The paper was published in the proceedings of the 7th Australia New Zealand Conference on Geomechanics and was edited by M.B. Jaksa, W.S. Kaggwa and D.A. Cameron. The conference was held in Adelaide, Australia, 1-5 July 1996.

Three-Dimensional Finite Element Analysis of Warragamba Dam

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Summary Nowadays, the availability of powerful desktop workstation and efficient finite element modelling software, which utilise three-dimensional real time graphics, enables a real-world complex problem to be simulated. This paper reviews some of the early physical and numerical models used in the analysis of Warragamba Dam. Then, the paper will describe recent finite element modelling technique involved in the analysis of the dam. The simulations of the original dam construction as well as future dam raising were carried out. Some possible scenarios namely, foundation softening, stress redistribution due to grouting of twist joints and rock creep were investigated in order to explain the occurrence and the closure of a crack which appeared in the upstream face of the dam three years after the completion of the dam in 1960. The behaviour of the raised dam under seismic activities was also investigated. The computed results were compared with observations and reasonable agreement in terms of geostatic stresses and dam deflections was obtained. Some interesting three-dimensional cross-valley action and stress arching were observed from the computations.

1. INTRODUCTION

Warragamba Dam, which is owned by Sydney Water, was to be upgraded for both dam safety and flood mitigation purposes. The option to raise the concrete gravity dam by 23m is now not to proceed by direction of the NSW government. Sydney Water is now preparing an EIS for an auxiliary spillway to satisfy dam safety issues. The behaviour of the existing dam due to the raising was investigated using three-dimensional finite element analysis (FEA). In order to establish the existing stress regime in the dam, the construction history of the dam had to be modelled as closely as possible. Much of the information regarding the geological conditions, the original dam design and construction was documented by Nicol (1964). A more recent raising of the dam by 5.1m and the proposed flood mitigation scheme were reported by Snape et al (1991and 1994).

2. A BRIEF REVIEW OF EARLY MODELLING TECHNIQUE

In the 1940s, the (then) 137m high dam was designed as a gravity structure which incorporated a thrust trench at the toe of the dam because of the low modulus Hawkesbury sandstone foundation. The original dam design used a Lattice Analogy Method and Trial Load Method for stress analysis. The former was used to analyse only one slice of the deepest cantilever in two-dimensions while the latter method was used to assess the horizontal beam action across the narrow gorge. Two physical models namely, a rubber model and a brass and Bakelite model, were built to investigate stress-strain behaviour in the dam. In the rubber model, two types of rubber were used to give the required concrete to foundation modular ratio of 15:1. In the brass and Bakelite model a modular ratio of only 8:1 was achieved. This model was also used to investigate the stress transfer between the dam, foundation rock and the concrete apron when the closing gap between the dam and apron was grouted.

In the early 1980s, it was recognised that a 2D FEA would not be capable of adequately modelling the behaviour of the dam. Therefore, a preliminary 3D FEA with 320 solid elements was carried out to investigate the effect of the Probable Maximum Flood (PMF) on the proposed raised dam. This simplified model assumed symmetry about the centreline of spillway. Although the inferred geological cross-section across the face of the dam revealed the geometry of the model was far from symmetrical, this approach reduced the number of elements used in the computation and hence decreased the solution time for the hardware available at the time.

In the simplified FE model, the concrete/rock interface was crudely modelled as a wedge shape using degenerated 8-node solid elements because tetrahedral elements were not available. The extent of the model was limited to about 100m in all directions away from the dam. Various geological units in the foundations were not included. The pore pressure inside the dam and foundation were not considered in the analysis.

The vertical twist joints between blocks of concrete in the original construction of the dam were not modelled. Instead, their influence on the dam was modelled by using alternate loading and changing stiffness. Stress-free element additions were not used to represent the building of the concrete dam. There was no attempt to model the formation of an upstream crack which occurred some time after the completion of the dam and there was no verification of the model which is a very important aspect of FEA.

