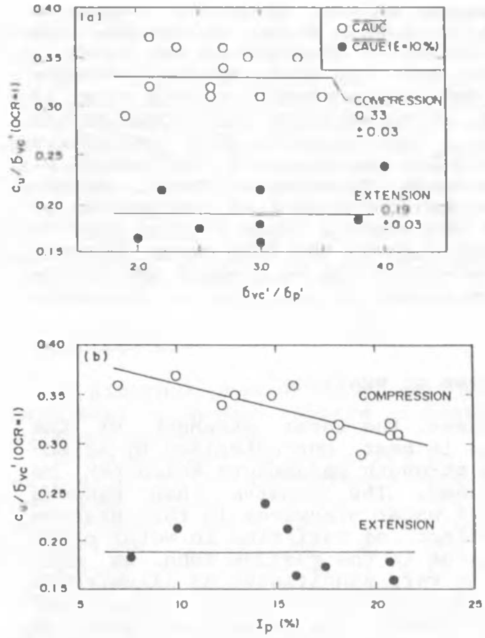

Figure 10. Undrained strength anisotropy

stress ratio for these samples was about 2. The samples were normally consolidated to vertical effective stresses which were 2-4 times greater than the maximum past pressure. Failure in the CAUE tests was taken at 10% vertical strain because of the strain hardening behavior exhibited in these tests. As indicated by the data shown on Fig. 10, there is significant strength anisotropy which is expected for low plasticity soils. From the lower portion of the figure, it is evident that undrained strength anisotropy increases with decreasing plasticity due to increased strength in compression. This behavior is similar to that observed in CIU tests and is considered to reflect material type. There does not appear to be any effect of plasticity on CAUE strengths although it is appreciated that the CAUE data are limited in number and somewhat scattered.

Using the normalized soil property approach proposed by Ladd and Foott (1974), the undrained strength of the layer 4 soils can be described as follows:

$$Su = S.\sigma_{vo}'.OCR^m$$

where S =0.32-0.36 for compression (active), 0.18 (passive). Based on published data for other soils, m is taken as 0.8.

5 EFFECT OF LAYER 4 ON DIAPHRAGM WALL PERFORMANCE

As indicated on Fig. 2, the layer 4 soils will have a significant effect on the performance of most diaphragm walls for TRTS excavations because they frequently form at least the upper material below the excavation invert. It is the passive support provided by the material in this zone that has a major influence on the magnitude of wall movements.

In the prediction of wall movements and design of measures to control these movements, the correct definition of strength in the layer 4 soils is critical. The most important consideration in defining strength in this zone is the degree to which swelling will occur during construction. For typical TRTS structure geometry and a typical range of coefficient of swelling measured in oedometer tests, simple swelling analyses indicate that the degree of swelling of the layer 4 soils in the passive zone will likely cover the full range between 0 to 100%. Determination of passive resistance depending on degree of swelling is discussed below.

5.1 High degree of swelling

In these cases, the shear strength of the layer 4 soils is best characterized by effective stress strength parameters which can be readily defined. The problem then becomes definition of water pressures in the passive zone. The effect of variation in water pressure conditions in the passive zone on wall deformation is very significant as illustrated on Fig. 11. Building settlements associated with these different wall deflection profiles would also be very different. Thus pore pressures developed in the passive zone are a major factor in determining the need for building protection measures. Since swelling would be largely complete, water pressures in the passive zone would be the result of groundwater flow from materials directly below the excavation base and seepage under the diaphragm wall. Steady state conditions would represent the worst case (i.e. highest water pressures in the passive zone) and could be used for design. This assumes that as the excavation progresses, any excess water pressures which are induced by wall deformation, would be effectively dissipated. There is significant uncertainty associated with prediction of strain induced water pressure and its dissipation behavior. Further, given inevitable variation in stratigraphy together with flow through imperfect walls, prediction of steady state conditions is also uncertain. Given these uncertainties, the only way to ensure safe working conditions is to design based on realistic water pressure assumptions and to ensure that these assumptions are in fact realized in practice. The soil conditions which allow the development of swelling and seepage pressures should also allow the effective control of these pressures by pumping using well points or other measures.

