

DIAPHRAGM WALL DESIGN CONSIDERATIONS FOR THE TAIPEI SUBWAY

by

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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 follows:

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 that there will be numerous cases where deep

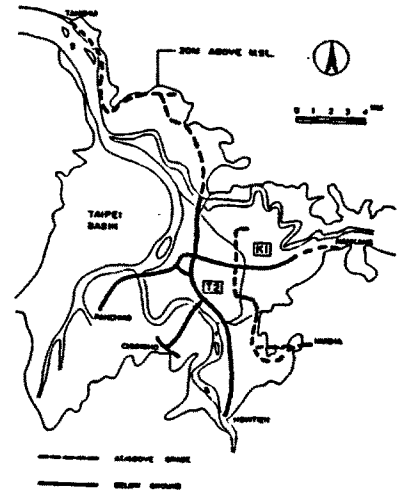


Figure 1. Taipei Rapid Transit System

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 direc-

tions. 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.

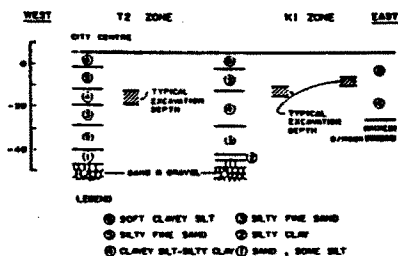


Figure 2. Typical east-west section showing simplified stratigraphy

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 much 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 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

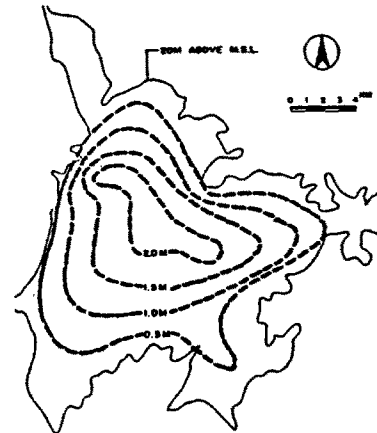


Figure 3. Contours of ground subsidence (metres) between 1955 - 1986

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.

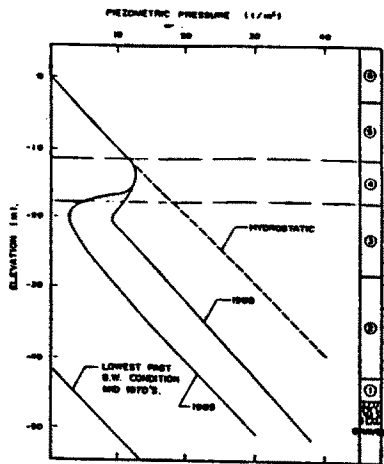


Figure 4. Changes in ground water conditions - city center area

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 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.

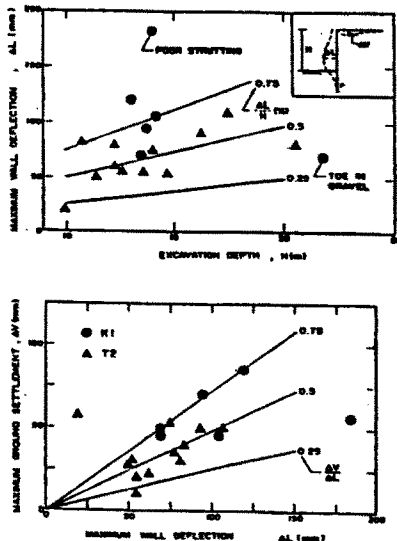


Figure 5. Deformations of diaphragm wall in Taipei

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 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

	MORE SILTY	MORE CLAYEY
WATER CONTENT (%)	20	40
I_p (CU)	10	20
D_{60} (mm)	0.05	0.005
K (cm/sec)	10^{-5}	10^{-7}
e_c SWELLING (cm ³ /sec)	3×10^{-2}	8×10^{-2}
C_c	0.2	0.4
ϕ' (DEG)	33	28

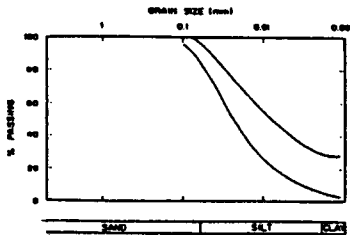


Figure 6. Typical engineering properties - layer 4

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 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

