STRUCTURAL INTERACTION BETWEEN ARCH DAM AND VALLEY, WITH SPECIAL REFERENCE TO THE EL ATAZAR DAM

A thesis submitted for the degree of Doctor of Philosophy in the Faculty of Engineering of the University of London

by

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To my parents
ABSTRACT

This thesis describes experimental and theoretical work undertaken to assess the stability of the El Atazar Dam in Spain. The dam is a doubly-curved thick arch dam, and its site has given rise to considerable foundation problems.

Experimental studies which included two large geomechanical models were undertaken at the request of the Spanish authorities, the Confederacion Hidrografica del Tajo, and the designer, Professor J. L. Serafim of Consulpresa. Both models were to a scale of 1:200, and both consisted of a microconcrete dam in a realistically modelled valley. They were tested to destruction under simulated hydrostatic and gravity load. Strains and displacements were recorded.

The experimental work has been complemented by finite element analysis. The computer program described, was written to form the foundation of a general purpose system. Solution is by the frontal technique, which proved to be efficient in solving the many three-dimensional analyses undertaken. Three main analyses were made. The first was of the shell of the dam, rigidly supported at its periphery. The second analysis was of the shell and socket supported by a rigid valley. The third, and most complex analysis, was of the dam supported on a linear elastic foundation, which included idealisations of the major faults.

The work shows that dam failure is induced by excessive movement of the valley, and that valley strengthening is required. The behaviour of the dam in the second model was unexpected, but found to be consistent with tensile cracking on the upstream face at the left bank dam and valley interface. The dependence of dam safety on the self-weight of the valley, and the detrimental effects of hydrostatic load, are also illustrated.

These points illustrate the general conclusion: arch dams draw on their hyperstatic reserves to solve problems associated with their immediate foundation, but their overall safety depends on valley strength.
ACKNOWLEDGEMENTS

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CHAPTER 1: INTRODUCTION

1.1 ARCH DAMS AND FOUNDATIONS

The major consideration in the design of arch dams is the prevention of the various possible forms of structural failure. Both practical experience and laboratory studies have shown that arch dams have a very high factor of safety, because they draw on their hyperstatic reserves to solve the stability and strength problems caused by the unpredictable behaviour of the foundations. This is confirmed by the fact that only one failure of an arch dam has been reported. The failure was not, however, due to a fault in the design or construction of the dam, but to an unknown weakness in the rock foundation.

It is therefore important to study the stability of the valley under the combined effect of the loads transmitted by the dam and the water pressure due to seepage from the reservoir. The solution of these problems depends on a detailed geological site investigation in which all the rock defects, such as jointing and weathering, are recorded. Given this information, the most likely modes of failure can be ascertained.

This thesis is the outcome of such an investigation. It describes model tests and theoretical work undertaken to assess the stability of the El Atazar Dam in Spain. It will be confirmed that valley strength is essential for dam safety.

1.2 THE DEVELOPMENT OF ARCH DAM TECHNOLOGY (1-5)

Isolated examples of arched dams have been constructed since Roman times, notably in Spain and Italy in the sixteenth and seventeenth centuries, but no real progress in dam technology was made until the mid-nineteenth century saw the emergence of reservoir dams to supply water to towns in the developing industrial countries. Few arch dams were built for this purpose, though there is one in France which was built between 1843 and 1854. The Zola Dam, which bears the name of its designer, was the first arch dam to attract international attention to the advantages of this type of structure. It had a sloping air face, it was
36 m high, 6 m thick at the crest and 13 m thick at the base, and was, for the period, a notably thin arch dam. There is a suggestion that Zola carried out some calculations in order to select the form of the dam. If true, this would unquestionably be the first design calculation undertaken for an arch dam. Prior to this everything had been a process of structural evolution, the tendency to develop the theoretical ideal in the absence of analytical methods of design.

The second half of the nineteenth century saw the formulation of methods of analysing the stresses in masonry dams. The main focus of attention was on solving gravity dams, but Delocre and Pelletrau were involved in determining to what extent arch action could be relied upon in a curved dam. Their inquiries were based on the assumption that an arch dam could be treated as part of a complete cylinder, and that its thickness at any depth was given by the membrane formula $t = \frac{pr}{\sigma}$. 