Depending on local stratigraphy and excavation geometry, it is not always the case that the diaphragm wall will cut off into a continuous clayey layer to avoid lowering of groundwater levels outside the excavation due to implementation of groundwater control measures inside the excavation. It is inevitable that in some cases, groundwater lowering will occur outside the excavation. Unless this is controlled by recharging, it will result in consolidation settlement of the adjacent ground surface. Given the history of groundwater level changes in the Sungshan deposits, it is unlikely that large scale settlements will occur in the T2 area. For the K1 area, larger consolidation settlements can be expected.

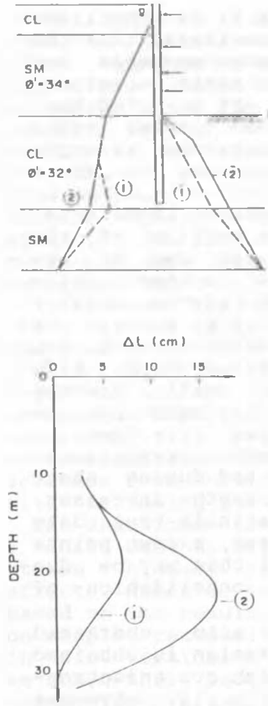

Figure 11. Effect of water pressures on wall deflection

5.2 Low degree of swelling

For areas underlain by deep plastic clays, little or no swelling may occur during construction. In this case, it is reasonable to use undrained strength parameters to calculate active and passive wall pressures. However, given the anisotropic nature of the layer 4 soils, CAUE and CAUC strengths should be used in the passive and active zones respectively. The use of undrained strengths, which automatically includes the porewater pressure at failure (at least for the laboratory stress paths), is probably realistic in the finer, more plastic soils given the relatively large strains which will occur.

The factors which result in a low degree of swelling will preclude the effective use of ground water control measures to reduce wall movements. It is known from previous experience, that large wall movements are likely in K1 areas. For deeper excavations in critical areas, large wall movements are likely to occur even if thick, heavily reinforced walls with heavy bracing and deep penetration are used. For these cases, measures such as jet grouted slabs or struts which are preinstalled below the excavation base, T-shaped wall sections and/or intermediate cross walls will be required to control deflections.

5.3 Partial swelling

Those situations where partial swelling is anticipated are most difficult to deal with. In theory, two approaches are possible:-

(1) Effective stress approach:- Effective stress strength properties can be readily

defined. However, a priori prediction of water pressure distribution in the passive zone is practically impossible because it requires the determination of the effects of swelling, excess pressures due to shear straining and the transient flow regime which is developing toward a steady state flow condition. Each of these processes is difficult to model independently; the problem becomes practically impossible when their inter-dependence is considered, together with the effect of even relatively minor variations from the assumed model stratigraphy.

(2) The problem can be simplified to some extent by using undrained strengths. Thus, the excess water pressures induced by wall deformation is equated to the porewater pressure at failure in the laboratory test which is probably not unreasonable. The problem then becomes one of coupled swelling and transient flow which is still extremely difficult to analyze reliably. However, for deep walls, deflection will be largely related to the strength of the materials immediately below the excavation depth where swelling is the controlling phenomenon. The reduction in undrained strength which accompanies swelling can be defined based on the SHANSEP approach as discussed previously.

Again even given the simplifying assumptions associated with this approach, there is still considerable uncertainty in the prediction of undrained strength because of the difficulty in accurately predicting. The effect of this uncertainty is significant in relation to wall deformation as illustrated on Fig. 12. Because of this uncertainty, porewater pressure measurement during construction is mandatory. The philosophy of ground deformation control in these cases is not as obvious as it is in the full swelling or no swelling cases. A conservative approach would dictate the same type of measures described for "low degree of swelling" cases. Depending on detailed stratigraphic variations, groundwater control measures may be equally practical.

