The Determination of Experimentally Based Load-Deformation Properties of a Mine Fill

by

B. G. RICHARDS, B.E., M.S.

Principal Research Scientist, CSIRO, Division of Applied Geomechanics

SUMMARY. Until recently, the load response of soil-like materials has been investigated with theoretical models, which assume either linear elasticity or perfect plasticity. Furthermore, laboratory samples, such as triaxial samples, have been considered as single homogeneous elements. These assumptions, being rather inadequate, often lead to a poor correlation between experimental data obtained from laboratory samples and field observations.

This paper investigates the problem of load response of a mine fill using concepts of non-linear material behaviour based on experimentally determined stress-strain curves. Since the laboratory sample is treated as a heterogeneous sample with stress-dependent properties, complications such as rough end plates usually encountered in the triaxial test can be avoided. As a result, the determination of more relevant material parameters is less dependent on the test procedure and the results can be used more confidently in geotechnical applications.

1 INTRODUCTION

The determination of soil properties is one of the most important problems in any design decision. Large scale field tests carried out in situ have many advantages due to sample size and material variability, however, with small scale laboratory tests on 'intact' or re-constituted samples, a wide range of stresses and environmental variables can be readily controlled and the constitutive relationships of the material determined.

With the widespread use of the finite element method, a large number of non-linear elastic and elasto-plastic models have been proposed. In particular, the hyperbolic stress strain model, developed for clays by Konder (Ref. 1) and for sands by Konder and Zelasko (Ref. 2) and applied to other soils by numerous research workers, is a simple and yet practical approach for idealising the entire laboratory stress-strain curve. Of special interest is Nelson's (Ref. 3) Variable Moduli Model II (VMII) which was successfully applied to triaxial testing by Radhakrishnam (Ref. 4). This model has been developed from the simpler hyperbolic model to describe the non-linear behaviour of soils up to failure conditions under full three dimensional stress states. Its successful application to practical problems has been achieved by using parameters determined in the tests with similar triaxial or plane strain stress paths. While the model can theoretically follow any three-dimensional stress path with its constitutive relationships, this has still to be verified in practice and should only be done with caution.

In this paper a variation of the Variable Moduli Model II is used to investigate the virgin load response of a mine fill (silty sand) and its suitability as a working platform in the mine. This constitutive model permits the description of non-linear stress-dependent material properties based on the entire experimentally determined stress-strain curves at least up to failure conditions. The laboratory sample is treated as a heterogeneous sample having stress-dependent properties and the boundary conditions, such as rough end plates usually encountered in a triaxial test are considered realistically. In fact, instead of attempting to eliminate end friction, it may not only be more convenient but more useful to use rough end plates.

As in full-sized geotechnical applications, the laboratory test model is not taken as a single element but consists of a large number of elements. This approach is aimed at defining more relevant material parameters which are less dependant on the test procedure and which can be used more confidently in the analyses of practical problems.

2 FILL MATERIAL

The fill material considered in this investigation was that used in the CSA Mine, Cobar, New South Wales, where the copper and zinc orebodies are mined as high rise stopes by the cut and fill method (Ref. 5). As stoping proceeds, a working platform is maintained by the introduction of screened and cycloned mill tailings ranging in size from silt to fine sand. This fill is placed hydraulically as a slurry of fill particles, mixed to approximately 65% by weight of solids.

The sedimentary aspects of the filling process have been described by Barton (Ref. 6), and some of the engineering properties of the fill have been previously investigated (Ref. 7; Ref. 8; Ref. 9; Ref. 10; Ref. 11). The results for the stope 12CE considered in this investigation can be summarized as follows.

(a) Grading

The grading characteristics of the fill as shown by the results of 75 analyses (Fig. 1) are confined within fairly close limits and no significant difference was detected between the surface and sub-surface samples.

(b) Density of Fill

The dry density of the fill varies with location but the mean of the values, typical of fresh fill, is of the order of 1.65 tonnes/m^3.

