Parameters for predicting deformations due to tunnelling

Paramètres pour la prédiction des déformations lors de la construction des tunnels

R.K.ROWE, Geotechnical Research Centre, University of Western Ontario, London, Canada
K.M.LEE, Geotechnical Research Centre, University of Western Ontario, London, Canada

SYNOPSIS: The implications of stress path with respect to the selection of elastic and plastic parameters are discussed. The importance of each soil parameter is evaluated and an approximate scheme for determining soil parameters suitable for use in the preliminary analysis of deformations caused by tunnelling in soft clays is suggested.

1. INTRODUCTION

Finite element techniques have been successfully used to predict the deformations of the ground surface due to the construction of a tunnel. However, the relevance of deformation calculations using these techniques is dependent upon the reliability and appropriateness of the input parameters. In the determination of soil parameters, it is essential to consider the stress path experienced by the soil during tunnel excavation and erection of the lining. The object of this present paper is to discuss the implications of stress path upon the selection of soil parameters for use in predicting deformations caused by tunnelling.

2. STRESS PATHS IN TUNNEL EXCAVATION

It has been recognized (Lo and Rowe, 1982; Ng, 1984) that the stress path experienced by the soil during tunnel excavation and lining erection is quite different from that experienced by a soil sample during a conventional triaxial compression test. The stress path followed by a soil element depends both on its position and the details of the construction procedure. In general, it is not possible to precisely define this stress path for practical problems; however, both theoretical analyses and field observation indicate that the stress relief around the tunnel boundary would result in "unloading" and extensive stress reorientation of the soil mass around the tunnel opening (Lee and Rowe, 1988a). The general trend of the stress path above the tunnel, therefore, would be closer to the triaxial test conducted in "extension" mode than in "compression" mode and consequently parameters determined from conventional triaxial compression tests may not be appropriate in the critical region above the tunnel.

3. SELECTION OF DESIGN PARAMETERS

In most soft ground tunnelling there will be significant plastic deformation within the soil. Thus, modelling the soil response involves selection of both the correct elastic parameters as well as providing a reasonable model of the plastic behaviour.

Since non-linearity, stress level and anisotropy play important roles in stress path dependency, the physical aspects of these effects will be examined.

3.1 Deformation parameters

3.1.1 Effect of stress path on the elastic modulus

Lo and Rowe (1982), Ng (1984) and Muckle (1984) have considered the implication of stress path upon the selection of modulus parameters for the soil adjacent to the Thunder Bay Sewer Tunnel (see Lo and Rowe, 1982). Conventional CK_0E and CK_0C triaxial tests were performed on good quality samples in both the extension and compression modes. It was found that in the extension tests, the stress-strain behaviour for both the drained and undrained tests were virtually identical and consequently the resulting elastic moduli are virtually identical (this can be explained by the effect of Poisson's ratio as discussed in the following section); however, very different responses were observed for the tests in the compression mode. Furthermore, a series of special "stress path tests" were also performed (see Ng, 1984 for details) corresponding to the stress path expected above the crown (denoted as "extension mode") and at the springline (denoted as "compression mode"). These results are summarized in Table 1. It is noted that in extension, Young's modulus determined from the stress path tests is approximately the same as that obtained from CK_0E tests. This is primarily because the stress paths are approximately the same in these tests. However, in compression, there is a significant difference between the stress paths followed in these tests and the test corresponding to the drained stress path test gave a modulus (i.e. 9.5 MPa) intermediate between the conventional drained and undrained values (i.e., between CK_0DC of 6.1 MPa and CK_0UC of 16 MPa).

From these results, it is clear that the stress path can have a substantial effect on the modulus. For predicting final equilibrium settlement, ideally, the deformation parameters should be evaluated from stress path tests by following field stress paths typical of that expected at the crown and the springline region. However, for most practical situations where

Table 1. Comparison of moduli from different tvpes of tests (after Ng, 1984).