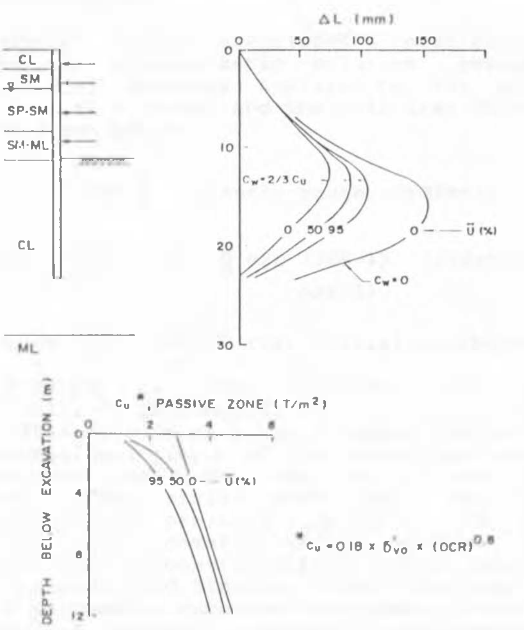

Figure 12. Effect of swelling and wall adhesion on predicted deformations

The example shown an Fig. 12 also demonstrates the major effect of wall adhesion assumptions on wall deflection for the case of no swelling. Given the magnitude of wall deflection it is considered inevitable that wall adhesion will be developed in the field. However, there is a paucity of data regarding the magnitude of adhesion which is developed at any stage despite its obvious importance in terms of wall performance.

6 CONCLUSIONS

(1) Accurate prediction of wall movements is necessary to assess effects of construction on adjacent properties.

(2) Care must be exercised in the direct extrapolation of performance data from previous excavations because of the significant differences which can exist between the conditions prevailing at the monitored locations and those which will prevail at proposed excavation locations.

(3) Prediction of the magnitude of wall deformation for the TRTS project depends to a large degree on reliable estimates of the strength properties of the layer 4 soils of the Sungshan formation. These materials are only lightly overconsolidated and exhibit a wide range in strength behavior from soft clay to loose-medium silty fine sand.

(4) The strength of the layer 4 soils depends on the degree of swelling which will occur during excavation which in turn depends on stratigraphy in relation to excavation geometry and the engineering characteristics of the materials. The full range from 0-100% swelling in layer 4 will occur across the TRTS project. Therefore different approaches to strength determination for design and ground deformation control measures will be required. Because of uncertainties in predictions of performance, an observational approach will be required during construction.

7 ACKNOWLEDGEMENTS

The authors wish to express their thanks to the Taipei Department of Rapid Transit Systems and in particular Dr. P. Lai for permission to publish some of the information included in this paper. The authors also thank their colleagues at Moh and Associates for their support in the preparation of this paper. Finally, the advice of C.C. Ladd in the development of the undrained strength data is appreciated.

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Discussion

T.B.CELESTINO, Themag Engenharia, São Paulo & São Carlos Engineering School, University of São Paulo, Brazil

INTRODUCTION

The soil loss volume (Vl) and the soil mass load on the support of tunnels (pi) are interconnected, but this dependency is not usually assessed unless complex analysis methods are used. Simple settlement prediction methods (e.g. Schmidt, 1968) assume an internal pressure acting on the excavation line and a relationship between settlement and pressure is given.

In these methods the pressure can be treated as a construction measure (e.g. compressed air) to decrease settlements, but there is no way of obtaining stabilized support pressure by these methods. On the other hand, simplified methods for obtaining loads on the support (e.g. Schwartz and Einstein, 1980) do not give any prediction of the soil loss. Coupling the two methods allows to back-analyze the suport pressure from results of soil loss or vice versa. The procedure presented here is valid for tunnels that cause negligeble volume change of soil during construction (e.g. tunnels in clays).

DESCRIPTION

Schmidt (1969) established a relationship based on an elastoplastic solution between the internal pressure applieed to the excavation line of a tunnel and the soil loss volume which is given below:

$$Vl = 3\ OFS\ \frac{c}{E} \quad \text{(elastic range, } 0 \leq OFM < 1) \tag{1a}$$

$$Vl = \frac{A}{1+A}\ ,\ A = 3\ \frac{c}{E}\ \exp\ (OFS-1) \quad \text{(plastic range, } OFS \geq 1) \tag{1b}$$

where $OFS = \frac{\gamma z - pi}{c}$ (γz: initial overburden pressure; c: shear strength) and E is the modulus of deformability.