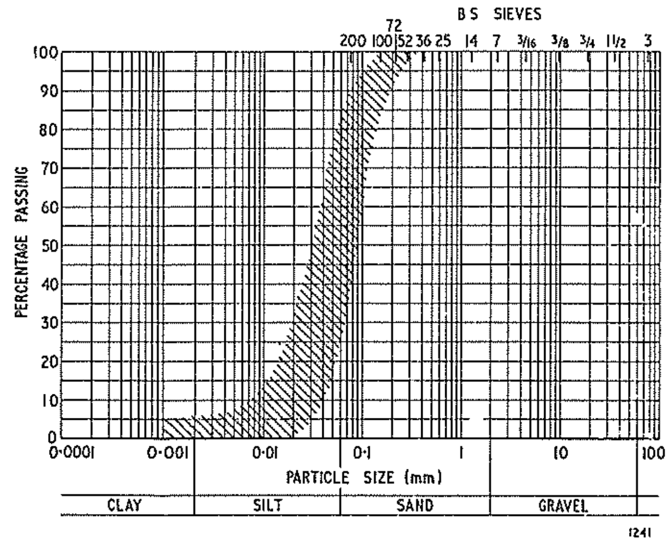

Fig. 1 Particle size of the fill material

(c) Stiffness and Shear Strength

Triaxial compression tests indicated that the fill material behaved frictionally with practically no cohesive component. The undrained shear strength parameters (C_u; ϕ_u) and the modulus and Poisson's ratio (E; ν) were independent of moisture content over the range tested (degrees of saturation were from 0 to 0.70), but increased significantly with increasing density. These parameters were also highly stress-dependent and showed large increases in both modulus and ϕ on unloading and reloading (e.g. compaction).

At a dry density of 1.65 tonnes/m^3, being typical of the fresh fill, $\phi_u = \phi_d = 35^o$ and the initial modulus (E was 21MPa) were suggested as typical values.

3 EXPERIMENTAL RESULTS

(a) Plate Bearing Tests

The plate bearing tests were conducted on a steel plate of 0.75 m in diameter, which was loaded by a hydraulic jack, using the back of the stope for reaction. The jack was pumped up to 7MPa in increments, reduced back to zero and then pumped up again until failure at the fill surface occurred or the maximum load of the jack was reached. At each loading step, the four dial gauges, which were mounted on a 5 m long reference frame and located around the edge of the plate at 90^o spacing, were read and averaged.

The filling of stope 12CE took place from January 25 to February 15, 1974. The first successful test was carried out four days after filling had ceased. A total of 11 tests were carried out until 61 days after filling. Both modulus and ϕ_u increased with time due initially to drainage and consolidation of the fill and later to compaction under mine traffic.

Full details of these tests have been reported elsewhere (Ref. 11). Test No. 3 was chosen for the investigation reported here as being representative of drained fresh, i.e. uncompacted, fill at its typical density of approximately 1.65 tonnes/m^3 is and is shown in Fig. 2.

(b) Laboratory Triaxial Tests

Stress-controlled triaxial tests were carried out on representative drained samples of the fill.

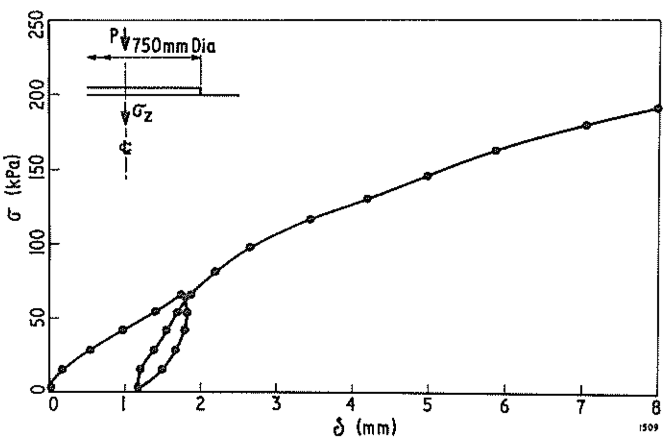

Fig. 2 Plate bearing test 12CZ 1/24/3

These samples, 100 mm diameter x 200 mm high were prepared to the desired dry density of 1.65 tonnes/m^3 by pouring the calculated weight of fill material, of which the water content was known, into a sample mould filled with water. By tapping the mould gently, the calculated weight could be made to occupy the desired volume. By lowering the burette attached to the bottom drain, with the top drain closed, so that a suction of 3.5 kPa was obtained, the mould could be removed from the sample and the triaxial cell assembled.

The first series of tests was conducted on samples drained to zero pore pressure, i.e. the water level, in the burette attached to the top drain was maintained at the level of the top of the sample. Four tests were carried out at constant lateral pressures (σ_3) of 70, 140, 210 and 280 kPa. The vertical pressure σ_1 was incremented by fixed amounts up to a deviator stress ($\sigma_1 - \sigma_3$) of approximately 200 kPa, then reduced by the same intervals to zero before being increased again to failure or 5% strain, whichever occurred first. After each increment of vertical stress, the vertical strain, volume change and pore pressure were recorded until all stabilized to a constant value; the next increment was then added. The experimental data were used in a computer program which computed preliminary parameters, including incremental modulus Poisson's ratio, assuming uniform stress states, and plotted the results graphically.

