Non-Linear Behaviour of a Silty Clay Dam Core

La Tenue Non-Linéare d'un Milieu Argileux d'une Barrage

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SYNOPSIS A programme of triaxial tests has been conducted to examine the non-linear deformation behaviour of a typical silty clay core, as used in Talbingo Dam. The material is shown to be well suited to hyperbolic modelling. Consideration is also given to the internal collapse potential that may be generated by the passage of a saturation front. The hyperbolic model has been checked by back-analysis of triaxial tests, and applied to an analysis of deformations and pore-pressures in Talbingo Dam.

INTRODUCTION

Compacted unsaturated clayey materials are normally used as the core element of fill dams. Service loadings are complex, even under normal operating conditions, and stress ranges are such that stress-strain behaviour is significantly non-linear. Non-linear methods should therefore be applied to the analysis of deformation of such structures, with parameters derived from laboratory tests which model the field situation reasonably.

First filling introduces water loads on the upstream face of a dam core, and initiates an advancing wetting front. Because compacted soils have a potential for structural collapse when saturated under stress, core materials must be checked in this respect, and also to provide data for deformation analysis.

This paper, which is complementary to Parkin and Adikari (1981), presents results of an investigation of non-linear stress-strain and collapse behaviour of the core material for Talbingo Dam (Snowy Mountains Scheme, Australia) and indicates the use of derived parameters in a finite element analysis. Details of the hyperbolic parameters and their application are given by Parkin and Adikari.

TESTING PROGRAMME AND MATERIAL PROPERTIES

The core material was a silty clay derived from Andesitic basalt, for which basic data is given in Table I.

TABLE I

w_L , w_p	42, 23
Linear shrinkage, % clay	7.2, 20
Solids density ρ_s (t/m^3)	2.80
Permeability k (m/s)	2.8×10^{-10}
γ_d (kN/m^3), w_{opt} (Standard)	17.0, 18.0
γ_d (kN/m^3), w_{opt} (Heavy)	18.1, 15.5

Initial testing consisted of a series of consolidated undrained (CU) triaxial compression tests performed by the SMA. However, as these tests could not provide sufficient data for a non-linear analysis of dam behaviour for comparison with field measurements, two further series of tests were performed at Monash University. These consisted of an unconsolidated undrained (UU) series and a consolidated drained (CD) series, as detailed in Table II. Of these, the CU series was tested unsaturated, with pore water pressure measurements, the UU series also unsaturated with initial pore-pressure only and volume change by cell water outflow, and the CD series saturated with volume change via burette. The UU tests model end of construction conditions and the CD tests the steady seepage condition. A further series of 7 consolidation tests was carried out to investigate collapse behaviour on wetting.

NON-LINEAR STRESS-STRAIN BEHAVIOUR

Results from the three triaxial series are shown in Figs 1, 2 and 3, and hyperbolic parameters derived from these graphs are given in Fig. 4

TABLE II

Details of Triaxial Compression Tests

Series	Preparation	Parameters
UU Monash	$\gamma_d = 18.0\ kN/m^3$ w = 15 - 17%	c_u = 85 kPa $\phi_u = 23°$
CU SMA	$\gamma_d = 18.4\ kN/m^3$ w = 16.7%	c_u = 70 kPa $\phi_u = 23.7°$ c' = 55 kPa $\phi' = 33°$
CD Monash	$\gamma_d = 18.0\ kN/m^3$ w = 15 - 17%	c_d = 55 kPa $\phi_d = 32°$