This relationship can be understood as a non-dimensional form of the so-called soil mass characteristc curve (Fig. 2). It was obtained for plane strain conditions. So, if the equilibrium pressure applied by the support, away from excavation effects is used, the soil loss due to constructuction will be obtained.

Schwartz and Einstein (1980) obtained by means of parametric numerical analyses a factor λd to correct support internal forces obtained from elastic, plane strain conditions for the 3-dimensional effect resulting from the fact that the support is installed at a distance Ld behind the face.

Their relationship is:

$$\lambda d = 0{,}98 - 0{,}57\ Ld/R \tag{2}$$

where R is the tunnel radius.

If the non-dimensional soil characteristic curve is known (equation 1) the value of the stabilized soil loss will correspond to the pressure on the support and the pressure on the cavity (equal after equilibrium, or stabilized conditions are reached.). The support characteric curve can be drawn through that point, and the value, Vlo of the soil loss when the support was initially installed (with no load) is obtained. Schartz and Einstein (1980) also show that the correction factor λd can also be expresse as:

$$\lambda\ d = \frac{Vlf - Vlo}{Vlf} \tag{3}$$

where Vlf is the fictitious final soil loss if the mass had linear elastic behavior throughout the full unload range.

CASE HISTORY

A double-track tunnel with approximately 10m diameter excavated in tertiary fissured clay for the São Paulo subway with overburden varying from 8 to 15 m was taken for the back-analysis. Settlement values and other details were given by Celestino et al. (1985). Rearranging equation 1b:

$$Vl = 3\ \frac{c}{E}\ e^{-(\frac{Pi}{c} + 1)}\ e^{\frac{\gamma z}{c}} \tag{4}$$

If the support pressure pi is first assumed not to vary substantially with the overburden, Vl versus logarithm of overburden (or Z/2R in non-dimensional form) will plot as straight line. The actual values are shown in Figure 1 which shows a very good agreement. From that figure it can be obtained that:

$$\frac{2R\gamma}{c} = 2{,}97,\ \text{or } c = 67\text{KPa} \tag{5}$$

Lateral extension, consolidated-undrained tests for the confining pressure corresponding to the overburden pressure gave about 110 KPa for the shear strength. Bacause the mass was very fractured, the back-analyzed value seems to be in agreement with laboratory values.

If the known value for E = 80 MPa (obtained from other instrumentation back-analyses) is substituted into equation 4, pi = 170 KPa is obtained. This value is in very good agreement with non-linear finite element alayses simulating the construction of the tunnel (Domingues, 1985).

The non-dimensional characteristc curves for the mass and the support are shown in Figure 2. Values for Vlo and Vlf obtained there are 0,40 and 0,55 respectively. For these values, equation 3 gives $\lambda d = 0{,}27$, and substituting this value into equation 2 leads to Ld/R = 1,2.

It is interesting to note that the support characteristc line shown in Figure 2 would be true is the support had been installed as cured, stiff shotcrete. As this was not the case the

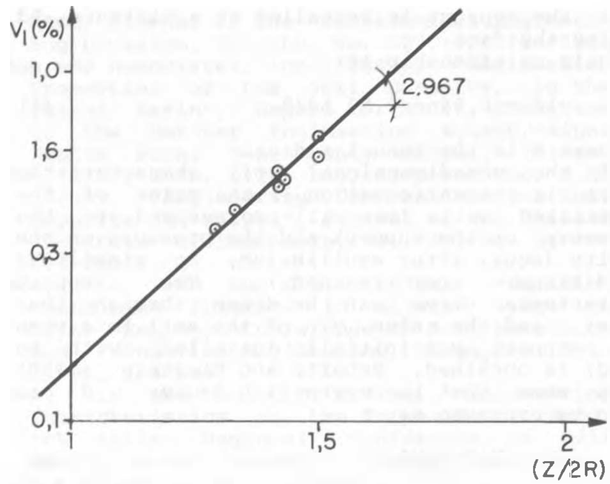

Figure 1 - Variation of soil loss with overburden.