This series of tests was repeated for various drained states, including free drainage before and during the test. These tests confirmed that the drained stress-strain relationships and soil parameters were not dependent on water content for a given dry density viz. 1.65 tonnes/m^3.

The results shown in Fig. 3 indicate the stress-strain curves for the drained tests with u = 0, with the unloading and loading cycle removed for clarity. These curves do not give any clear indication of failure, but an examination of the incremental Poisson's ratio permitted definition of the stress state where dilation (i.e. ν = 0.5) occurred. Mohr circles drawn on this basis gave good failure envelopes indicating that the fill parameters were of the order c_d = 0 and $\phi_d = 35^o$ (Ref. 11).

4 THEORETICAL ANALYSES

Previous finite element analyses with an incremental bi-linear elasto-plastic model using the Mohr-Coulomb criteria enabled the plate bearing test to be modelled (Fig. 2) giving reasonable parameters, viz. an initial modulus E of 21,000 kPa and

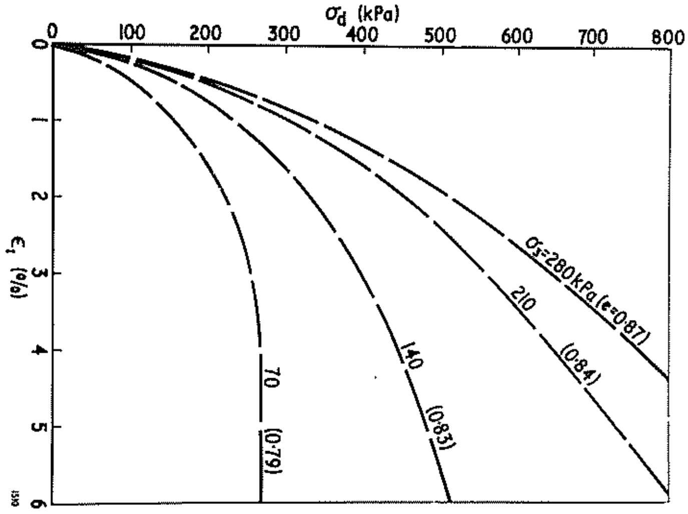

Fig. 3 Results of drained saturated triaxial test

ϕ_d = 35° (Ref. 11). However, when they were applied to the triaxial test itself, no agreement could be obtained, suggesting that the model was inadequate to describe the constitutive relationships for the fill generally.

Consequently, a modified variable moduli model was applied to the above experimental results. The model can be briefly described as follows :

(a) Modified Variable Moduli Model

The bulk modulus for the virgin loading K_{1d} is given as -

$$K_{1d} = K_1\sigma_0{}^n + K_0; \ \frac{d\sigma_0}{dt} > 0 \text{ and } \sigma_0 > \sigma_{max} \qquad (1)$$

where σ_0 = mean stress or octahedral normal stress
σ_{max} = maximum previous mean stress
K_1, K_0 and n are material constants.

The bulk modulus for unloading and reloading $K_{un} = K_{re}$ is expressed as

$$\left.\begin{matrix} K_{un} \\ K_{re} \end{matrix}\right] = K_{1u}\ \sigma_{max}^{n'} + K_{ou}; \ \frac{d\sigma_0}{dt} \leqslant 0 \text{ or } \frac{d\sigma_0}{dt} > 0 \text{ and } \sigma_0 < \sigma_{max} \qquad (2)$$

where K_{1u}, K_{ou} and n' are material constants. The shear modulus for the virgin loading

G_{1d} is given as

$$G_{1d} = G_1\ \sigma_0^m\ (1 - \left(\frac{\tau_0}{\tau_f}\right)^{p'}) + G_0; \ \frac{d\tau_0}{dt} > 0 \text{ and } \tau_0 > \tau_{max} \qquad (3)$$

where τ_0 = octahedral shear stress
τ_f = octahedral yield stress
τ_{max} = maximum previous octahedral shear stress
G_1, G_0, m and p are material constants.

The shear modulus for unloading and reloading $G_{un} = G_{re}$ is expressed as

$$\left.\begin{matrix} G_{un} \\ G_{re} \end{matrix}\right] G_{1u} = \tau_{max}^{m'}\ (1 - \left(\frac{\tau_0}{\tau_f}\right)^{p'}) + G_{ou}; \ \frac{d\tau_0}{dt} \leqslant 0 \text{ or } \frac{d\tau_0}{dt} > 0 \text{ and } \tau_0 < \tau_{max} \qquad (4)$$

where G_{im}, G_{ou}, m' and p' are material constants.