Type of Test	Modulus			
	Extension		Compression	
	E_u	E'	E_u	E'
Servo	11.6	11.6	-	-
Total Stress Controlled	11.8	11.8	16.4*	9.5*
CK_oE or CK_oC	11.3	11.3	16	6.1*

*Note: There is a significant difference between the stress paths followed in these tests.

there is limited data, an approximate value of elastic modulus obtained by CK_oUE test can be adopted. Where only the result of a conventional CID compression test is available, a correction factor (usually greater than 1.0) should be applied.

3.1.2 Anisotropic elastic parameters

Lo et al. (1977) describe a technique which was developed for the determination of the anisotropic elastic properties of soils. This approach was used by Ng (1984) and Yuen (1976) to obtain results for three clayey soils, and the results are summarized in Table 2. Examination of these results indicates that:

i) The elastic modulus (both vertical and horizontal) in unloading are consistently higher than the loading modulus. The ratio of unloading to loading modulus for all three types of soils was approximately two.

ii) The modulus ratio E_h/E_v is lower in the Leda clay (E_h/E_v = 0.55, E^*_h/E^*_v = 0.67; note: * denotes unloading) than the other two clays.

Table 2. Anisotropic elastic parameters from CID tests for loading and unloading condition (based on data from Ng and Lo, 1986; Yuen, 1976)

Anisotropic Elastic Parameters	Thunder Bay Silty Clay Depth 10.15m	Thunder Bay Varved Clay Depth 13.4m	Gloucester Leda Clay Depth 2.4m
E_v (MPa)	6.5	10.1	4.7
E^*_v (MPa)	12.1	23.4	9.7
E_h (MPa)	5.3	6.8	2.6
E^*_h (MPa)	10.9	15.8	6.5
G_{hh} (MPa)	2.3	3.0	1.2
G^*_{hh} (MPa)	3.8	5.9	2.5
G_{vh} (MPa)	2.7	4.3	2.1
G^*_{vh} (MPa)	3.3	4.7	2.7
ν_{hh}	.17	.13	0.10
ν_{vh}	.12	.07	0.08
ν_{hv}	.15	.11	0.08
ν^*_{hh}	.45	.33	0.29
ν^*_{hv}	.41	.20	0.23
ν^*_{vh}	.46	.30	0.30
E_h/E_v	.81	.67	0.55
E^*_h/E^*_v	.90	.68	0.67
G_{vh}/E_v	.42	.43	0.45
G^*_{vh}/E^*_v	.27	.20	0.28
E^*_v/E_v	1.9	2.3	2.1
E^*_h/E_h	2.1	2.3	2.5

Note: * denotes unloading.

This indicates that the deformation behaviour of the Leda clay is more anisotropic. However, the sensitivity study performed by Lee and Rowe (1988a) suggests that even this degree of anisotropy will not significantly affect prediction of deformations (this will be discussed later).

iii) There is a significant difference between the Poisson's ratios measured in compression and extension. In particular, the Poisson's ratio in extension is significantly higher than that in compression and hence one would not expect much volume change due to dissipation of excess pore pressures in undisturbed soil above the tunnel. This probably explains the almost identical stress-strain responses of the CK_oUE and CK_oDE tests discussed in the previous section.

iv) The independent shear modulus determined in extension and compression only differed by a maximum of 35% for the Leda clay and less than 20% for the Thunder Bay clays, which indicates that G_{vh} is relatively insensitive to stress path (i.e., unlike Young's modulus). This finding is confirmed by the values of G_{vh} obtained independently using the Norwegian simple shear apparatus (Lo and Rowe, 1982). As a consequence, the ratio of G_{vh}/E_v differs significantly between compression and extension (with G^*_{vh}/E^*_v being significantly smaller than G_{vh}/E_v). This is important because 1) the independent shear modulus has an appreciable effect on the shape of the settlement trough (this will be discussed in more detail later); and 2) the assumption of isotropy for unloading conditions (i.e. assuming $G^*_{vh}/E^*_v = 0.5/(1+\nu)$) would greatly overestimate the independent shear modulus (G^*_{vh}).