At the time, the pre- and post-processing facilities and computer speed were very limited by today's standard. This severely limited the generation of any complex 3D mesh. Results visualisation was non-existent except using manual plots. Given the above limitations and the coarseness of the mesh, any detailed analysis of the dam was almost impossible.

3. FINITE ELEMENT MODELLING

Significant advances in mesh generation technique have been made and commercial software is readily available to design engineers. Although much of this has been used in mechanical engineering applications, there is an increasing usage of this technique in geomechanical analysis in recent years. For example, in the FEA of underground structures reported by Grabinsky et al (1993).

In the present analysis, the geometric modelling and mesh discretisation of Warragamba Dam were carried out using the MSC/Patran modelling software. The hardware graphics capability of the Silicon Graphics computer allows real time rotation of the model enabling geometry and meshing creation to be preformed efficiently. The numerical computation was carried out by a general purpose FE software, ABAQUS, on a IBM RS6000 workstation.

3.1 Geometry Creation

The topography covering about half a kilometre upstream (U/S) and downstream (D/S) of the dam was digitised from a contour plan. The geometry of the dam/rock foundation interface (ie. the excavation boundaries) was taken from a series of drawings showing a number of cross-sections along the dam. From these digitised points, contour lines were created using a number of curve fitting routines. And from these lines, smooth surfaces were generated. Finally, the solid model of foundation and dam was generated from these surfaces.

The foundation solid consists of five geological units. The orientation of each unit's interface was interpolated within the area bounded by drill-hole information and extrapolated beyond.

3.2 Mesh Generation

Within each solid, the density of the finite elements could be selected. A high mesh density within the dam and its immediate surrounds was generated and the mesh transition in the three directions could be achieved without much manipulation. The nodal coordinates and nodal connectivity information were calculated automatically from the solid model.

Over 30,000 elements were employed in the mesh. In addition to the 3D elements, other elements were also used to represent various aspects of the dam. The foundation rock and concrete dam were modelled using tetrahedron, prism and brick elements. Two-dimensional triangular and quadrilateral elements were used to model the training walls and water mass on the U/S face of the dam (for earthquake analysis). Two-node line elements were used for modelling post-tensioning cables and scalar elements were used to represent the mass of structure not included in the model. A perspective view of the mesh is shown in Figure 1. In addition to this large model, a smaller FE mesh with 1,197 elements was used for sensitivity studies and various modelling techniques were carried out in a much shorter time than for the large model. The appropriate modelling technique was then applied to the large model.

The base of the model which was about 200m below the dam was fixed in all direction. The vertical boundaries of the model were on roller supports.

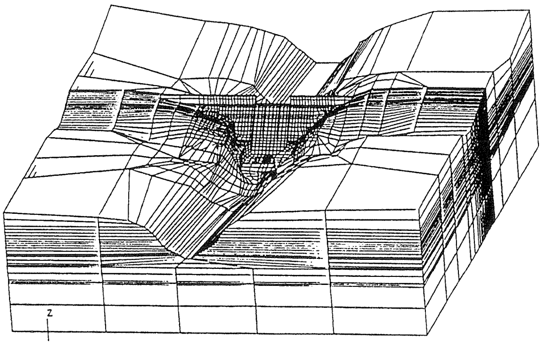

Figure 1. FE mesh of the model looking from downstream to upstream direction

3.3 Loadings

The loadings involved in the analysis were the gravity loads due to the self weight of the foundation and the concrete, the hydrostatic pressure on the dam and foundation and the post-tensioning loads in the cables. Pore water pressure distribution within the dam and foundation was calculated by a separate thermal analysis in which the pore pressure was represented by temperature and the equivalent values of thermal conductivity to represent the permeability were used. This technique was validated for the small model by using the transient seepage capability in ABAQUS. The effect of the grout curtain and internal drains within the dam wall were also taken into consideration. The pore pressure was then modelled as bodyforces. This approach had been described in detail by Nayler et al (1981).