(b) Analysis of Triaxial Tests

The triaxial specimen was modelled by a finite element mesh with the boundary conditions shown in Fig. 4. With a distributed load (i.e. stress, σ) applied to the stiff top cap, no lateral displacement relative to the top cap is assumed to take place at the interface between the specimen and the top cap. The lateral strains in the soil specimen at this interface are insignificant at maximum lateral restraint at the top cap; this approximates to the observed experimental conditions. With the top cap in Fig. 4 removed, the case of no lateral restraint at the top cap can be analysed. This is the case for uniform homogeneous stresses, assumed in the classical analyses of the test.

The actual method of analysis used in the computer program is an incremental-iterative load procedure using the tangential stiffness over the increment of load. In this method, the increment of load is applied using the tangential stiffness at the stresses before the increment is applied. This provides an estimate of the stresses after the increment is applied. This loading step is repeated in an iterative manner, now using the average of the initial and final stresses for this load increment until convergence is obtained. Usually only two to three iterations are required for a satisfactory result. Following convergence the next increment of loading is applied and the iterative process repeated.

In the investigation, described in this paper, only the modelling of the virgin curve has been attempted. However, unloading of some elements may occur during the virgin loading cycle, so the unloading and reloading moduli given in equations (2) and (4) were used with constants equal to the virgin values. Further work is now being undertaken to find better expressions for the unloading and reloading cycles and ways of determining the appropriate material parameters.

In order to make a first estimate of the virgin material constants K_1, G_1, n and m, the initial values of K_{1d} and G_{1d} obtained from the experimental results were plotted versus σ_0, giving the constants

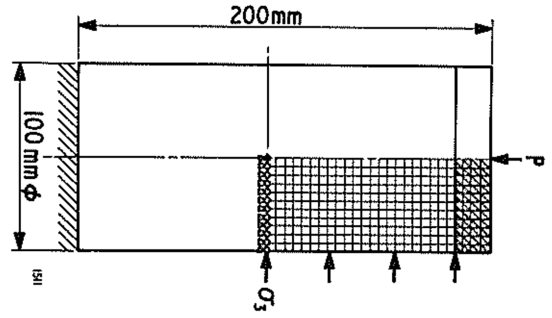

Fig. 4 Finite element mesh for triaxial test

shown in Fig. 5. The first estimates of the shear strength parameters were obtained from the envelope of the Mohr circles for the triaxial tests at 5% strain, viz. c_d equal to zero and ϕ_d equal to 42°. Using these constants and assuming the value of p to be 0.3 (approximating to a second order hyperbola), the finite element analysis of the triaxial test series with full end restraint is summarized by the stress-strain curves in Fig. 6.

An examination of Fig. 6 suggested that ϕ_d, for example, was too high. However, lowering ϕ_d caused the shape of the whole stress-strain curves to change, indicating that all the soil constants or parameters were inter-dependent. With experience the material constants can be adjusted in only one or two trials to give a reasonably good fit. Considering the void ratios of each test as shown in Fig. 3, the fit shown in Fig. 7 was selected as the most satisfactory.

The constitutive relationships for the virgin loading of this fill material can therefore be described by the expressions

$$K_{1d} = 6,100.\ \sigma_0^{0.33} \tag{5}$$

$$G_{1d} = 2,830.\ \sigma_0^{0.49} \left(1 - \left(\frac{\tau_0}{\tau_f}\right)^{0.3}\right) \tag{6}$$

and $$\phi = 40^{\circ} \tag{7}$$

(c) Analysis of Plate Bearing Test

This constitutive relation was tested in the analysis of the plate bearing test, carried out in situ on the fill material, as described above. The finite element mesh and the boundary conditions together with the result are shown in Fig. 8. The agreement with the experimental results is excellent for the lower displacement. The comparisons at higher displacements suggest a slightly lower value of ϕ might have given better agreement. The unloading and reloading cycle was included. While the agreement was reasonably good, it is not the purpose of this investigation to consider unloading and reloading parameters.