In spite of the theoretical work and the existence of thin arch dams in Italy, France and Spain, arch dams were relinquished in favour of gravity and earth dams. However, a dam was built on the Rio Grande in Central America in 1888, and it was in North America and Australia that arch dams began to flourish towards the end of the century. Some of these were constructed of concrete and in many instances were more slender than the contemporary state of knowledge really justified.

The major developments in arch dam technology in the twentieth century have taken place in the United States and Europe. Each has formed its own approach to designing and analysing arch dams.

In the United States, the design of the Pathfinder and Shoshone Dams, which were completed in 1907 and 1910, represented a major advance. The method of analysis considered the interaction between arches and cantilevers, by combining the methods of analysing gravity dams as cantilevers, and arch dams as a number of horizontal arches. The water load acting on the dam was distributed between the two structural systems to give some compatibility in the deflections of the
arches and the cantilever.

In 1905, the radial deflections of each arch at its crown was calculated by the membrane formula, and that of the crown cantilever by simple beam theory. By trial-and-error, the water load was adjusted between the two systems, so that the arch deflections were equal to the deflection of the crown cantilever at each point of intersection. Developed by the Bureau of Reclamation, it became known as the Trial Load method and was further improved in 1929 by introducing elastic theory to calculate the arch deflections in conjunction with a series of cantilevers rather than just one. It was further extended in 1930 in order to design the Hoover Dam, when not only the radial deflections but also the tangential and twist deformations at each point of intersection were made compatible.

The Trial Load method never became popular in Europe. To European engineers the essence of an arch dam was that it should resist the loads by arching action, so they concentrated on designing arch dams on an independent arch theory. The dam was treated simply as a number of separate arches which were fixed at their ends and bore all the load. This enabled the dam's thickness, radius of curvature and central arch angle, to be varied at different levels to help produce the most effective shape. In France, Coyne proposed the concept of 'plunging arches', where the dam is divided into a series of sloping arches rather than horizontal ones. He used the concept to design the Roseland Dam.

The adoption of analytical techniques marks the beginning of the great developments which have taken place in relatively recent times in efforts to reduce the volume and cut the cost of dams. Before 1920, arch dams were generally cylindrical with a vertical upstream face and a sloping downstream face. After 1920, the radii of the arches began to be varied, and dams tended towards the type with a constant or nearly constant central angle for the arches. The characteristics of the arches developed rapidly, the vertical cross-sections passing from a triangular
form with a thick base, to a curved profile with the upper arches sometimes overhanging downstream at the crest. The plan consequently became more complex, and even more so with the principle of varying the thickness in the individual arches with appropriate thickening from the crown to the abutments. The simple arch dam was therefore rapidly transformed into a doubly-curved structure which, with a thinning of the arches, produced a shell shape which was eventually characteristic of cupola dams.

Although many dams had actually utilised the double action of an arch dam, the way in which the self-weight of the dam and the hydrostatic load have counteracting effects, it was only after 1945 that the concept was used in design to exploit further the behaviour of the cupola dam. However, because they were inexperienced with and reluctant to use the only really suitable method of Trial Load, European engineers turned to models as a basis for design.

In Portugal, experimental work involving large models has been carried out under Rocha at LNEC, Lisbon, with plaster as the main material. In Italy, under Fumagalii at ISMES, Bergamo, models were of microconcrete using cement and pumice. Interest in arch dams in England was stimulated by a report on an experimental and theoretical investigation carried out at Imperial College into the proposed Dokan Dam. A number of rubber models were made and manual relaxation techniques were used to solve the three-dimensional equations of elasticity. This was the first time this had been done to calculate the stresses due to water load and temperature effects for an arch dam. Later work at Imperial College included microconcrete models of designs for the Monar Dam and for the Arch Dam Research Committee of the Institution of Civil Engineers. The Committee was responsible for specifying five standard dams and a standard valley profile which would be useful for the comparison of various model and theoretical techniques. The various dams were analysed using the Trial Load method, three thin-shell approaches, dynamic relaxation and finite elements. Only the use of computers made
such an extensive theoretical study feasible.