3.4 Joints

Six vertical twist joints were used during the construction of the dam in order to allow cantilever action to develop, and to reduce shear-twist action. They were only grouted when the whole dam was completed and the storage water level was about three-quarters of the full storage level (FSL). Therefore, it was important to model these twist joints in the 3D model. A 2D plane strain FEA would not be able to model the true behaviour of the dam.

In the 3D FEA, the original dam construction was modelled as two lifts. Each lift consisted of the addition of a series of blocks of concrete elements. To ensure the concrete blocks were in a stress free state and no interactions develop between the concrete blocks across all the contact surfaces, multi point constraints equations were applied to the nodes situated at these surfaces. This technique was used successfully in a rock/structure interaction analysis for 3D tunnelling by Chen (1993).

In the present model, these equations were used in the following regions: the closing gap between the dam and apron, between concrete blocks and rock foundation, in the vertical twist joints, horizontal contact surfaces between the first and second lifts of concrete blocks and between concrete blocks and rock abutments.

3.5 Material Properties

The material properties for the six units of sandstone, chocolate shale and the grey shale bands are given in Table 1. There are a number of thin horizontal shale bands in the sandstone units. In order to determine the best technique to model these thin layers of shale bands, which exhibit nonlinear shear behaviour, a simplified 3D FE model was used. However, after a number of sensitivity studies using both interface elements and 3D elements to model the shear bands, it was found that these shear layers in the dam abutments were very unlikely to fail and their influence on the behaviour of the dam was not significant, therefore they were not modelled in the analysis.

3.5.1 Foundation softening

The physical testing for the two sandstone units prior to and during dam construction had shown that the foundation rock had a much lower elastic modulus than that of the abutment. In September 1963, about three years after the completion of the dam, a horizontal crack occurred across the U/S spillway blocks causing much leakage. This crack gradually closed over the next 18 months. In order to mimic such an event in the FEA, a probable scenario of foundation softening due to "wetting" was investigated. Although the softening of foundation is a gradual process, it was modelled as a time independent event in the analysis.

There is limited published information on the effect of saturation on stiffness. Pells (1993) has shown that such an effect is variable. More recent laboratory testings have indicated that the majority of the tested samples show a reduction in stiffness (up to 55% reduction) when saturated (see Table 2). This value of reduction has been adopted in the analysis and the extent of the softening was assumed to be confined to the U/S foundation.

3.5.2 Creep

The creep behaviour of the sandstone was modelled using the following power law relationship,

$$\dot{\varepsilon}_{cr} = A\sigma^n t^m \qquad (1)$$

where $\dot{\varepsilon}_{cr}$ is the uniaxial equivalent creep strain rate, σ is the equivalent stress (Mises or Hill's definition), t is the total time and A, n and m are parameters obtained by fitting the test data.

Although short duration (22 days) creep tests were carried out (Pakchung, 1946), they were inappropriate to simulate long term (over 30 years) creep. A long term movement monitoring of the dam gave a better indication of the creep behaviour. The history of the vertical movement near the base of the dam since the dam completion has been monitored since early 1962. Despite some fluctuations in the upward movement, a general trend can be postulated: a heave of about 3mm underneath the dam over a 30 year period was assumed in the creep model.

3.6 Geostatic stress simulation

During rock excavation for the dam foundation, large lateral displacements of drilled holes and vertical excavation walls were observed which suggested that a significant horizontal stress field existed in the rock near the bottom of the valley. More recently, an in-situ stress measurement programme carried out by CSIRO in the vicinity of the dam site has shown the presence of significant near surface horizontal stresses (Enever et al, 1988). Braybrooke (1994) suggested that the range of near surface stresses in the Hawkesbury Sandstone within the Central Business

Table 1. Material properties.