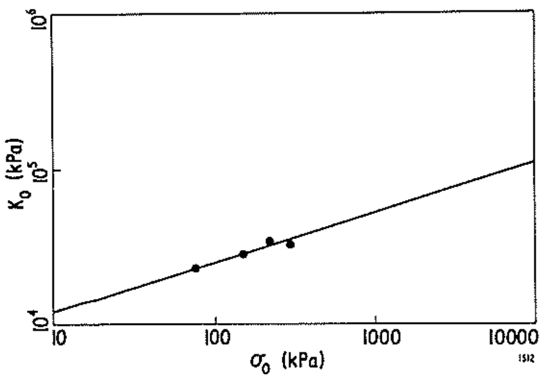

Fig. 5a Initial bulk modulus (K_0= 5650. σ_0 $^{0.326}$)

(d) Analysis of Radial Strain and Volume Change in the Triaxial Test

The constitutive relationships described above by equations (5) to (7) were selected on the basis of stress-strain relationships only. Therefore other variables measured in the triaxial test will provide a further check on the validity of these relationships.

The lateral or radial displacements of the side of the triaxial specimen are shown in Fig. 9 for two stress levels. As no direct measurements of the lateral strain were made, no direct comparison could be made. However, the predicted displacements were similar to those visually observed and are more correct than those normally assumed.

A better quantitative check could be made on the mean displacements or volume change as shown in

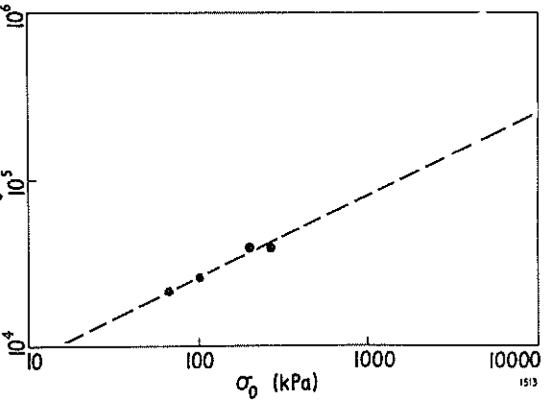

Fig. 5b Initial shear modulus (G_0= 2620. σ_0 $^{0.490}$)

Fig. 10. The trends are the same, although agreement is not good over the whole curve. The dilation shown at the higher stress levels could not be modelled by the computer program. The experimental and theoretical values of Poisson's ratio tended to exceed 0.5, but a check in the program limited the value to less than 0.5. Therefore the model must be used with caution if and when dilation occurs.

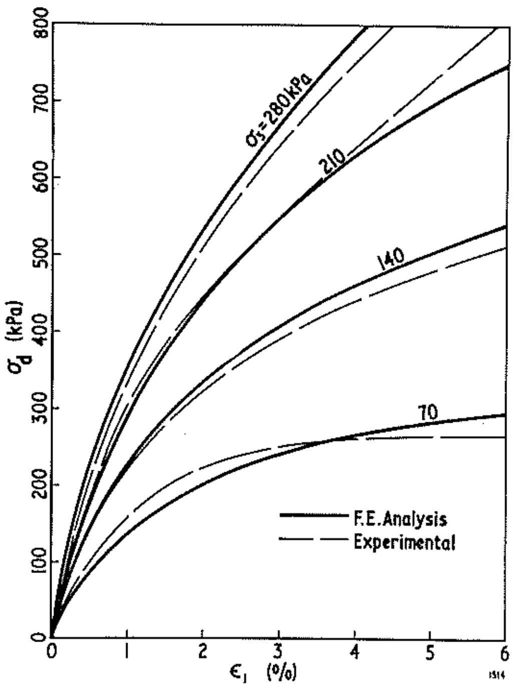

Fig. 6 Analytical result for triaxial test + smooth rigid cap (K = 5650. $\sigma_0^{0.33}$, G = 2620. $\sigma_0^{0.99}$F, F = (1-$f^{0.30}$), ϕ = 42°)

(e) Effect of Top Cap

The results for full end restraint by the top cap have already been demonstrated in Fig. 7. A similar analysis was made assuming no end restraint, but the same constitutive relationships for the specimen. The results are summarized in Fig. 11. This suggests that end restraint would have little effect on the stress-strain curves. In fact, further analyses assuming no end restraint indicate that the stiffness parameters K_{1d} and G_{1d} would have been under-estimated by about 8% but ϕ_d would have increased from 40° to 42°, which may be more significant in practice, e.g. bearing capacity.