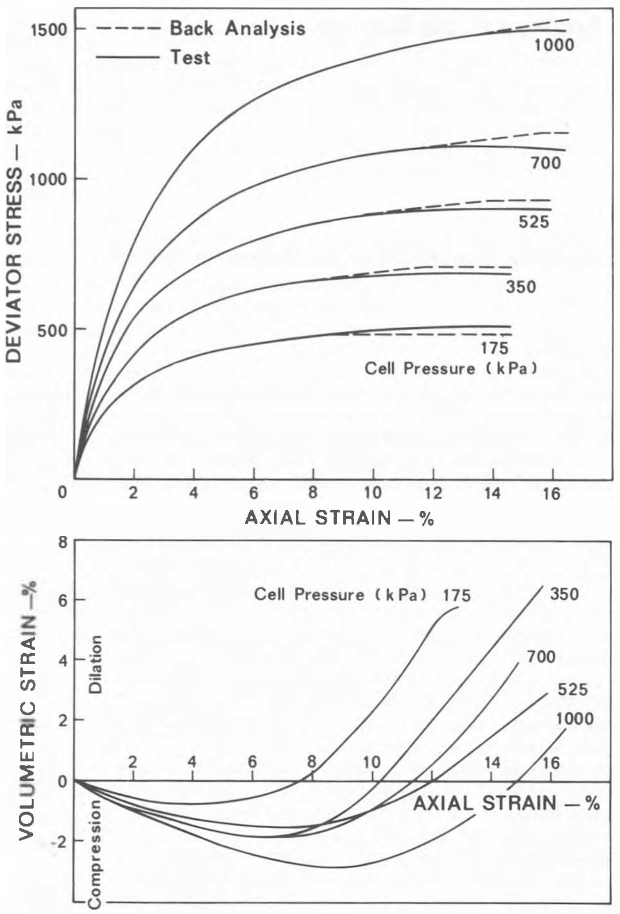

Fig. 1 Stress-strain behaviour, UU tests

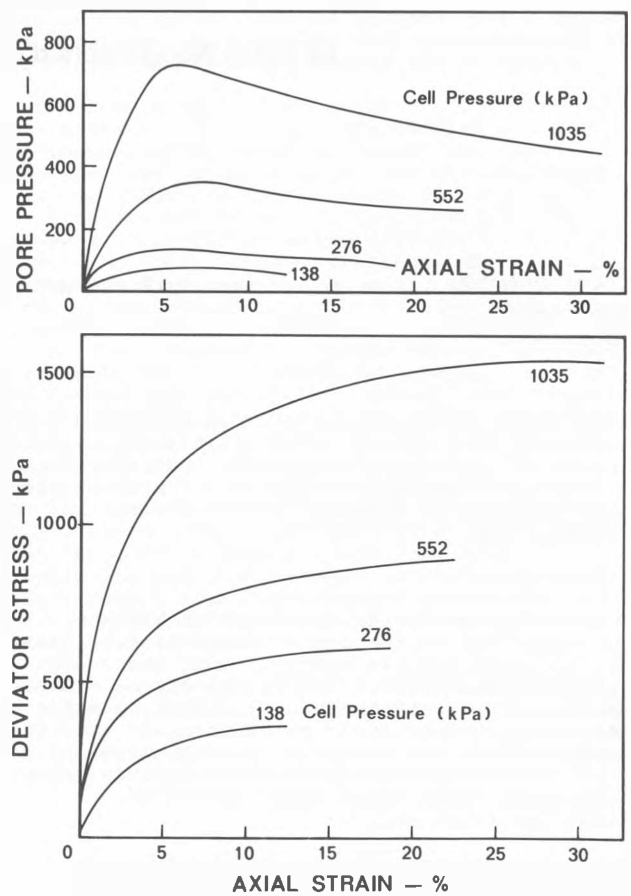

Fig. 2 Stress-strain-pore pressure, CU tests

and Table III. The essentially linear Mohr-Coulomb envelopes are given in Fig. 5, with both total and effective stress interpretations of Series CU.

PORE PRESSURE PARAMETERS

Incremental pore pressure parameters are required for the finite element analysis. As Series UU and CU specimens both contained air and water it is necessary to define the incremental pore water pressure change as

$$\Delta u_w = \Delta u_{wa} + \Delta u_{wd} \qquad (1)$$

TABLE III

Hyperbolic Parameters (from UU tests)