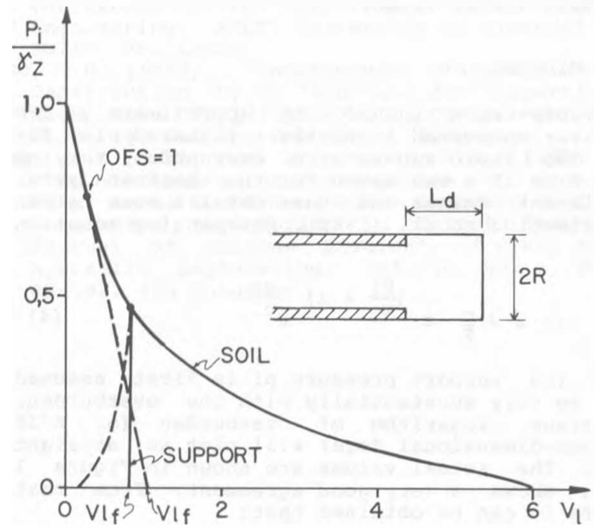

Figure 2 - Characteristic curves for the soil mass and the support.

actual dashed curve is shown resembling qualitatively the progressive stiffening of the shotcrete.

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Discussion

B. LADANYI, Ecole Polytechnique, Montréal, Canada

I have been sitting here for the last two hours, listening to a number of most interesting contributions to the subject matter. Many subjects were covered in the discussion, but, surprisingly, an important aspect of the problem has not even been mentioned up to now. A missing piece in this discussion was, in my opinion, the determination of soil or rock parameters that would help evaluating the delayed response of the ground to the stress change produced by underground excavation and lining installation.

In spite of ample experimental evidence and field experience, both before and after the well-known Terzaghi's (1946) paper on loads on tunnel supports, in which he clearly defines "squeezing" and "swelling" rock, it seems that the majority of efforts in soil mechanics literature connected with soft ground tunnelling, have been directed towards solving the problem of instantaneous elastic-plastic response of the ground and its subsequent consolidation around the opening.

However, a delayed response, not necessarily connected with the consolidation, but rather due to a deviatoric creep has often been observed in the field both in weak rocks and in clays under undrained conditions (e.g., Ward, 1978; Ladanyi, 1981). In addition, it is known that, even in granular materials and in rock masses, the process of stress redistribution continues around the opening for some time after excavation and lining installation, either for rheological reasons (e.g., Lo and Yuen, 1981; Ladanyi and Gill, 1988), or simply because of changes in ground properties with time (e.g., Ladanyi, 1974).

Determination of parameters characterizing the long-term behaviour of geomaterials seems to me to be an essential part of the parameter selection for underground construction.

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Discussion

BIRGER SCHMIDT, Parsons Brinckerhoff, San Francisco, Calif., USA

The discussion leader (R. J. Mair) in his introduction noted that the ground loss experienced during tunnelling in clay is often less than that predicted by theoretical means (e.g., Clough and Schmidt, 1981). In searching for a reason, he inquired whether the theoretical prediction might, in fact, be invalid for shallow tunnels, where the proximity of the ground surface changes the value of the stability number or overload factor for which the tunnel remains stable. The discusser offers the following comments in this regard.

The theoretical ground loss prediction (Clough and Schmidt, 1981) is based on an elasto-plastic continuum analysis that assumes axi-symmetric conditions; i.e., the effects of the depth of the tunnel and of the typical strength increase with depth are ignored. Analyses show that the plastic zone gets quite large for high values of the overload factor; for shallow tunnels, the plastic zone can easily reach the ground surface. For this reason, the analysis cannot be expected to give accurate results for shallow tunnels. However, this is probably not the principal reason for the discrepancy.

With modern shield tunneling techniques using face pressure, ground movements towards the tunnel face can generally be controlled, and the maximum possible ground loss is limited to the volume of the tail void left behind the lining as the shield advances. For large overload factors, this volume is substantially smaller than that predicted by the theoretical analysis, so it is to be expected that the actual ground loss is less than that predicted by the theoretical analyses. In fact, the theoretical analyses, whether by simple closed solutions such as Clough and Schmidt (1981) or by complex computer analyses, attempt only to predict the ground loss expected if there is no restraint to the radial movement of the soil towards the tunnel; the upper limit is, in fact, determined by the means taken to restrict face and tail void soil movements.