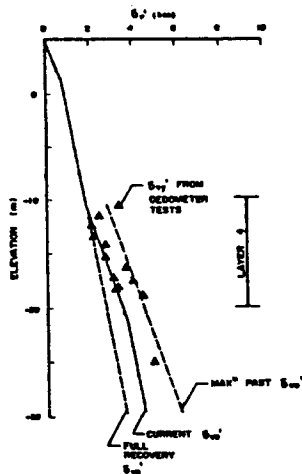


Figure 7. Typical stress history profile - layer 4

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 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.

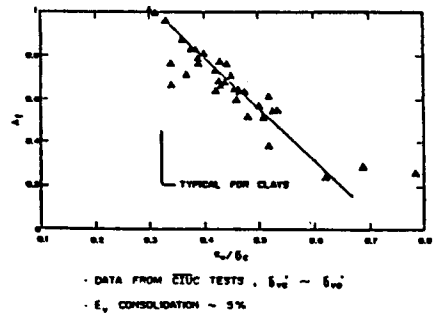


Figure 8. Undrained strength and PWP behaviour - layer 4

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} in-

creases 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.

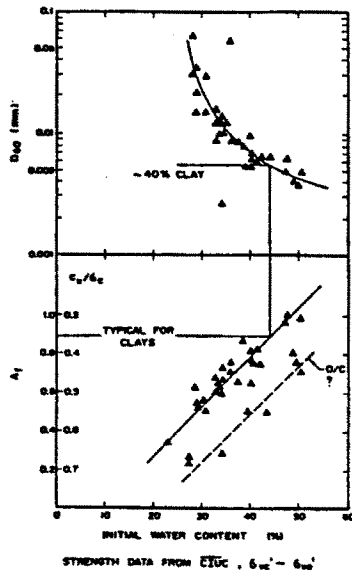


Figure 9. Dependence of strength and P.W.P. behaviour on material type - layer 4

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 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:

$$S_u = S \cdot \sigma_{vo}' \cdot OCR^m$$

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.

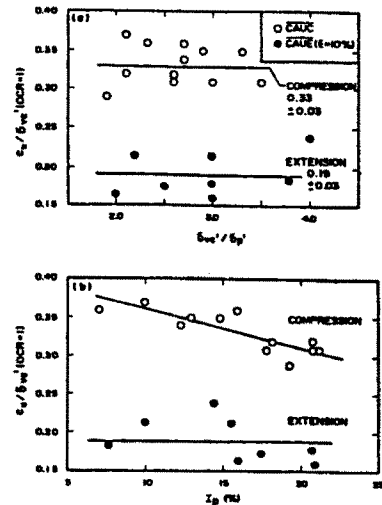


Figure 10. Undrained strength anisotropy

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.

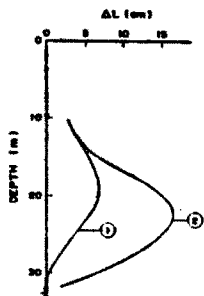
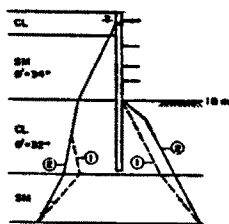


Figure 11. Effect of water pressures on wall deflection

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.

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 pre-installed 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 meas-

urement 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.

The example shown in 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.

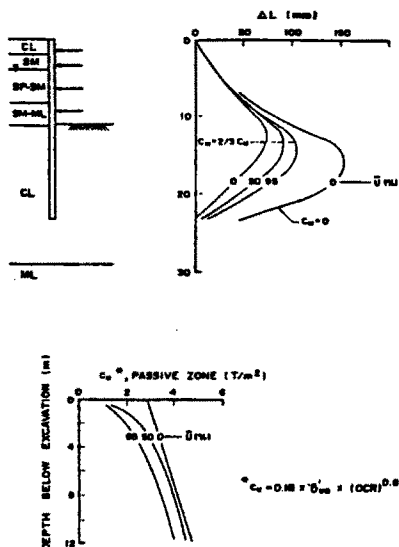


Figure 12. Effect of swelling and wall adhesion on predicted deformations

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

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