K	250	R_f	0.86	F	0.315
K_{ur}	400	d	5.0	G	0.045
n	0.45				

where Δu_{wa} and Δu_{wd} are components due to all-round stress $\Delta\sigma_3$ and stress difference $(\Delta\sigma_1 - \Delta\sigma_3)$ and as $\Delta u_w = B_i [\Delta\sigma_3 + A_i \, \Delta(\sigma_1 - \sigma_3)]$ (2)

the incremental pore-pressure parameters are

$$A_i = \Delta u_{wd} / B_i \, (\Delta\sigma_1 - \Delta\sigma_3) \qquad (3)$$

and $$B_i = \Delta u_{wa} / \Delta\sigma_3 \qquad (4)$$

The B parameters were investigated for specimens compacted to a simulated field condition. Initial negative pore-water pressures, measured according to Bishop et al. (1960), averaged 70 kPa. Pore pressures were then recorded for 100 kPa increments of cell pressure, leading to graphs of B and B_i versus cell pressure (Fig 6). These curves can be described by the hyperbolic equations:

$$B = \sigma_3/(400+\sigma_3) \qquad B_i = \sigma_3/(150+\sigma_3) \qquad (5)$$

for σ_3 in kPa. These relationships are also plotted in Fig. 6.

The A_i parameters were calculated from Series CU tests using equation (3), as shown in Fig. 7. In kPa units, these curves can be expressed as

$$A_i \, B_i = 5.5 \times 10^{-4} \, (\sigma_1 - \sigma_3) + 0.3 \qquad (6)$$

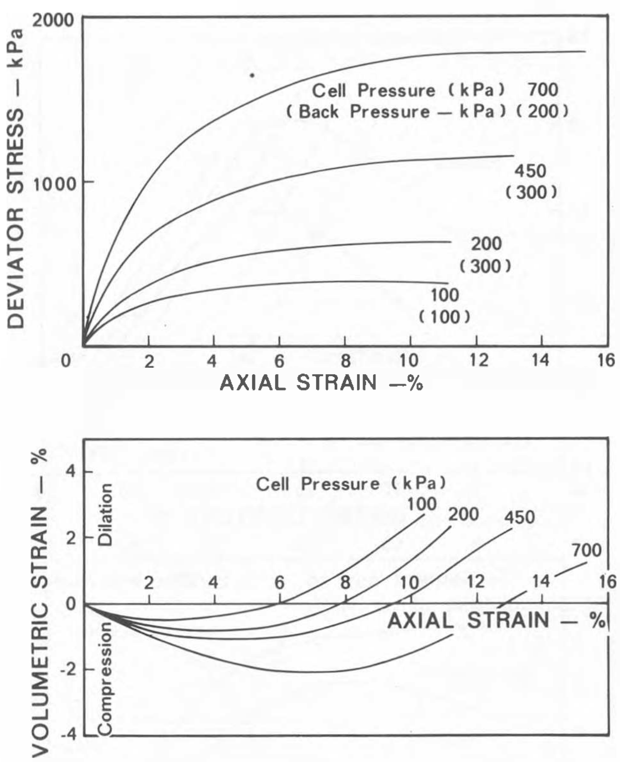

Fig. 3 Stress-strain behaviour, CD tests

for values of $(\sigma_1 - \sigma_3)$ below peak, and B_i given by (5) above. As no elements in the dam reached failure, equation (6) is adequate for use in pore pressure predictions.

COLLAPSE BEHAVIOUR

The collapse potential of core material depends on clay content, initial moisture content and dry density, and stress level. Settlement during construction occurs mainly by compression of air voids, but additional settlement may follow the passage of a wetting front. This latter component of total settlement is significant when the clay binder content is small (e.g. silty soils) and the compaction water content considerably dry of optimum.

Specimens were compacted, at moisture contents ranging from 5% to 27.5%, into a special consolidometer 100mm dia. by 65mm high. Samples were loaded to 1,600 kPa (representative of core stress at elevation 445m) and then flooded, with vacuum assistance.

Fig. 8 shows settlement curves for the dry and saturated states under the 1600 kPa load: the latter curve shows increased settlement for "dry" compaction, and again for "wet" samples, where it is essentially consolidation. A minimum occurs, in this case, about 5% dry of optimum. An upper limit to the collapse potential of dry materials can also be defined from the percentage residual air voids.