For practical construction reasons, the tail void thickness tends to be similar, regardless of the finished diameter of the tunnel. Therefore, the tail void volume and, hence, the maximum ground loss, as a percentage of the total excavated tunnel volume, would tend to be lower for larger tunnel diameters, as is typically found.

Much of the presentation by the General Reporters for this Session was based on work by one of the Reporters, C. Sagaseta, in part presented in Geotechnique (Sagaseta, 1987). Analyses made by the discusser (Schmidt, 1988) indicate that the proposed method for predicting the shape of the settlement trough due to tunneling generally produces troughs much wider and shallower than actually measured in practice. Hence, the proposed method is not conservative and cannot be recommended.

The discussion leader inquired about the parameters required for assessing long term settlements due to consolidation of soft clay around tunnels. Readers are referred to the paper presented by this discusser for this Session on the topic. It is concluded that knowledge of the factors making up the overload factor (undrained shear strength, overburden pressure, and internal support pressure), the pore pressure parameter A, and construction parameters (face pressure, tail void displacement allowed), will permit a prediction of the occurrence of long-term consolidation settlements. The next step is the prediction of consolidation settlements and a confirmation of the method by actual field measurements.

As a final note, it is the opinion of this discusser that much attention has been paid to theories of ground behavior and ground-lining interaction, with many valuable results, but that many investigators have ignored essential effects of the details of the construction process. Future work should concentrate on two areas of development: A detailed understanding of the stresses and strains suffered by the soil adjacent to the tunnel during the phases of construction, and their effects on long-term settlements and lining loads; and a better methodology for assessing the effects of specific construction details on ground movements and soil-lining interactions. The practical and theoretical ramifications of a new emerging technology, sequential construction of tunnels in soil - by some termed New Austrian Tunneling Method, NATM - also require the attention of researchers and practitioners.

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Discussion

G.GUDEHUS, Institute of Soil Mechanics & Rock Mechanics,
University of Karlsruhe, Germany

Some special problems arise with tunnels in very soft saturated soil. Typically such soils have a sandwich structure of soft clay and very loose silt and fine sand. According to our field and laboratory experience, such soils are underconsolidated due to a stagnation gradient i_o, and therefore have excess pore pressure from the beginning. They also show high contractancy and are therefore prone to collapse and liquefaction.

These properties have the following consequences for tunnelling. Support of groundwater by compressed air produces unsaturated zones of rather uncontrolled extension. These zones collapse into denser packing after flooding. Inclusions of soft clay and also of sand are not even stabilized by bentonite suspension. Any even moderate shear, such as produced by front cutting and wall shift, produces pore pressure increase and subsequent settlement. Long term settlements due to drainage by the tunnel structure are almost inevitable.

Discussion

R. J. JARDINE, Department of Civil Engineering, Imperial College, London, UK

The General Report includes a discussion on analyses of tunnels and deep excavations in which the soil behaviour is assumed to be linear isotropic, or cross-anisotropic, elastic. I would like to make some comments concerning this assumption and the choice of appropriate stiffness parameters.

Firstly, recent advances in soil testing techniques have demonstrated that the stress-strain relations of most soils, including heavily overconsolidated stiff clays, cannot be considered to be linear or elastic at working stresses; Jardine, Symes and Burland (1984). This observation has been reinforced at this conference by Yubdir and Wood (1989).

Field measurements, such as those discussed by the Reporters, show that in most cases the mass of soil surrounding excavations and tunnels experiences relatively small strains as a result of construction; St John (1975), Jardine et al (1986). Analyses indicate that the soil-structure interaction is principally controlled by two non-linear aspects of soil behaviour;

(i) the stiffness characteristics of the soil mass at shear and volumetric strains between 0.002% and 0.2%, and

(ii) the large strain plastic response of soil within restricted zones of local yield.

