Finite Element Analysis of an Earth Dam and Foundation

by

I. M. PARTON, B.E.(Hons), Ph.D., M.N.Z.I.E.

and

M. J. PENDER, B.E.(Hons), Ph.D., M.N.Z.I.E.

Ministry of Works and Development Central Laboratories, N.Z.

1 INTRODUCTION

Pukaki High Dam is a key link in a chain of lakes and canals being constructed to form the basis of the Upper Waitaki Power Development Scheme. The dam is founded upon outwash gravels and ice contacted sands at the outlet of Lake Pukaki. The object of the dam is to raise the level of Lake Pukaki approximately 40 m to provide controlled water storage.

Because of the large number of dams already built, marginal dam sites are now being used throughout the world which would not have been considered previously. Further, because less desirable sites are being used, the quality of available construction materials is also declining.

A feature of this problem was the lack of precise information concerning the foundation materials and the precise geological profile at the time of design. Numerous borings and Benoto shafts have enabled geological conditions to be determined over a wide area, but the irregularities generally associated with glacial deposits make for difficult interpretation.

Undisturbed block samples taken from one area of the foundation have enabled strength parameters to be determined under triaxial conditions for one member of the ice contacted sands present. Strength parameters have also been determined for compacted samples of dam core and shoulder material.

The objectives of the finite element analysis were:

(a) to assess likely deformations within the dam,

(b) to check for the presence of zones of tensile stress within the dam at any stage during construction,

(c) to investigate the effect on the dam of foundation deformation.

This paper describes the laboratory testing of undisturbed samples, the analyses performed and presents computed results. A more complete description of the work has been given by Pender and Parton (1) and Parton (2).

2 GEOLOGY

Geotechnical investigations began at the Pukaki High Dam site in 1964. Geophysical investigations using seismic and resistivity surveys provided preliminary information which has since been supplemented by Calweld and Benoto shafts. Excavation for the dam commenced in November 1971. A detailed geological log of the shafts and all exposures has been prepared.

The major portion of the dam will be founded upon proglacial lake deposits with the shoulders of the dam on Tekapo Till and Mt John outwash gravel. The proglacial lake deposits are heavily contorted and range in texture from gravel to clay sizes (Read (3)).

The Tekapo Till forms a terminal moraine loop and is exposed in the steep, right bank above the Pukaki River.

The dam foundations are dry although perched water tables exist below that level.

3 LABORATORY TESTING

Four blocks of undisturbed sandy silt from the proglacial lake deposits were cut from the dam foundation excavation and returned to the laboratory. Classification and grading tests showed the material to be a slightly cemented silty-sand with non-plastic fines (SM). Additionally disturbed samples of core and shoulder material were returned to the laboratory for classification, compaction and strength testing.

A total of 92 triaxial compression tests were performed on the dam and foundation materials. Most of the specimens were tested at a low degree of saturation (50-70%) so the tests were effectively drained. Some specimens were saturated by the application of high back pressures and subjected to undrained tests with pore pressure measurement. The specimen diameters ranged from 38 mm to 210 mm. Twenty-seven of the tests were on foundation material, 58 on the core and 7 on the shoulder material.

The stress-strain behaviour of each specimen was carefully monitored, allowing a linear relationship between initial tangent modulus E_i and confining pressure to be determined (Eqtn. 3.1).

$$E_i = E_o + \alpha \sigma_3 \qquad (3.1)$$

where E_i = initial tangent modulus
σ_3 = confining pressure
and E_o and α are material parameters.

Typical results for the foundation material are plotted in Figure 1.

As the prime reason for the laboratory tests described in this paper was to gather data for use in Finite Element (FE) simulations, it was of interest to study the manner in which the tangent modulus values varied with increasing principal stress difference.