The collapse potential of the Talbingo core, placed marginally wet of optimum, is considered

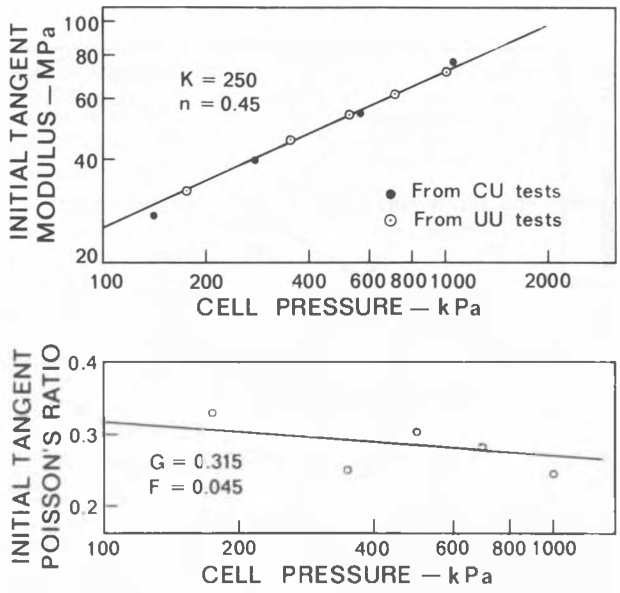

Fig. 4 Hyperbolic deformation parameters

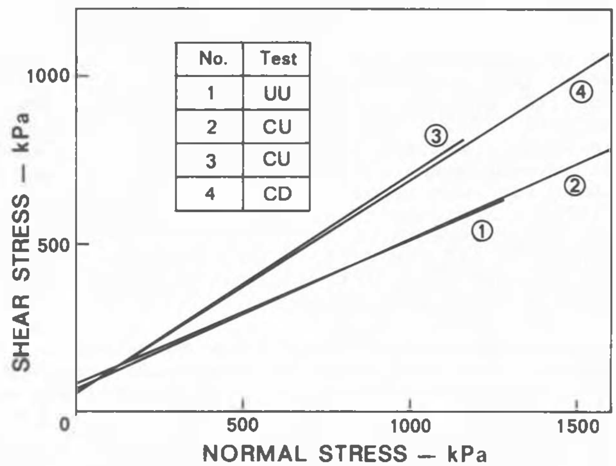

Fig. 5 Mohr-Coulomb envelopes

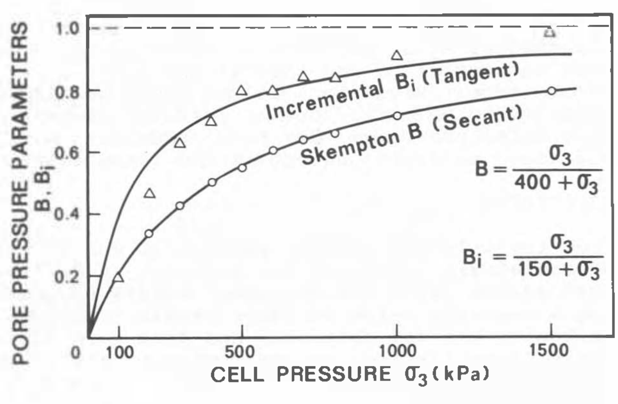

Fig. 6 Tangent pore-pressure parameter B_i

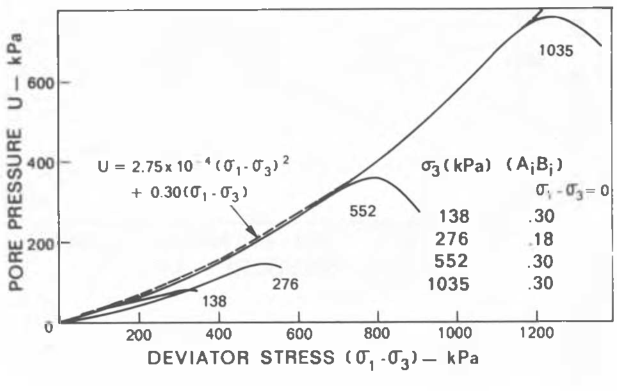

Fig. 7 Tangent pore-pressure parameter A_i

small, and has not therefore been incorporated into the present analysis. If required, however, it can be included, following the method of Nobari and Duncan (1972) as applied to wetting collapse of an upstream rockfill shell, but it would not appear that this has been done yet for a core.