Material	Description	Material Model	Young's Modulus (GPa)	Poisson's ratio	Unit weight (kN/m^3)	Permeability (m/s)	A	n	m
Unit 1	Bulgo Sandstone	Isotropic elastic	2.5	0.1	24	3E-8	3.12E-27	3	-0.8
Unit 2	Bald Hill Clay-stone (Chocolate Shale)	Anisotropic elastic	9.6 (in plane) 3.2 (⊥ to laminations) G = 6.67GPa	0.1	24	3E-8	3.12E-27	3	-0.8
Unit 3	Coarse grained Sandstone & Conglomerate	Isotropic elastic	2.5	0.1	24	3E-8	3.12E-27	3	-0.8
Unit 4	Hawkesbury Sand-stone - fine grained sideritic	Isotropic elastic	2.5	0.1	24	3E-8	3.12E-27	3	-0.8
Unit 5	Hawkesbury Sand-stone - fine to coarse grained	Isotropic elastic	6.9	0.1	24	3E-8	3.12E-28	3	-0.8
Unit 6	Grey shale bands	Not modelled	-	-	-	-	-	-	-
Concrete	-	Isotropic elastic	40	0.1	24	1E-12	-	-	-
Grout	Grout curtain	-	-	-	-	3E-9	-	-	-

Note: A, n and m are creep parameters

Table 2. Saturated and Dry Tangent Moduli @ 0.5UCS of Sandstone

Location	Material	Dry E_d (MPa)	Sat. E_s (MPa)	Ratio E_s/E_d (%)
Bondi	Massive, fresh	13.8	8.1	59*
Waterloo	Laminated, fresh	10.3	11.0	107*
Waterloo	Bedded, fresh	11.6	8.7	75*
Kirribilli	Laminated, SW	4.8	5.0	104*
Kirribilli	Thin bedded,SW	12.0	8.4	70*
Fr Forest	Thin bedded,MW	12.7	8.6	68*
Elizabeth St.	Thin bedded, fresh	11.7	13.9	119*
Warragamba Dam +	Massive, fresh	6.96	4.04	58
Warragamba Dam +	Current bedded,MW	13.79	15.75	114
Warragamba Dam +	Current bedded,MW	15.64	8.55	55
Warragamba Dam +	Current bedded,MW	11.52	7.68	67

* after Pells (1993). + Public Works Dept. (1987).

District of Sydney was given by:

$$\sigma_h = (0.5 \text{ to } 0.6)\text{MPa} + (2.0 \text{ to } 2.5)\ \sigma_v \quad (2)$$

where σ_h = horizontal stress and σ_v = vertical stress.

In the present FEA, the geostatic stress state in the foundation due to the original valley erosion and the actual excavation for the dam construction was established by assigning the following initial stresses to the rock elements:

$$\sigma_v = \rho g h \quad (3)$$
$$\sigma_h = 0.5\text{MPa} + \sigma_v \quad (4)$$

where ρ = rock density, g = gravity and h = depth below a datum level of RL180. These stresses were allowed to redistribute during the first step of the analysis. The horizontal stresses and the orientation of the principal stress would be influenced by the topography of the FE model.

3.7 Dynamic analyses

Two linear dynamic analyses were carried out on the 23m raised dam. A conservative 5% damping representing energy dissipation within the concrete dam due to viscous damping was assumed. The constitutive relationship in the concrete and rock remained the same as for the static condition. In order to avoid reflection of vibratory energy back into the dam foundation from the model boundary, the foundation rock was assumed to be massless. In this case, no vibration waves could be developed within the foundation and no energy could be transmitted away from the dam (Zienkiewicz et al, 1986). This also filtered out the eigenmodes in the relatively soft rock foundation.

The reservoir interaction on the dam was modelled using equivalent added mass based on Westergaard (1931). In this analysis the interaction of the storage and the dam was not considered. The dam was assumed to be rigid with respect to the storage and the water was assumed to be incompressible. The mass of water was modelled by placing 2D shell elements against the U/S face of the dam. The nodal thickness of these elements was varied such they produce the equivalent water mass on the dam. The water density was assigned to these elements and their modulus was set to a negligible value.