A further point of interest was the distribution of the vertical and radial stresses and strains

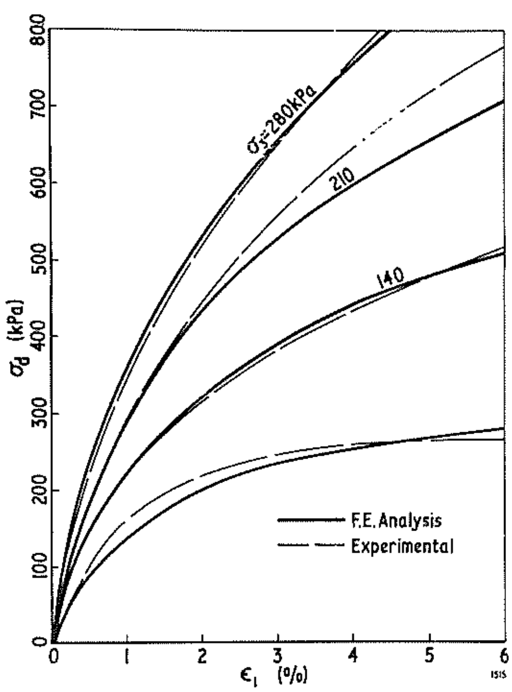

Fig. 7 Analytical result for triaxial test + rough rigid cap (K = 6100. $\sigma_0^{0.33}$, G = 2830. $\sigma_0^{0.49}$F, F=(1-$f^{0.30}$) ϕ = 40°)

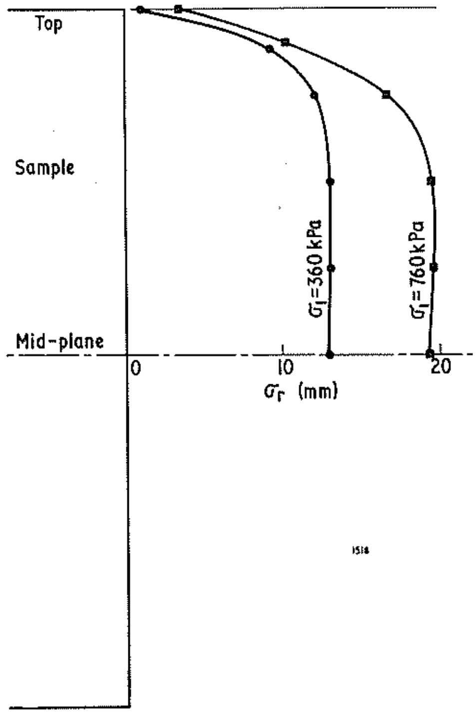

Fig. 9 Radial displacements in triaxial test (σ_3 = 280 kPa)

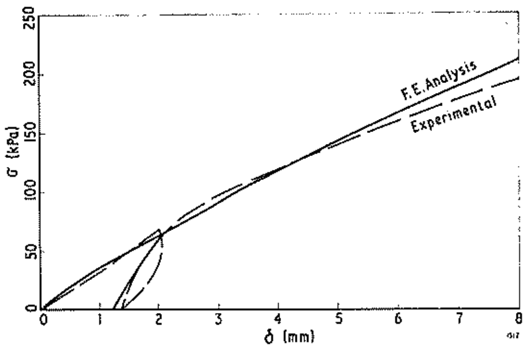

Fig. 8 Finite element mesh and analytical result for plate bearing test.

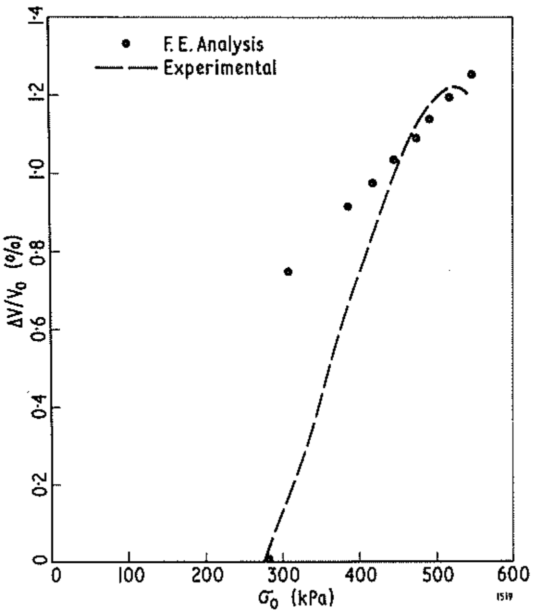

Fig. 10 Volume changes in triaxial test (σ_3 = 280 kPa)

predicted in the triaxial specimen. For no end restraint, both the vertical and radial stresses and strains were uniform throughout the specimen and are not shown. For full end restraint, the vertical stresses and strains were also reasonably uniform throughout the specimen. However, the radial and shear stresses and strains can be seen to vary considerably from the nominal values near the top cap (Fig. 12 and 13).