DEFORMATION AND PORE-PRESSURE IN TALBINGO DAM

Using parameters for the core derived from the above test programme, and parameters for the rockfill and filters as derived by Parkin and Adikari (1981), the FE method has been applied to analyse the end-of-construction, first filling and steady seepage situations. The first of these has been reported by Parkin and Adikari (1980). Representative results are given here for displacements on a horizontal section at elevation 457.2 m (Fig. 9) and for pore-pressures in the core (Fig. 10), together with the corresponding field measurements.

With respect to settlement, the computational models were found to be in reasonable agreement with field measurements, but with a tendency for higher values in the core and lower values in the rockfill shells. However, for horizontal movement, it was found that the non-linear model is substantially low on the upstream side, which may be related to numerical instabilities (in E and ν) occurring in the outermost elements of the core and rockfill zones on this side.

The pore-pressure predictions, based on the incremental A_i B_i parameters, would appear to be more successful in the central and lower portions of the core. Deviations in the upper portion are believed to be partly due to residual negative pore-water pressures, due to a reduction in the placement moisture content in the later stages.

DISCUSSION

From the triaxial testing programme, non-linear stress-strain behaviour was apparent in all three test series, with the deviator stress approaching a constant value at large strain. Work softening was not observed, and this form of curve is well-suited to hyperbolic modelling, as is evident from the back-calculated triaxial curves (Fig. 1).

Significant initial volume decreases occurred for both the UU and CD tests, with no net

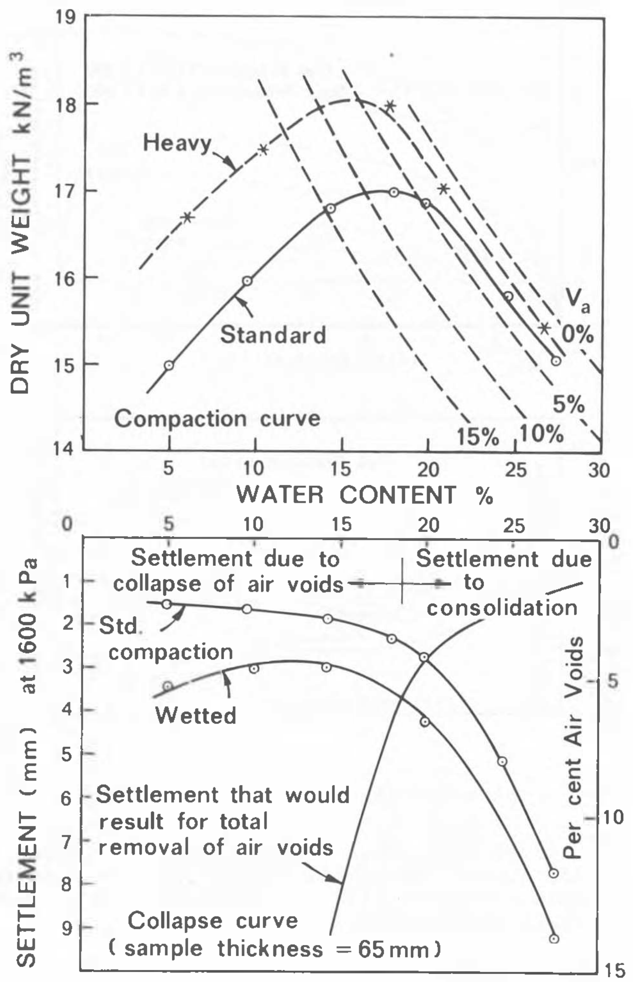

Fig. 8 Saturation collapse behaviour

dilation until near failure. This behaviour was reflected in the measured pore-water pressures during the CU tests, although this series was performed without back-saturation. This suggests, and measurements confirm, that initial negative pore-water pressures on compaction were small, so that (u_a-u_w) can be expected to be insignificant at test pressures (Bishop and Henkel, 1962) This should apply for all samples compacted at or wet of optimum moisture content. Further confirmation is found in the failure envelopes of Fig. 4, where the effective stress envelopes for CU and CD tests almost coincide, which would not occur if there was an appreciable air-water pressure difference (Donald, 1963).