A total of twenty eigenvalues and eigenvectors were extracted for the response spectrum and modal transient dynamic analyses. The spectral acceleration was based on the response spectrum for a return period of 500 years and a probability of exceedance of 0.5 for the Warragamba Dam site (Gibson, 1995). For the modal transient dynamic analysis, an accelerogram obtained at D/S of Pacoima Dam in the Northridge earthquake on 17 January 1994 in California was used. The peak acceleration (0.425g) was of the same order as for a 1 in 10,000 years return period for the Warragamba region. For a high hazard dam like Warragamba Dam the maximum design earthquake (MDE) is likely to have an annual exceedance probability the order of 1 in 100,000 years. This would mean scaling up the accelerations of the Pacoima Accelerogram by two times. This amount of scaling is not considered to be credible at this stage. The unscaled accelerogram was therefore analysed as a first step in simulating the MDE. The acceleration was applied to all fixed boundaries in the U/S-D/S direction which was considered to be the weakest direction of the dam under earthquake loading.

The behaviour of the dam was observed by superimposing results from the dynamic analysis onto those computed from the static analysis.

3.8 Analysis Steps

The simulation of the dam construction and various loading stages were carried out in a number of analysis steps. They are listed as follows:

Original dam construction:

1. Establish geostatic stress state in foundation.
2. Add first lift of concrete elements to RL53.
3. Apply water load (RL53) to the dam.
4. Add second lift of concrete elements to the dam's full height.
5. Apply three-quarters of FSL.
6. Grout all vertical twist joints and apply additional water load (RL107.6).
7. Apply FSL (RL116.7).
8. Transient pore water flow in the dam and foundation.
9. Foundation softening.
10. Creep simulation.

Recent dam raising (interim works):

11. Raise dam crest by 5.1m by adding concrete elements. Install post-tensioned cables and apply post-tensioning loads.
12. Study effect of the original design super flood (RL125).
13. Study effect of the existing design flood (RL130.4)

23m dam raising proposal:

1. Repeat steps 1 to 11 above.
2. Add new concrete elements to the downstream face and crest in three stages as shown in Figure 2.
3. Extend existing post-tensioned cables and reapply post-tensioning loads.
4. Increase water load to simulate flooding (RL153).

Dynamic analyses of the raised dam:

1. Eigenvalues and eigenvectors extraction.
2. Operating basis earthquake (OBE) of 1 in 500 years with storage (conservatively) at the PMF level using an acceleration response spectrum solution.
3. Initial MDE of 1 in 10,000 years with storage at FSL using a transient dynamic solution.

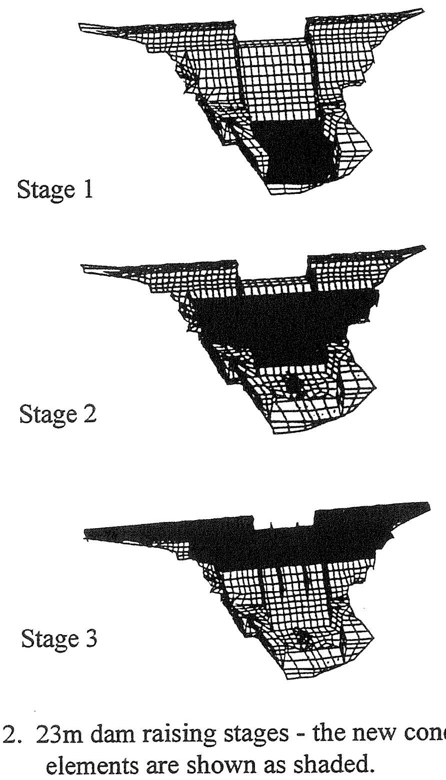

Figure 2. 23m dam raising stages - the new concrete elements are shown as shaded.

4. ORIGINAL DAM CONSTRUCTION SIMULATION

4.1 Geostatic stresses

The major and minor horizontal principal stresses at three locations not far from the dam site after the geostatic stress state were compared. In general, the computed stresses were in good agreement with the measured values even though the topography of the 3D model was somewhat simplified.