Dealing with the plastic behaviour first, Figures 1 and 2 show the stress-strain and stress-path behaviour of a normally consolidated low plasticity clay tested in undrained triaxial compression and extension. The peak

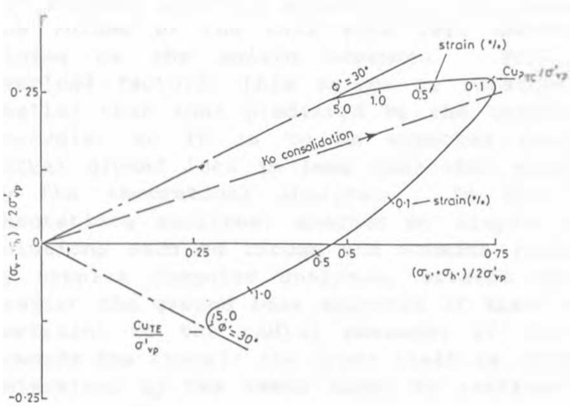

Figure 1. Stress-paths for a Ko consolidated low plasticity clay tested in undrained extension and compression from OCR = 1; after Gens (1982).

strengths are completely different - but this is not the only important factor. The compression test reaches failure at very small strains (<0.1%) and strain softens post-peak, whilst the extension test is ductile and develops large plastic strains before reaching its peak deviator condition (at 12% strain). The pore pressures induced by shearing are also completely different in the two cases. Features such as these could have a marked effect on the behaviour of tunnels and excavations in soft clays.

Moving on to the question of ground stiffness at small strains, it has been conventional to assume that soil stiffness cannot be measured reliably in the laboratory. Elastic constants are usually evaluated from rough rules of thumb, site specific experience or linear interpretations of in situ tests; see for example Poulos (1989). However, when local strain measuring transducers are used to overcome laboratory experimental errors, the reasons for the usual disparity between field and laboratory parameters become clear.

This is illustrated in Figure 3 using data from tests on London clay. These show the significance of errors in conventional external measurements (due to bedding, non-central loading, sample tilting etc) on the early parts of the soil's stress strain curve. The local measurements show that the initial stress-strain behaviour is in fact both far stiffer and more non-linear than is usually appreciated. This is further emphasized in the secant

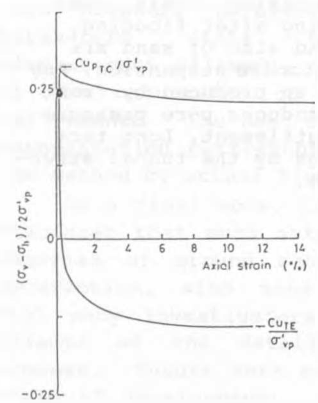

Figure 2. Stress-strain curves for a Ko consolidated low plasticity clay tested in undrained extension and compression from OCR = 1; after Gens (1982).

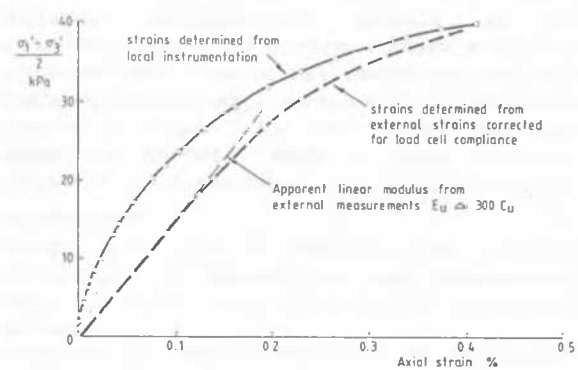

Figure 3. Stress-strain curves from UU tests on London clay - local and external strain measurements; after Jardine et al (1985).

stiffness-strain plot deduced from the same data - which is shown on Figure 4.

Suites of such tests have been performed at Imperial College to investigate the effects of Ko consolidation, OCR, stress level, ageing, loading stress-path and sampling on the stiffness characteristics of a range of soils. Insitu experiments have also been used to confirm that the laboratory characteristics reflect behaviour in the field; Jardine et al (1985).