As time passes, it becomes necessary to construct arch dams in places which might once have been ignored due to the unsuitable properties of the ground. The failure of the Malpasset Dam in 1959 drew attention to the fact that rock mechanics has an important role to play in the assessment of future dam sites. With better knowledge of the geology and the properties of the rock, these difficulties may be overcome with the application of better site construction techniques to strengthen the valley. The application of these techniques depends upon an increased analytical and laboratory facility using methods such as the finite element method, and complex models in which the dam, valley and major geological features are represented.

1.3 DAM INCIDENTS

The safety of an arch dam lies in its shape, and in its being a highly indeterminate structure. These two features enable the dam to overcome both inadequacies in calculation and imperfections in construction. Provided the dam is well designed, its safety does not rest in itself, but in the strength of the foundation.

ICOLD has found it useful to study failures and accidents to large dams in order to increase knowledge in dam building technology. They have reported 20 incidents occurring between 1900 and 1965 involving arch dams over 15 m high. Their report also covers a number of special cases concerning incidents to temporary cofferdams and spillways, major repairs and slides in reservoir banks, of which 10 involve arch dams.

The 20 incidents can be divided as follows:

1) Major failures which involved the complete abandonment of the dam; 4 incidents.

2) Failures which were severe at the time, but did permit the damage to be repaired and the dam again brought into action; 2 incidents.
3) Accidents to dams which had been in use for some time, but which were prevented from falling by immediate remedial measures; 9 incidents.

4) Accidents which had been observed during the initial filling of the reservoir and in which failure was prevented by immediate remedial measures; 4 incidents.

5) Accidents during construction which had been caused by settlement of foundations and local sliding failures, and which occurred before any water was impounded. After the essential remedial measures had been carried out, the reservoir was safely filled; 1 incident.

The 4 major failures involved the Moyie River, Vaughn Creek and Gallinas Dams in the United States, and the Malpasset Dam in France. The Moyie River failure was due to a spillway discharge affecting the foundation, the Vaughn Creek disaster resulted from internal water action in the foundation, and the Gallinas failure derived from poor maintenance and supervision.

The Malpasset Dam failed in 1959 and practically the whole arch was destroyed, the only parts remaining being on the right bank at the base of the central monoliths. On the left bank, a dihedral shaped gash approximately 40 m wide and 30 m deep appeared in the gneiss foundation.

Below the dam there was an impervious fault dipping upstream at an angle of 45° and crossing the valley almost perpendicularly. There were also many undetected potential shear planes dipping downstream, almost parallel to the arch near the left abutment. The design calculations showed that the wedge of rock, formed by the fault and any upstream shear surface, would be stable under normal loading conditions. This is not true, however, if the uplift acting upwards on the shear planes is considered. Due to the large area on which the uplift acted, the force was sufficient to lift the dam with its foundation. Failure occurred because the orientation
of the shear planes near the left abutment was such that the arch thrust, instead of spreading out into the foundation, was confined within a narrow section of the foundation, producing high compressive stresses.

Gneiss, which is moderately impervious in normal conditions, becomes completely impervious when compressed. Under the dam compression, it therefore became a watertight diaphragm and prevented any seepage. Under full hydrostatic pressure, and because of the low cohesive strength of the shear surfaces, the foundation moved a little downstream and a crack developed in the shear surfaces through which the uplift had the opportunity to act. A huge block of rock and the dam above it were violently lifted as the uplift forces suddenly penetrated the crack.

Under such circumstances, a grout curtain would not have prevented the disaster. Drainage galleries directed upstream, however, would have been effective and this is now often done as a result of the Malpasset failure.

One of the special incidents reported concerns the doubly-curved, 265 m high Vajont Dam, which was completed in 1960. During the initial filling of the reservoir, creep movements on a large area of the left bank of the reservoir near the dam, and a superficial slide, were observed. Along a perimeter crack, a rock mass some 2000 m long and 1200 m wide began to creep. As a result, the reservoir level was reduced slowly and the creep movements ceased.