3.1.3 Finite element analysis

Rowe et al. (1983) performed a sensitivity study for a shallow tunnel constructed in clay using a closed face tunnelling machine. This study showed that:

1) Stress relief due to tunnelling will give rise to a predicted heave of underlying soil. If the correct soil parameters are adopted this predicted heave will be small (as it usually is in practice). However, careful attention must be given to the selection of an appropriate modulus for the soil below the tunnel. The adoption of a modulus that is too low will result in a large heave and an underestimate of the surface settlement.

2) In cases where there is only moderate loss of ground, subsidence will be largely governed by plastic deformation above the tunnel. Thus, provided the elastic parameters are not unreasonable, a precise determination of these parameters is not generally necessary. In this regard, it may be better to err by overestimating rather than underestimating the bulk modulus provided that a reasonable shear modulus is adopted.

Elasto-plastic finite element analyses were also performed by Lee and Rowe (1988a) to determine the effect of elastic anisotropy on the surface settlements caused by tunnelling. This study showed that:

1) The effects of varying the ratio of horizontal modulus to vertical modulus (E_h/E_v) and of varying Poisson's ratio ν_{hh}, ν_{vh} were examined and it is found that, in general, anisotropy in terms of these parameters (i.e., for E_h/E_v<1.5 and $\nu_{hh} \neq \nu_{vh}$) does not greatly affect the surface settlement for typical ranges of these parameters.

2) The effects of G_{vh} on the surface

settlements caused by (a) a surface loading, and (b) tunnelling are shown in Fig. 1. The results are presented in terms of the ratio (R_a) of the maximum surface settlement obtained for a given ratio of G_{vh}/E_v divided by the value obtained for an isotropic soil. It can be seen that for G_{vh}/E_v in the typical range from 0.2 to 0.4, for the case of surface loading, the effect of neglecting anisotropy is quite small (i.e., less than 15%). However, for the case of tunnelling, the settlement may vary significantly (i.e., by up to 70%) from that obtained assuming isotropy, and the deviation from isotropy increases for lower G_{vh}/E_v ratios. This is particularly important since, as discussed earlier, the ratio of G_{vh}/E_v under unloading condition for most soils is lower than the isotropic value of $G_{vh}/E_v = 0.5/(1+\nu)$. Lee and Rowe (1988a) did a finite element analysis on the results of a centrifugal model test conducted at Cambridge University (Mair et al., 1981). It is found that the independent shear modulus has a significant effect upon the shape of the settlement trough and that reasonable agreement between observation and theory is obtained for G^*_{vh}/E^*_v between 0.2 and 0.25. This range of values is consistent with the values shown in Table 2. It is therefore concluded that when attempting to predict settlement induced by tunnelling, attention should be given to the effect of elastic anisotropy and, in particular, to the ratio of the independent shear modulus G^*_{vh} to vertical modulus E^*_v.

3.2 Strength parameters

3.2.1 Effective stress parameters (c' and ϕ')

A study on the effects of test type and stress paths on the effective failure envelope has been conducted by Ng (1984) and Muckle (1984) on the Thunder Bay silty clay. When plotted together, the results obtained from conventional CID, CK_oD, CK_oU and special stress path tests each fell close to the same envelope, indicating that the strength envelope is not sensitive to test type or test path employed. The failure conditions for the "extension" tests also fall in close proximity to the mirror image of the compression failure envelope. It is apparent that the effective failure envelope of this elastically anisotropic soil is essentially isotropic and is not significantly affected by the test type and stress path.