Turning to anisotropy at small strains, Figure 5 presents the normalised undrained secant stiffness characteristics found for Ko consolidated London Clay at two OCR's. The monotonically overconsolidated samples show a markedly different pattern to that assumed conventionally. London Clay is usually thought to have higher horizontal stiffness than vertical, but in these tests the measured undrained compression stiffnesses were approximately twice those found in extension. This disparity has been explained by noting that the small strain characteristics of an element are affected by its recent stress history and may be completely realigned by even perfect sampling procedures; Jardine (1985), Hight, Gens and Jardine (1985).

This finding has many practical implications. For example, unconsolidated tests performed on perfect block samples will show the opposite anisotropy to the in situ characteristics of monotonically overconsolidated soils. It is also true that reloading an overconsolidated deposit (by man or nature) can cause further changes in the material's characteristics, with a tendency for compression stiffness to fall and extension stiffness to increase.

To obtain representative stiffness parameters it is

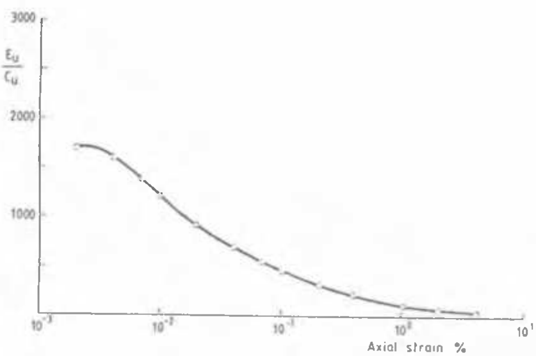

Figure 4. Secant stiffness-strain plot from data shown in Figure 3.

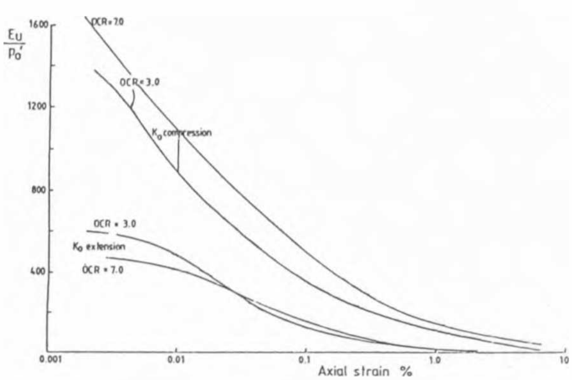

Figure 5. Stiffness characteristics of Ko consolidated London clay at OCR's 3 and 7.

therefore necessary to reconsolidate to Ko conditions following a path that retraces the most recent steps of the layer's (probably anisotropic) stress history. This observation throws into doubt most of the procedures used in practice, particularly those employed to estimate the cross anisotropic parameter, G_{vh}.

Independent field measurements, reported by Jardine et al (1985), suggest that the in situ variation of G_{vh} with shear strain (for London clay) is similar to the triaxial compression stiffness characteristic shown in Figure 3.

Indeed, when the curves were plotted as G_{secant}/p' against $\gamma_{octahedral}$ there was close agreement - suggesting that it may be reasonable, as a first estimate, to assume $G_{vh} = Euv/3$.

Analyses have been carried out at Imperial college to assess the effects of such steep non-linear soil stiffness characteristics on a range of geotechnical problems. Curves have been specified to describe the normalised shear and bulk modulus characteristics of different materials at small strains and these have been incorporated into existing elasto-plastic soil models. Comparisons between predictions and field behaviour indicate that the approach is at least as successful as conventional methods in predicting centre line displacements; Fourie, Potts and Jardine (1986), Jardine and Potts (1989). More importantly, considerable improvements can also be made in the prediction of other features - such as the distributions of earth pressures applied to any structural elements (eg a tunnel lining), the fields of induced pore water pressure change and the profiles of nearby surface settlement.

I further suggest that stress-strain non-linearity may be the principal reason for the 'anomalous' tendency for the widths of tunnel surface settlement troughs to be narrower than predicted conventionally. This explanation appears to be more satisfactory than the argument put forward concerning the ratios of notional, linear, cross-anisotropic soil moduli.

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