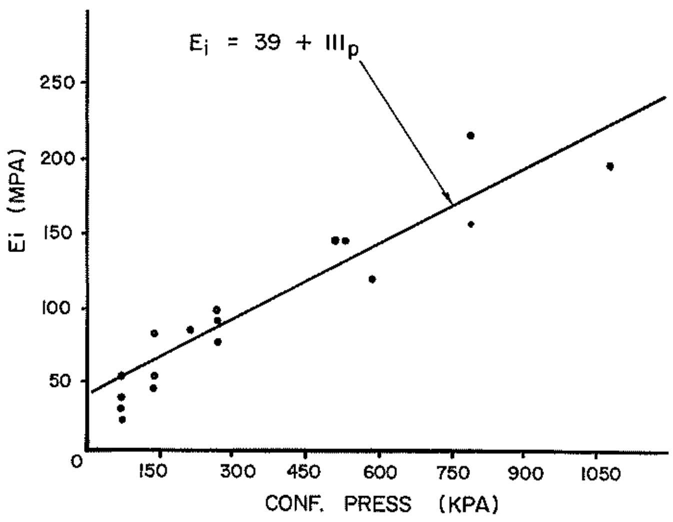


Fig. 1 Initial tangent modulus vs confining pressure

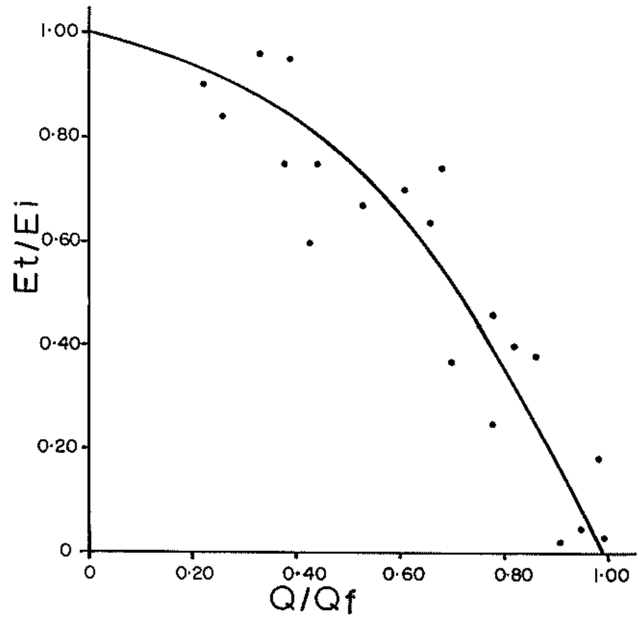


Fig. 2 Tangent modulus variation with stress

Stress-strain plots from individual triaxial tests typically showed an initial linear portion (from which the initial tangent modulus E_i could be computed) and a subsequent non-linear portion leading to failure. Tangent moduli (E_t) were calculated at selected points on each stress-strain curve and plotted as a function of principal stress difference. The following relationship (Eqtn. 3.2) was found to exist:

$$E_t = E_i \ (1 - s^n) \tag{3.2}$$

where E_t and E_i are tangent modulus values

and $s = \frac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_f} = \frac{Q}{Qf}$

and f denotes failure.

The value of the exponent 'n' was found by manually fitting the curve given by equation 3.2 to the plotted test results. A typical plot is presented in Figure 2 for the foundation material. The laboratory testing and application is described in more detail by Parton (4).

The relationship expressed by equation 3.2 is worthy of further discussion at this point. Duncan and Chang (5) have used the hyperbolic stress-strain relationship proposed by Kondner (6) to develop a relationship between tangent modulus and principal stress difference. Briefly, the relationship proposed by Duncan and Chang may be expressed as:

$$E_t = E_i \ (1 - R_f s)^2 \tag{3.3}$$

where R_f is a factor relating the assymptotic (hyperbolic) principal stress difference to that at failure.

The tangent modulus values given by expressions 3.2 and 3.3 are compared in Figure 3. It is interesting to note that equation 3.2 gives a distribution of modulus with stress rather different from that predicted for the hyperbolic stress-strain relationship. The 'n' values for the Pukaki materials were found to be in the range 0.9-2.0. Duncan and Chang found for the sands and clays they tested that R_f values were in the range 0.7-1.0.