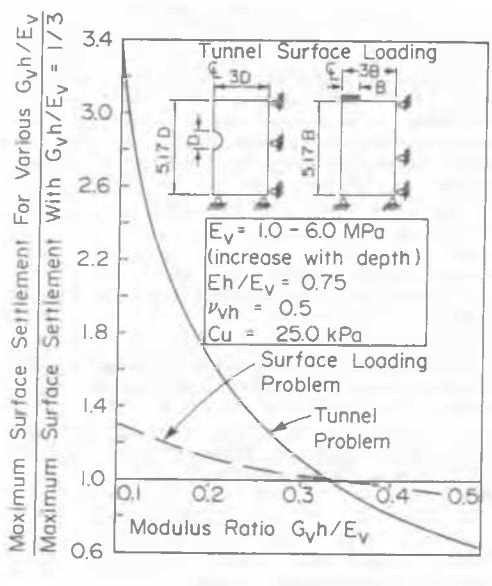

Figure 1. Effect of modulus ratio on settlements

In spite of the ambiguity which sometimes exists in the computation of effective stress parameters, literature suggests that anisotropy in terms of c' and ϕ' is more likely to be encountered in macro-anisotropic soils (e.g. stratified Welland clay; Conlon et al., 1971) or soils having complex microstructures (e.g. anisotropic Winnipeg clay; Freeman and Sutherland, 1974) and in the case of sensitive marine clays (Lo and Morin, 1972). Although some particle parallelism is likely to exist in homogeneous clays and some particle alignment during shearing, the effective strength parameters (especially ϕ') tend to exhibit only modest anisotropy with differences in the order of less than 3 degrees as observed by Muckle (1984).

3.2.2 Undrained shear strength (c_u)

In soft ground tunnelling, the determination of undrained shear strength is mainly associated with the establishment of the stability ratio,N. This ratio is used as an "index parameter" to aid in selection of an appropriate tunnelling technique (e.g. type of shield or use of compressed air).

The undrained shear strength is also required for predicting deformations. Irrespective of methods of ground control that are used, some three-dimensional movements at and in front of the face usually take place during excavation of the working face. Since these three-dimensional soil movements will develop at a distance of not more than a few tunnel radii away from the face of the tunnel, for soft clays these movements may often be considered to be undrained. Similarly, movement of soil into the tailpiece void of the tunnelling machine also often occurs under essentially undrained conditions. A method of estimating the three-dimensional soil displacements ahead and behind the tunnel machine is currently being developed by Lee (1988). Based on limited available data and the results of theoretical studies it would appear that for a shield driven tunnel with moderate value of "face loss", a reasonable estimate of the undrained shear strength can often be obtained from field vane tests (exceptions arise out of the discussion below).

3.2.3 Effect of strength anisotropy on surface subsidence

Elasto-plastic finite element analyses, which incorporate the anisotropic strength theory proposed by Davis and Christian (1971), were performed by Lee and Rowe (1988b) to determine the effect of strength anisotropy on the ground deformation. Based on a survey of published data, the authors classified the undrained anisotropy into three basic categories: "M-anisotropy", where the horizontal strength c_{uh} is the minimum strength (compared with any other direction); "C-anisotropy", where c_{uh} is the maximum strength; and "K-anisotropy", where the minimum strength is obtained at an orientation other than vertical or horizontal (typically it is about 45°). It is found that, for a lined tunnel, the effect of strength anisotropy upon the surface settlement profile will depend upon the size of a so-called "gap" parameter. In a plane strain finite element analysis, the "gap" represents the net effect of loss of ground and some "workmanship" factors (see Rowe et al., 1983; and Lo et al., 1984 for details). Increasing the "gap" eases the restrictions imposed by the tunnel lining upon possible soil deformations, thereby increasing the effect of strength anisotropy. However, it is shown that for a lined tunnel with a moderate value of "gap", the detailed consideration of strength anisotropy may not be necessary. On the other hand, for the case of an unlined tunnel or large loss of ground, attention should be given to the effect of strength anisotropy particularly for a soil possessing type "K-anisotropy". For this type of anisotropy, the vane test may not provide sufficient and adequate data for predicting deformation or stability.

4 SUMMARY AND CONCLUSIONS

In numerous cases analyzed by the authors and co-workers (Lee and Rowe, 1988a; Ng, 1984; Rowe and Kack, 1983), the use of appropriate elastic and strength parameters determined for the expected stress path greatly improved the shape of the calculated surface settlement trough as well as the lateral deformations. Although it is always desirable to determine all parameters from actual tests, the authors have found that in many cases an approximate scheme of determining soil parameters for preliminary design analysis of tunnelling in soft clays may be suggested:

1. Determination of soil stratigraphy and general soil properties by conventional methods for an overview of soil behaviour. In addition, the initial state of stresses (i.e., K_o) should be estimated (in many cases empirical relationships may be adequate).
2. Determination of a continuous undrained shear strength profile with depth (e.g. using the field vane test). This information will facilitate the choice of tunnelling technique and the estimate of the "loss of ground" ahead of the tunnel face (Lee, 1988).
3. The elastic parameters under "extension" mode may be evaluated from CK_oUE tests. Where only the result of conventional CID compression tests are available, the vertical modulus in extension can, as a first approximation, be taken to be twice the modulus in compression (i.e., $E_v^* \approx 2\ E_v$).
4. Independent shear modulus, G_{vh}, can be approximately obtained using drained triaxial tests in compression to determine E_v and ν and then assuming isotropy to obtain G_{vh} (i.e. assuming G_{vh} in extension is approximately equal to that in compression, $G_{vh}^* \approx G_{vh} \approx 0.5\ E_v/(1+\nu)$).
5. Determination of the effective stress failure envelope by CIU tests is desirable. It is often sufficient to assume that it is isotropic.

It should be noted that the variation of the required parameters with depth should also be established.

Clearly, care is required in applying any generalized approximations such as those noted above. The appropriateness of parameters for any given project must be assessed by a qualified geotechnical engineer. For important projects, (i.e. projects where the consequences of excessive deformation are high), any preliminary estimate of parameters should be confirmed by appropriate testing.

5 ACKNOWLEDGEMENT

This paper represents part of a general programme of research into soft ground tunnelling being funded by grant A1007 of the Natural Science and Engineering Research Council of Canada awarded to the senior author.

6 REFERENCES

Conlon, R.J. and Tanner, R.G. 1971. The geotechnical design of the Townline Road rail tunnel. Can. Geot. J., 8: 299-314.

Davis, E.H. and Christian, J.T. 1971. Bearing capacity of anisotropic cohesive soil. J. Soil Mech. Found. Div., ASCE, 97(SM5): 753-769.

Freeman, W.S. and Sutherland, H.B. 1974. Slope stability analysis in anisotropic Winnipeg clays. Can. Geot. J., 11: 59-71.

Lee, K.M. 1988. Prediction of surface subsidence above shallow tunnels in clays. Ph.D. Thesis, Univ. of Western Ontario (in preparation).

Lee, K.M. and Rowe, R.K. 1988a. Deformations caused by surface loading and tunnelling: The role of elastic anisotropy. Submitted to Geotechnique.

Lee, K.M. and Rowe, R.K. 1988b. Effects of undrained strength anisotropy on surface subsidences induced by the construction of shallow tunnels. Submitted to Can. Geot. J.

Lo, K.Y. and Rowe, R.K. 1982. Prediction of ground subsidence due to tunnelling in clay. Res. Rept. GEOT-10-82, Fac. of Engrg. Science, Univ. Western Ontario, London, Canada.

Lo, K.Y., Leonards, G.A. and Yuen, C. 1977. Interpretation and significance of anisotropic deformation behaviour of soft clays. Nor. Geot. Publ., No. 117, Oslo, 1-16.

Lo, K.Y. and Morin, J.P. 1972. Strength anisotropy and time effects of two sensitive clays. Can. Geot. J., 9: 261-277.

Mair, R.J., Gun, M.J. and O'Reilly, M.P. 1981. Ground movements around shallow tunnels in soft clay. 10th ICSMFE, Stockholm, 323-328.

Muckle, J.G. 1984. The effect of stress path on the behaviour of Thunder Bay clay. M.E.Sc. Thesis, Univ. Western Ontario, London, Canada.

Ng, R.M.C. 1984. Ground reaction and behaviour of tunnels in soft clay. Ph.D. Thesis, Univ. of Western Ontario, London, Canada.

Rowe, R.K. and Kack, G.J. 1983. A theoretical examination of the settlements induced by tunnelling: four case histories. Can. Geot. J. 20: 299-314.

Rowe, R.K., Lo, K.Y. and Kack, G.J. 1983. A method of estimating surface settlement above shallow tunnels in soft soil. Can. Geot. J., 20: 11-22.

Yuen, C. 1976. Cross-anisotropic deformation behaviour of a Leda clay. M.E.Sc. Thesis, Univ. of Western Ontario, London, Canada.