In the summer of 1962, a second attempt was made to fill the reservoir, but when the water reached an elevation 26 m below the crest, creep began again, the reservoir level was reduced and the creep eventually stopped. A third attempt to fill the reservoir was made in 1963, but when it reached an elevation of 50 m below the crest, the mass started to creep again. When the water reached 15 m below the crest, the creep accelerated and a slow reduction of the reservoir level had no effect.
On the day of the disaster, the creep changed to a sudden slide, and an enormous mass of rock moved over a horizontal distance of 400 m including the 90 m wide gorge. It squeezed out a mass of water which, pushed to a maximum height of 260 m above reservoir level, naturally overflowed the dam.

The strength of arch dams is highlighted by the fact that the Vajont Dam was not damaged by the enormous load to which it was subjected when the water overflowed the dam, and showed no signs of distress from the new loads imposed on it by the solid material which replaced the water.

1.4 TECHNIQUES OF IMPROVING THE PROPERTIES OF ROCK MASSES

Techniques exist for improving the physical properties of rock masses. The two main techniques are pressure grouting and rock reinforcement.

1.4.1 Pressure Grouting

This involves injecting fluid grout into the rock mass, to replace all the air and water in the fissures and cracks, by a set product. The grout material usually consists of cement and water.

The Kariba, Mauvoisin and Dokan Dams are only three of a great number of arch dams possessing a grout curtain, a zone of rock which has been grouted to a considerable depth, length and width. Its purpose is to form a wall of low permeability within the rock mass and on the flanks of the dam. To a limited extent, it can also reduce uplift, and is especially effective when used in conjunction with drainage galleries which, because of the reduction in leakage, will keep the pore pressures down without causing unacceptable water losses. If high pore pressures are allowed to build up in the rock abutments near the valley surface downstream of the dam, there is a risk that land slips may occur, leading to progressive failure of the abutments.
The foundation of a dam should be waterproof and of adequate compressive strength, and also of reasonable homogeneity to avoid differential settlement. Grouting is often carried out to consolidate the foundation and improve the bearing capacity. In the process, however, the cohesion and angle of internal friction of the rock mass may also be improved. The improvement is generally measured in terms of the elastic moduli of the treated and untreated rock.

1.4.2 Rock Reinforcement

Rock bolts and stressed cables are the most useful forms of rock reinforcement. They are used to load the rock so that the shear resistance along the planes of weakness is improved. An additional method is the immediate application of a gunite layer to an excavated surface, to prevent superficial deterioration of the rock.

Rock bolts are used to prevent rock which has already fractured, from failing and falling away from the face of an excavation. It follows, therefore, that their main purpose is to stabilise areas where stress redistribution has occurred, leaving low pressures in areas of rock which have loosened or broken during the redistribution of pressure.

A typical use of rock bolting is in the roof of a tunnel where, due to the development of a tensile zone, the bolting should be designed to carry the weight of the unsupported rock. A second use is on rock slopes, to prevent blocks of rock falling away from the main mass when isolated by joints or faults, and to prevent slabbing of steep-dipping laminated rock.

Cable anchorages perform the same functions as rock bolts, but for much greater volumes of rock. Sometimes the cables are not stressed so that as soon as a rock mass attempts to move, they prevent any significant movement by becoming taut. The length of the cables is determined by the most probable slip surface, the anchorages being extended beyond this.
Rock in the vicinity of the right abutment of the Castillon Dam in France was consolidated by the construction of heavy buttresses, some of which were kept in place by prestressed cables carried down into sound rock. Prestressed cables were also used to stabilise the abutments of the Repulse Dam in Tasmania, where they were used to increase the effective weight of the rock in order to improve the resistance to sliding.

The problems which had to be overcome during the construction of the El Atazar Dam were enormous. They involved extensive use of pressure grouting, guniting, rock bolting, and prestressed cables, without which construction would have been impossible.

1.5 THE EL ATAZAR DAM

Droughts in 1947 and 1950 caused serious water shortages in Madrid. Consequently, plans were proposed in 1952 to harness the waters of the River Lozoya by constructing a series of five dams, one of which was the El Atazar Dam. The plans were approved in 1964 and construction began in October 1965. A general plan of the dam is shown in Figure 1.

1.5.1 The Site

The dam, whose centre-line runs predominantly in a W-E direction, is situated in an almost symmetrical valley with slopes of 40°. The topography at the site was suitable because the contours tended to be normal to the arches, except in the lower part of the left bank (as viewed from upstream), due to the river.