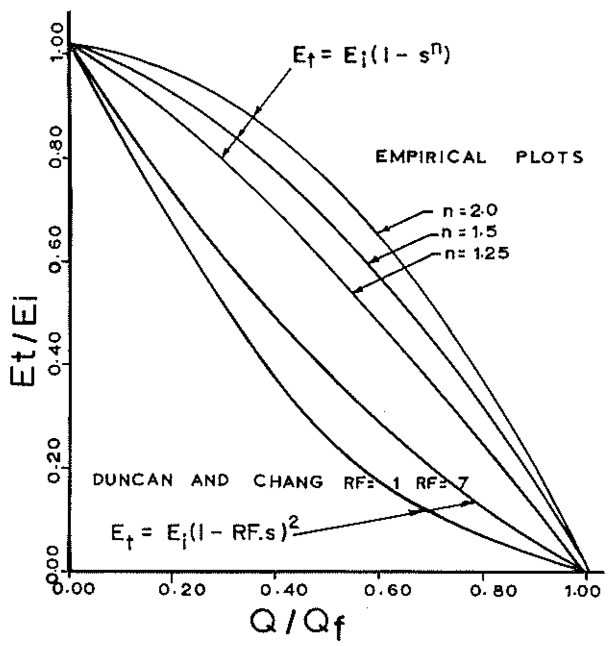


Fig. 3 Comparison of modulus variation

Attempts to determine Poisson's ratio for the specimens did not provide consistent or satisfactory results. Instead, it was decided to study the sensitivity of the analysis to Poisson's ratio and use values considered representative.

4 FINITE ELEMENT PROGRAM DEVELOPMENT

The FE computer program used to analyse stresses and deformation within the dam and foundation was developed from an existing FE program (Hollingshead and Raymond (7)) capable of handling materials with a multi-linear stress-strain specification. However, the existing program did not have the capability to model incremental construction of the dam (simulating actual construction) nor did it have the facility to handle a material specification as described in the previous section.

Studies by Clough and Woodward (8) and Kulhawy, Duncan and Seed (9) have shown that it is desirable to use an incremental loading procedure rather than a simpler single lift analysis in which the gravity body forces are applied to the entire structure at one time. Their studies indicate that while reasonable stress distribution may be obtained for homogeneous embankments by the latter method, displacement patterns considerably different from those calculated by incremental analyses and measured in real embankments may be obtained.

When calculating values of mean principal stress and maximum principal stress difference to compute the elastic modulus for the succeeding increment three possible alternatives exist; "past stresses" from the previous iteration may be used, "present stresses" from the new load application may be used, or "average stresses" from "past" and "present" may be used. "Average stress" solutions provide the most accurate results with minimum increments although "past stress" solutions provide acceptable accuracy if sufficient increments are used.

Necessary modifications were made to the program to enable incremental construction to be simulated and tangent modulus values for each element to be calculated from the element stresses. Because of severe restrictions existing on available computer time at that time, it was decided to use the "past stress" solution. A fixed value of Poisson's ratio was input for each element and, because failure was not approached, this was considered satisfactory.

5 NON-LINEAR ANALYSIS OF DAM AND FOUNDATION

(a) Finite Element Mesh

The finite element mesh used for incremental analysis of the dam and foundation is shown in Figure 4. The configuration of the mesh is such that the irregular shape of the core and shoulders may be included. Different material specifications were used for the foundation, core and shoulders.

As the effect of foundation depth upon computed stresses and displacements was unknown initially, several meshes were analysed. Results showed that even for very deep foundations, all significant displacements occurred within the top 60 m.

(b) Computed Vertical Displacements

The effect of the number of lifts used in the analysis was investigated. Analyses using 1, 5 and 10 lifts showed that the vertical displacement patterns for the extreme cases showed different mechanisms. The single lift analysis shows vertical displacements which are a maximum at the top (being the integrated strains developed over the full height) while the 10 lift analysis shows minimum displacements at the top with maximum displacements being developed in the lower part of the structure. These findings are in agreement with those reported by Clough and Woodward (1966).

The Pukaki analysis (Figure 5) showed that because the foundation is relatively flexible, the contour of maximum displacement extends into the foundation. The effect of the non-linear stress-strain behaviour can be gauged by examining the vertical settlement profile of the dam foundation interface. An analysis simulating a 10 lift embankment construction was performed with the foundation element moduli calculated from the in-situ stresses (K_o assumed to be 0.5), so that the foundation properties were linear but non-homogeneous. The results of this analysis were compared with the full non-linear case, as calculated from Equation 3.2. This comparison showed that the shape of the deflection profile was the same for both cases, but the maximum settlement was increased by 30% as a result of accounting for the non-linear behaviour.