5 CONCLUSIONS

A modified version of the variable moduli model to assess the constitutive relation for the fill material was used for analysing both the laboratory triaxial tests and a field plate bearing test. When adjusted to fit the laboratory tests, it gave good agreement with the field test. Its ability to analyse the complete stress-strain behaviour of soil is still questionable, and more development is required particularly in regard to volume change. In addition it is not applicable to soils when dilation occurs (i.e. $\nu > 0.5$).

In spite of this, it does show good potential for determining constitutive relations from laboratory tests, which can be extrapolated confidently to field situations. This is not always possible with other currently available methods. It also permits more flexibility and, possibly, simplification of the laboratory testing by making the

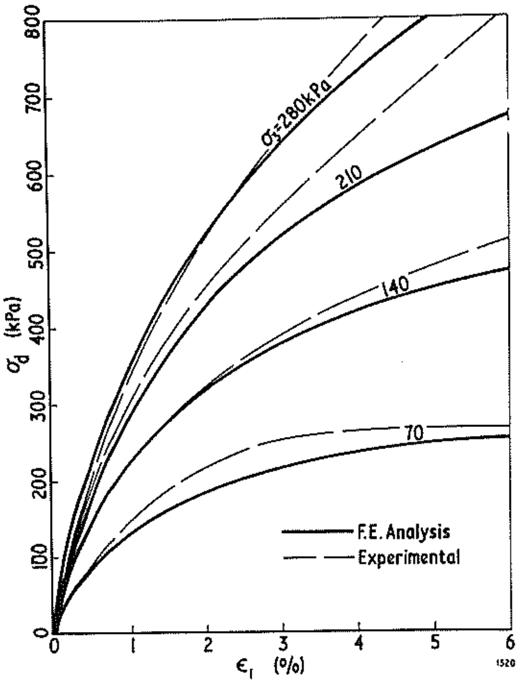

Fig. 11 Analytical result for triaxial test + smooth rigid cap (K=6100. $\sigma_0^{\ 0.33}$, G=2830. $\sigma_0^{\ 0.49}F$, $F=(1-f^{0.30})$ $\emptyset = 40^{o}$)