The rock consists mainly of Silurian slate which has been folded and refolded many times. Its planes of schistosity have a bearing which is almost perpendicular to the river, and a dip of 70°. The slate is crossed by two major families of fracture planes which subdivide the strata into parallelepipedons and wedges of various sizes. Family D1
are almost parallel to the right bank and dip to the north; Family D2 are almost parallel to the left bank and dip to the south. They are very nearly perpendicular to each other and they are considered dangerous because of their orientation and because they are filled with clay and crushed slate which gives them poor mechanical properties. Besides the two major fracture planes, there are also a number of fault planes which are important mechanically, because they are continuous, thick and filled with weak material. Another weakness on the site is on the left bank, just upstream of the dam, where there is a zone 20 - 30 m thick of almost vertically folded rock.

Although an exploratory programme had predicted certain difficulties from the geological point of view, a large part of the excavation for the dam was carried out. Serious problems, however, were encountered with regard to the stability of the slopes, especially on the left bank. A vast programme of geophysical surveys, boreholes, exploration galleries and geomechanical tests, both in the laboratory, and in situ, was therefore undertaken. This established accurately the geology of the site, Figure 2, and enabled the excavation for the dam to be completed and the slopes of both banks to be stabilised.

The angles of friction of the joint materials vary from $17^\circ$ for the most clayey, to $35^\circ$ where the fill is only fragmented slate without any traces of clay. The angle of friction for the schistosity planes is also $35^\circ$.

1.5.2 The Dam

Four types of dam were considered in the draft project,

- Straight line gravity
- Prestressed thin arch
- Thick arch
- Rockfill
and with the knowledge available, the thick arch seemed to be the most economical, although studies had shown that a prestressed arch was also possible. Models of two designs of the dam were undertaken at MIT\(^{(11)}\) but because of a lack of experience with prestressed thin arch dams, the choice of a thick arch dam was agreed on.

The final form of the El Atazar Dam was determined by means of radial adjustment calculations and model tests at LNEC, Lisbon. The dam is symmetrical, with its upper part resting laterally on two gravity type abutments, and its lower part on a socket. The socket provides the arch with a regular and continuous support, unifies the stresses due to any irregularities in the excavated surface, distributes and centralises the loads transmitted by the arch to the foundation, and reduces the stresses in the rock.

The dam is characterised by a pronounced downstream overhang in the crown cantilever, and by three-centred arches for both upstream and downstream faces, Figures 3 - 4. The thickness of the arches increases from the crown towards the abutments, and the faces have greater radii in the abutment zones. The geometry of the arches was chosen so as to lead to minimum bending moments and a more normal incidence on the valley slopes, thus permitting a better joining of the arch to the ground.

The general characteristics of the dam and the reservoir are:

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<td>Height above socket</td>
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<td>Length of crest between contours</td>
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<td>Thickness at the crest</td>
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<td>Thickness at the base</td>
</tr>
<tr>
<td>Radius of the reference surface (central zone)</td>
</tr>
</tbody>
</table>
Radius of the reference surface (abutment zone) 300.00 m
Central arch angle (central zone) 52.00 d
Total arch angle at the crest 92.93 d
Volume of concrete 1,100,000 m³
Volume of excavations 2,000,000 m³

Reservoir

Catchment area 924 km²
Surface area 1069 ha
Length of reservoir 17 km
Average yearly inflow 358 hm³
Reservoir capacity 426 hm³

To complete the design, two analyses of the dam were made using the Complete Adjustment Method. In the first analysis, the elastic modulus of the foundation was assumed to be constant and equal to 1/5 that of concrete. In the second analysis, the modulus was assumed to be 1/2 that of concrete below level 745, 1/8 that of concrete above level 842, and to vary linearly between these two levels.

1.5.3 Stability of the Slopes

In view of the orientation and weakness of the fracture planes, Family D1, on the right bank, a study was made of the stability of the slope using the mechanical properties of the rock and joint materials obtained from laboratory and in situ tests. The slope was divided into a number of solids, each delimited by a system of fault planes which appeared to make it unstable. The safety factor for each solid was calculated by considering its weight, the loads transmitted by the dam, the mechanical properties of the joint materials, and forces due to uplift. The calculations were based on assumptions which made the problem statically determinate.
The solids near the surface and in the middle zone of the right bank were found to be the most unstable due to their small stabilising self-weights: the safety factors were only slightly higher than unity. Although the safety factors did increase with the depth of the base plane, the orientation of the base plane was found to influence the stability more than the self-weight of the solid. It was found that planes below the true depth of Fault Eloisa, Figure 2, presented no doubts about stability. Solids on the left bank were found to be stable.