4.2 Crack simulation

One of the objectives in the FEA was to simulate the formation and closure of a horizontal crack on the U/S face of the spillway section of the dam. One possible scenario was due to foundation softening in the rock U/S of the dam. When a 55% stiffness reduction was applied to the large FE model, a reduction in the compressive stress was computed at the U/S base of the dam. The actual crack occurred at about 50m above the base of the dam which may probably have been at a weak junction between successive concrete pours. It can be demonstrated that when the foundation softened, the base of the dam settled and tension stress could develop on the U/S face of the dam. The actual extent of the softening and the percentage reduction in stiffness could be difficult to determine.

The effect of the grouting of the vertical twist joints was investigated. For the ungrouted case, the cantilever bending action resulted in a peak tensile stress increment at the base of the dam. However, the grouted model shows the high tensile stress increment took place at the mid-height of the dam close to where the crack occurred. Also according to the incremental cross-valley stress across the width of the dam, the grouted dam behaved in a combined cantilever and 3D beam action manner. These results demonstrated the importance of the 3D model and introduced another scenario for cracking. The timing of grouting up the twist joints relative to the rising water level was important and it could be possible that tensile stresses at the crack level could be due to a much larger stress increment. The timing of pore water pressure uplift forces, which really were continuous with dam filling, would also influence the stress levels in the dam.

The closing up of the crack may probably be attributed to a number of possible causes such as the readjustment of the pre-construction stress, tectonic forces, and creep movement. In the present analysis, the U/S face of the dam became more compressive as the base of the dam heaved upwards due to creep. It was therefore possible that the crack closure was caused by this creep movement.

There have been many speculations as to the cause of this crack opening and closing (Carter, 1972). The present 3D FEA demonstrated some possible scenarios in which this event could be simulated. A more important aspect of the FEA was to compare the computed results with the actual performance of the dam in order to validate the FE model so that the design adequacy of future dam raising, or current dam under earthquake loading, could be investigated thoroughly.

4.3 Comparison with observations

In November 1961, a major flooding took place. The maximum water level was recorded at RL119.5. The lateral deflection of the dam at the centre section was recorded prior to and during the flood (water levels at RL106 and RL112.7 respectively). This change in water level was similar to that between analysis steps 6 and 7 in the analysis. The incremental lateral deflection profile is shown in Figure 3 and a good agreement was obtained between the computed results and the actual measurements.

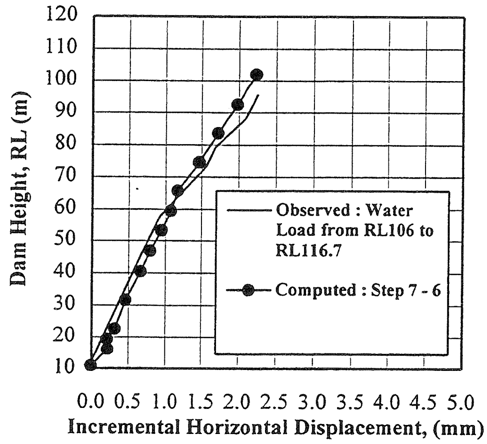

Figure 3. Incremental horizontal deflection profile.

The horizontal displacements on the D/S face of the dam were monitored at a number of stations since the completion of the dam. The history of one of them is shown in Figure 4. The water level record and the computed displacement results are also plotted on the figure. It can be observed that the dam moved D/S during the filling of the reservoir. When the water level was more or less at FSL over the next 17 years, the dam seems to move gradually in the U/S direction. This trend of horizontal movement was simulated by the FEA since the effect of the foundation softening assumption caused the dam to deflect U/S. In this analysis step, the softening was assumed to happen instantaneously. However, this probably took place over a period of time. Nevertheless, the results showed that the U/S movement could be due to foundation softening. The effect of creep on the horizontal movement was observed to be insignificant. Further U/S movements recorded between 1979 and 1983 were caused by the lowering of the water level. This was not modelled in the analysis. Similar results were also observed from the other monitoring stations.