(c) Computed Horizontal Displacements

Horizontal displacements at the interface between the dam and foundation were also of interest. A number of analyses were performed with various values of Poisson's ratio. Both the magnitude and direction of the horizontal displacement were found to be sensitive to the value of Poisson's ratio used. However, the magnitude of the greatest horizontal displacement was only about 5% of the maximum vertical displacement. For this reason horizontal displacements were not considered further and a fixed value of 0.3 was used for Poisson's ratio.

(d) Computed Stresses Within the Dam

Major and minor principal stresses are obtained from the computer analysis. Generally the principal stress contours tend to follow the shape of the dam, values being slightly less than the over-burden pressure at the centreline and slightly greater than over-burden pressures beneath the outer portion of the slope. The maximum principal stresses are within the range of cell pressures used in the laboratory testing.

An indication of the severity of the stress conditions within the dam may be obtained if the principal stress differences are replotted in terms of percentage of strength mobilised. Percentage of strength mobilised has been defined as:

$$\frac{(\sigma_1 - \sigma_3) \text{ mobilised}}{(\sigma_1 - \sigma_3) \text{ failure}} \times 100\%$$

The value of the principal stress difference at failure may be determined from the geometry of the Mohrs circle diagram if it is assumed that failure in a given element occurs with no change in the value of σ_3.

The contours of strength mobilisation (presented in Figure 6) show increasing values towards the centre of the core. The relatively low values suggest that non-linear deformations are not large within the core.

The highest mobilisation factor computed within the dam-foundation mesh is 55%. This corresponds to a factor of safety against local failure of 1.82. It is of interest to note that the contours of strength mobilisation in the dam shoulders approximate to circular arcs, with mobilisation factors of 50%. Circular arc stability analyses performed during the design stage showed static factors of safety slightly in excess of 2.

Finally, one of the basic objectives of the study was to check for areas where tensile stresses existed. The core material, being non-plastic in nature, was thought to be susceptible to tensile cracking. No zones of tensile stress were identified in the analysis.