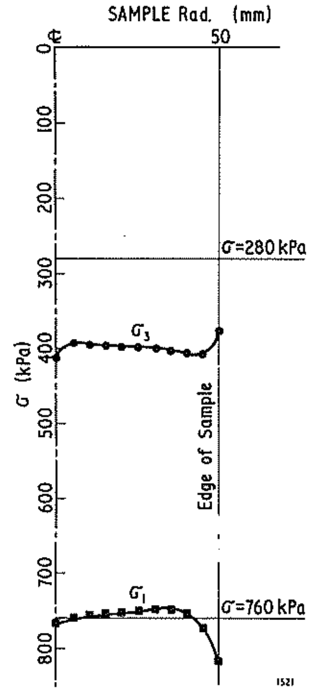

Fig. 12 Stresses under top cap in triaxial test

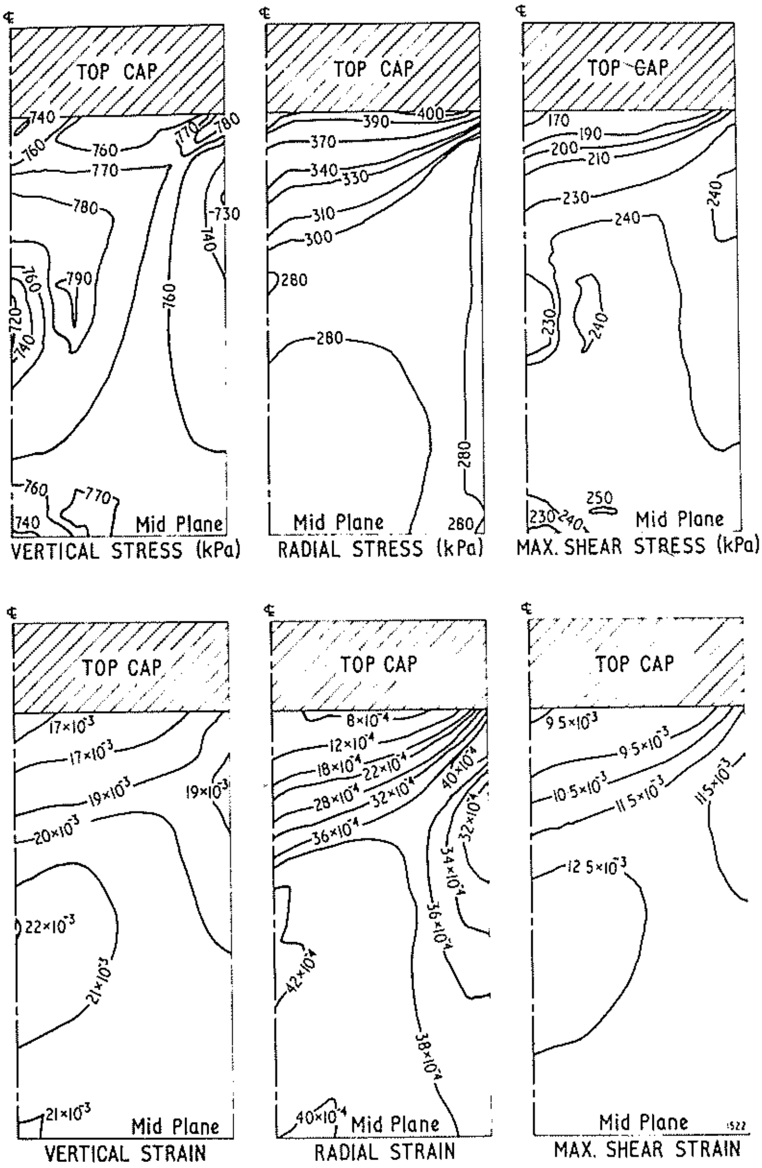

Fig. 13 Displacements and stresses in triaxial specimen with full end restraint

material parameters less dependent on the test, but it still requires at this stage that the testing be relevant to the field situation and the appropriate stress paths in the field to be followed. It is also advisable that the constitutive relations so obtained be checked by large scale field testing as described in this paper, before extending it to practical situations.

This paper has only been concerned with the virgin loading curve but previous work referred to in this paper has obtained satisfactory predictions for unloading and reloading. Further work with this model is continuing in this direction.

6 REFERENCES

1. KONDER, R.L. Hyperbolic Stress-Strain Response: Cohesive Soils. J. Soil Mech. & Foundations, Div., ASCE, 1963, Vol. 89, No. SM1, pp. 115-143.

2. KONDER, R.L. and ZELASKO, J.S. A Hyperbolic Stress-Strain Formulation for Sand. Proc. 2nd Pan-American Conf. on Soil Mech. & Fndn. Engng, 1963, Vol. 1, pp. 289-333.

3. NELSON, I. Investigation of Ground Shock Effects in Non-linear Hysteretic Media. Report 2, Modelling the Behaviour of a Real Soil, Report S-68-1, Contract DACA 39-67-C-0048, Paul Weidlinger Consulting Engineer, U.S. Army Waterways Experiment Station, 1970.

4. RADHAKRISHNAN, N. Analysis of Triaxial Tests by the Finite Element Method. Applications of the Finite Element Method in Geotechnical Engineering. Proc. Symp. Vicksburg, Mississippi, Vol. III, 1972, pp. 949-1003. Editor C.S. Descu, U.S. Army Engineer Waterways Exp. Stat., Corps of Engineers.

5. BRADY, J.T., OWERS, N.F. and ANNEAR, C.H. Ore Breaking and Handling at the CSA Mine, Cobar, N.S.W. Proc. Aust. Inst. Min. Metall., 1969, No. 229, pp. 7-15.

6. BARTON, C.M. Sedimentary Aspects of Fill in the CSA Mine, Cobar, N.S.W. Div. of Applied Geomechanics, Technical Report No.18, CSIRO, Australia, 1973.

7. LANG, J.G. Report on Tests on Hydraulically Placed Fill, Stope 12CE/22/1, CSA Mine, Cobar, N.S.W. The Geomechanics of Filled High Rise Stopes. Progress Report No. 2, Appendix I, CSIRO, Div. of Applied Geomechanics, 1971.

8. LANG, J.G. Triaxial Compression Tests on Hydraulic Fill. The Geomechanics of Filled High Rise Stopes, Progress Report No. 3, Appendix B, CSIRO, Div. of Applied Geomechanics, 1972.

9. FOULGER, D.C. Permeability of Flocculated Fill. CSIRO. Div. of Applied Geomechanics, Internal Report No. 9, 1972.

10. LANG, J.G. and GERRARD, C.M. Permeameter and Oedometer Tests on Fill. The Geomechanics of Filled High Rise Stopes. Progress Report No. 3, Appendix A, CSIRO, Div. of Applied Geomechanics, 1972.

11. RICHARDS, B.G. Investigation of the Bearing Capacity of the Fill at the CSA Mine, Cobar, N.S.W. The Geomechanics of Filled High Rise Stopes. Project Report No. 3, CSIRO, Div. of Applied Geomechanics, 1974.