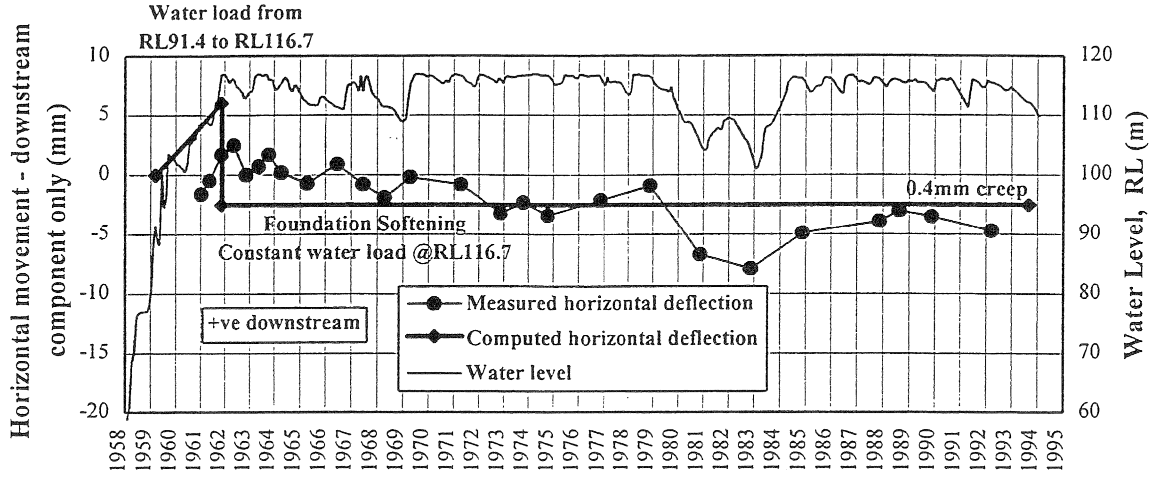

Figure 4. History of horizontal movement at Station T12A.

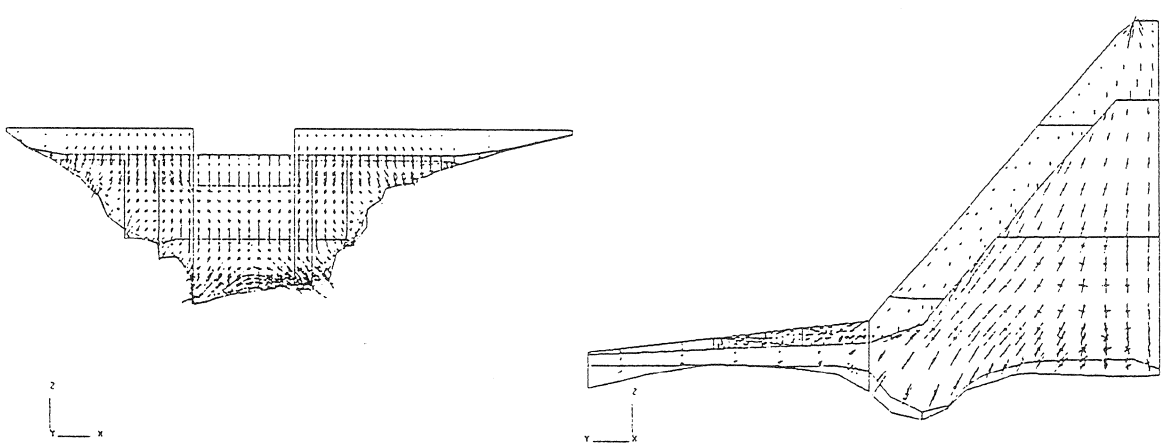

Figure 5. Principal stress tensors plots across the upstream face and through the spillway of the dam.