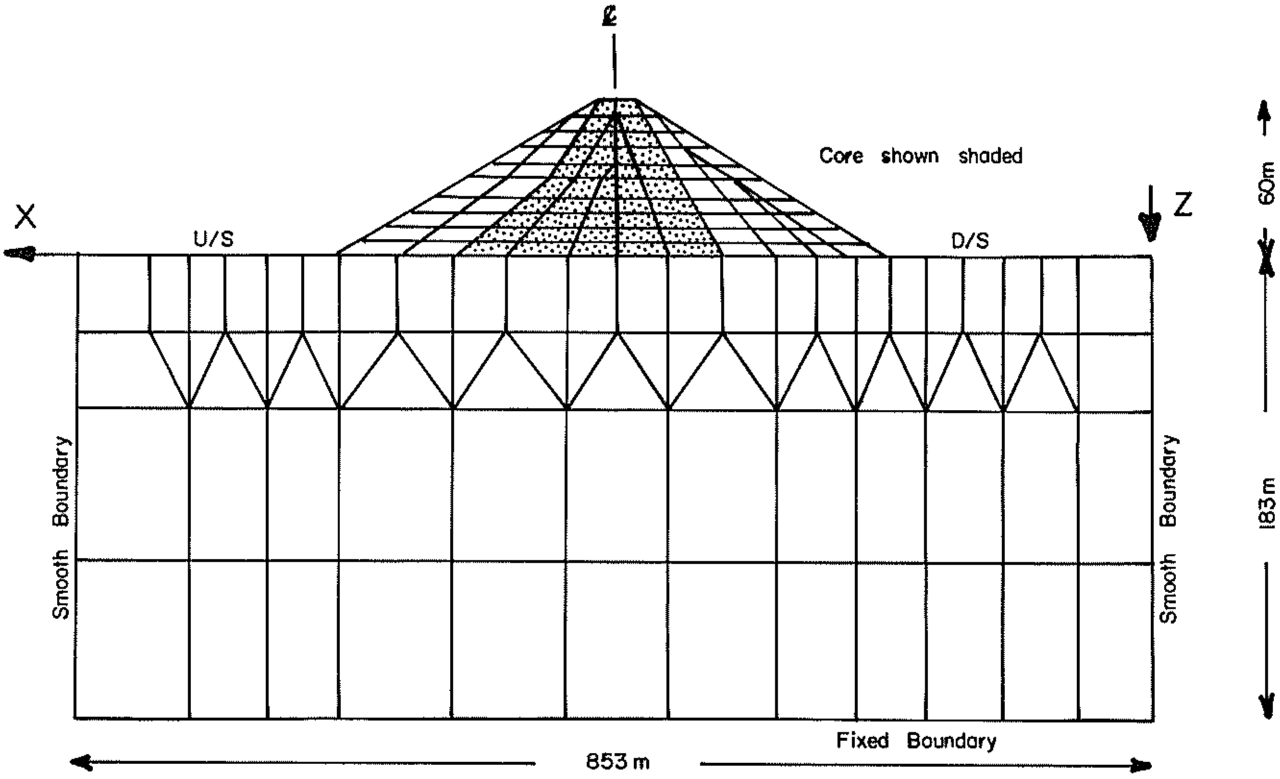


Fig. 4 Finite element mesh

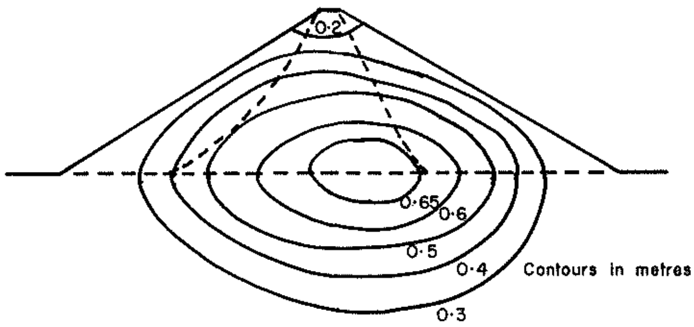
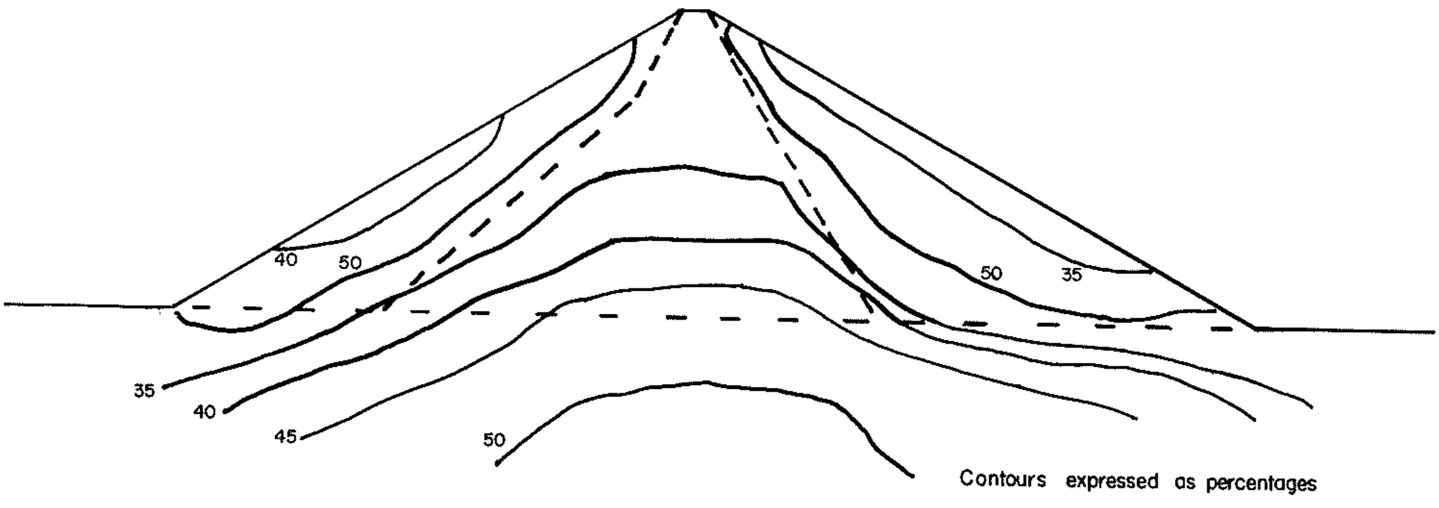