From Fig. 4, the undrained failure envelopes for UU and CU tests are also very close, showing that the UU samples consolidated to equilibrium almost immediately by the compression cf air voids. These tests (UU), which were to simulate rapid construction conditions, had to be performed quickly to minimize air diffusion through the triaxial membrane.

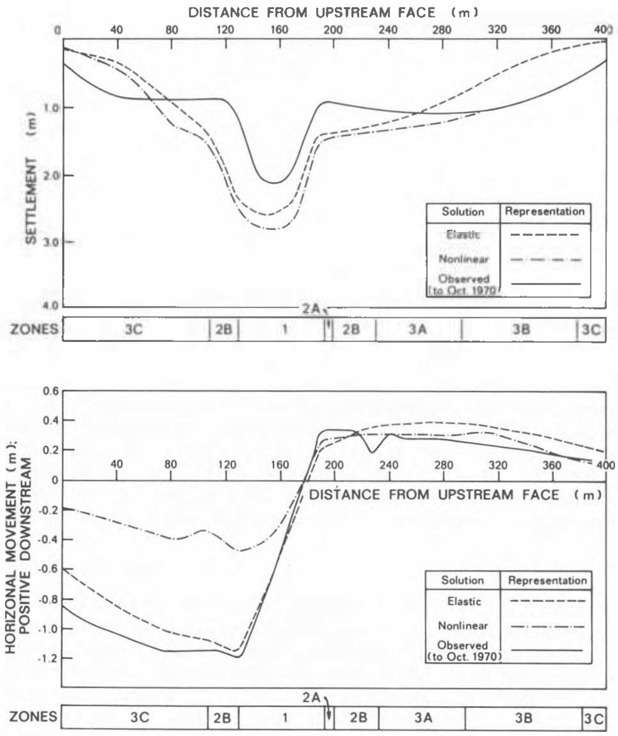

Fig. 9 Deformations at elevation 457.2 m

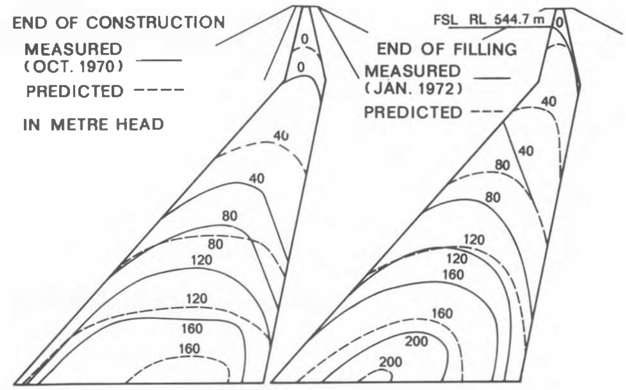

Fig. 10 Observed and predicted pore-pressures

CONCLUSION

In the case of Talbingo Dam, the hyperbolic model has been found to be eminently suitable for formulating stress-strain laws from triaxial test data. It has also been found that the pore pressure parameters can be expressed incrementally in terms of the principal stresses in a form which can, again, be conveniently incorporated into finite element analysis.

Only specimen results of the analysis of Talbingo can be included here, but these indicate the typical order of correlation between field observation and laboratory prediction. This indicates that the non-linear model can lead to improved predictions in some areas, but that the simpler elastic model also performs very well.

A modified oedometer test has been used to examine collapse potential on saturation. Although the collapse potential is small for the Talbingo core, principles have been established for further development. The stress changes on a core element during saturation are, however, not simple.

ACKNOWLEDGEMENTS

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APPENDIX

Sample dimensions for the triaxial tests were 150 x 300 mm in the case of the SMA tests and 75 x 150 mm for the Monash tests. Axial loading rates were as follows:

UU Series	0.5 mm/min
CU Series	0.17 mm/min
CD Series	0.027 mm/min

Parameters for elastic analysis are given by Parkin and Adikari (1980).