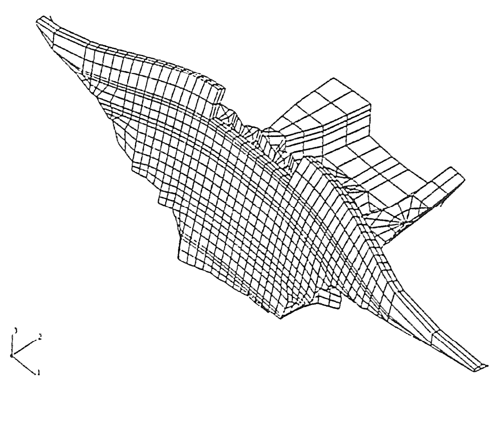

Figure 6. Fundamental mode shape at 1.5Hz.

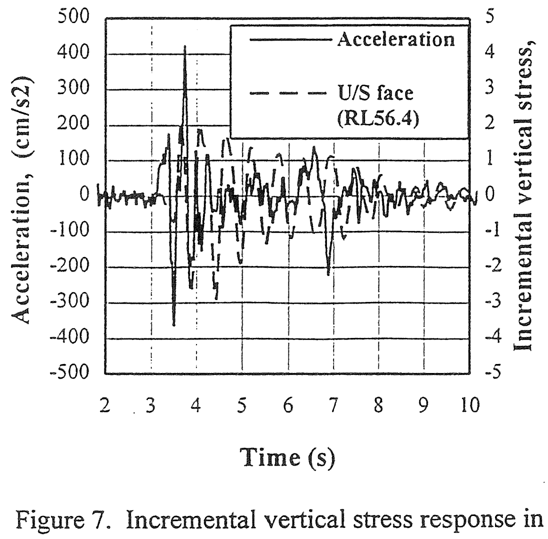

Figure 7. Incremental vertical stress response in the modal dynamic analysis.

5. RECENT DAM UPGRADE SIMULATION

The interim works construction (1987 to 89) increased the vertical compression stress on the U/S dam surface. The post-tensioned cables also provided some additional compression. Although stress measurements were not carried out, the computed results in terms of stress magnitude and location were in good agreement with the design assumptions for both flood situations.

6. 23m DAM RAISING SIMULATION

From the analysis, the dam remained in compression during the application of new concrete loads to the back of the spillway and abutments. A two-way stress arching in the spillway could be seen as shown in Figure 5 for the case of the PMF discharging from the spillway. It should be noted that the thermal and concrete shrinkage effects have not been incorporated in the present study. The behaviour of the raised dam under the flood was that of a complex 3D beam and cantilever action. However, no tension stress was computed on the U/S face of the dam.

7. SEISMIC ANALYSIS

The first twenty natural frequencies were in the range of 1.50 to 8.66 Hz. These values were consistent with those measured (peaks at 1.9, 2.4, 2.8, 3.3 and 8.1Hz) by the Dam Safety Group of Australian Water Technologies in 1991 in spite of the simplified dam geometry and the low concrete stiffness used in the FEA. The fundamental mode of the dam at FSL is shown in Figure 6.

Under OBE condition, the spillway remained in compression in general. However, under MDE condition, significant tension occurred and some cracking at the base of spillway would take place under this earthquake excitation. The incremental vertical stress response at the centroid of an element located half way up in the U/S spillway is shown in Figure 7.

8. CONCLUSIONS

The methodology of the 3D FEA, which utilised the latest software and hardware capabilities, for the potential raising of Warragamba Dam is described in the paper. The original dam construction, recent interim works raising, proposed raising and earthquake loadings have been successfully simulated. The model has been thoroughly validated against measured response and forms a firm basis for potential future design studies.

9. ACKNOWLEDGEMENTS

The authors wish to thank Sydney Water for allowing this paper to be published, Prof. John Carter from Sydney University for his advice and assistance in the selection of geotechnical models for the foundation materials and their client, the Dams & Civil Section of the NSW Department of Public Works and Services.

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