Fig. 5 Settlement contours



$$\text{Percentage of strength mobilized} = \frac{(\sigma_1 - \sigma_3)\,\text{mobilized}}{(\sigma_1 - \sigma_3)\,\text{failure}}$$

Fig. 6 Strength mobilisation contours

6 CONCLUSIONS

The studies described in this paper indicate that the results of non-linear finite element analyses, conducted using properties measured under appropriate laboratory test conditions, can provide information which is useful both in design and construction of a large earth dam.

The study is of particular interest because the dam is founded upon a considerable depth of glacial material, much of which is not able to be sampled or recovered by conventional methods. The study was able to provide a good "feeling" for the manner in which the dam is likely to behave and the significance of related variables. It is the opinion of the authors that the study reported herein provided a rational method of using the test information available to assess likely foundation effects and the effect on the dam.

The materials used in the construction of the Pukaki Dam show stress-strain behaviour significantly different from that published by Duncan and Chang (5). It is apparent that not all materials exhibit the hyperbolic stress-strain behaviour as has been suggested.

Deformation studies have been focussed upon vertical movements. Studies by other writers have shown that this variable may be predicted with best accuracy. It is considered unlikely that a vertical foundation deformation of 0.7 m will be exceeded. Using non-linear techniques the actual deformation within the dam is dependent upon the construction sequence simulated.

Finally, determination of strength mobilisation factors has enabled the severity of the stress conditions within the dam to be assessed. The close agreement between the circular arc stability analysis and the finite element analysis highlights an important use of the finite element method which is being used increasingly in geotechnical analysis.

7 ACKNOWLEDGEMENTS

The permission of the Commissioner of Works to publish this paper is acknowledged.

8 REFERENCES

1. PENDER, M.J. and PARTON, I.M. "Finite Element Analysis of the Pukaki Dam and Foundation, Part I, Introduction and Foundation Analysis". MWD Central Laboratory Report No. 416, 1972.

2. PARTON, I.M. "Finite Element Analysis of the Pukaki Dam and Foundation, Part II, Non Linear Dam and Foundation Analyses". MWD Central Laboratory Report No. 416, 1973.

3. READ, S.A.L. "Preliminary Geological Report on the Pukaki High Dam Site Covering Geotechnical Investigations since 1964 and Geology Exposed During Excavations to Early August 1972", 1972. Unpublished N.Z. Geology Survey Report, Engineering Geology Section.

4. PARTON, I.M. "Further Testing of Undisturbed Sandy Silt from the Site of the Pukaki High Dam". MWD Central Laboratory Report No. 450, 1972.

5. DUNCAN, J.M. and CHANG, C.Y. "Non-Linear Analyses of Stress and Strain in Soils". Proc. A.S.C.E., Vol. 96, SM 5, September, 1970.

6. KONDNER, R.L. "Hyperbolic Stress-Strain Response: Cohesive Soils". Proc. A.S.C.E. Vol. 89, SM 1, 1963.

7. HOLLINGSHEAD, G.W. and RAYMOND, G.D. "Prediction of Undrained Movements Caused by Embankments on Muskeg". Canadian Geotechnical Journal, Vol. 8, p.23, 1971.

8. CLOUGH, R.W. and WOODWARD, R.J. "Analysis of Embankment Stresses and Displacements". Proc. A.S.C.E., Vol. 93, SM 4, 1967.

9. KULHAWY, F.H., DUNCAN, J.M. and SEED, H.B. "Finite Element Analyses of Stresses and Movements in Embankments During Construction. Report No. TE-69-4, University of California, Berkeley, 1969.