The results also showed that, with a well-drained foundation, the safety factor could be improved by as much as 70%. The study thus showed that a good drainage system was needed, so that uplift would not increase with time, and that the middle zones of the right bank needed to be reinforced.

1.5.4 Treatment and Reinforcement of the Foundations

Slides started high up on the left bank during excavations. To solve the problem, horizontal reinforced concrete beams were constructed at various levels and anchored by cables prestressed to 200 tonnes to a depth of 50 m. A system of holes and galleries was also constructed to drain the ground water. The fragmented surfaces exposed in the higher regions of the excavation were protected against superficial deterioration by rock bolting and guniting the rock surface.

To prevent possible slides during partial emptying of the reservoir, a grid of horizontal and vertical beams was constructed and anchored deeply into the ground, Figure 5. Between the beams and in the areas of highly fractured rock immediately upstream of the foundation, the ground was protected with a slightly reinforced concrete curtain.

The joint material in some of the larger Family D1 fractures immediately below the socket of the dam, was replaced with concrete.
On the right bank, flat surfaces below the concreting installations and upstream of the excavations for the foundation, were reinforced by constructing a system of concrete buttresses anchored to the ground with 140 tonne cables, and by intermediate rock bolting of the flat surfaces with 20 tonne bolts.

Downstream, the slopes were stabilised by beams anchored by cables prestressed to 230 tonnes to a depth of 50 m, Figure 5. They were constructed before the excavations for the stilling basin and mid-height spillway, both of which left the rock unsupported. The rock could not be removed because its weight helped to stabilise the slope against the thrusts of the dam.

Pressure grouting was used extensively in the treatment, consolidation and drainage of the foundation, Figure 6. During this work, geological studies were made which provided information to enable the stability of the slopes to be examined, Section 1.5.3. As a result, the socket in the middle zone of the right bank was replaced downstream by a reinforced ring beam, Figure 7, which was anchored with cables. A modification was also made to the right abutment where the thrust of the dam was taken deep into the ground by two horizontal piles, Figure 8.

The grout curtain on the right bank was inclined upstream so that the hydrostatic load on it acted in a more favourable direction. To reduce uplift, a deep gallery was constructed between levels 710 and 740 to drain the ground downstream of Fault 31, which runs across the valley and undercuts the central monoliths of the dam.

In those faults in the foundation of the dam which were more than 0.5 m thick, the fill was replaced with concrete. To ensure minimal decompression of the rock, the fill was removed by a double-emptying process. The treatment was carried out in five faults on the left bank and one on the right bank. Shear keys were thus created in Faults 11, 12, 13, Fault 2, Fault K and Fault Eloisa.
1.5.5 Scale Models

At the request of the group of construction companies, MEDEA, two models have been built at Imperial College to a scale of 1:200. Cement mortars based on pumice, cork and sand, were used to construct the models, because their inelastic behaviour under high stress is similar to that of concrete. This was especially important because the models were to be tested to failure in order to determine the factor of safety.

Model I was built without any of the valley reinforcements. Model II was built using smaller briquettes than in Model I, and including the modelling of the stilling basin and mid-height spillway, and the major reinforcements, the ring beam, shear keys and horizontal piles in the right abutment.

The models are considered in detail in Chapters 2 and 3.

1.5.6 Theoretical Study

As a separate, and entirely independent investigation to that requested by MEDEA, a finite element computer program has been developed and used to study the stability of the dam and its foundation.

The computer program is described in Chapter 4 and the study is reported in Chapter 5.
CHAPTER 2: MODEL I

2.1. INTRODUCTION

The purpose of the first model, which was built to a scale of 1:200 and according to the original design, was to study the behaviour of the El Atazar Dam under full scale loading, and to examine the overall stability of the dam and valley under various overload conditions. The overload conditions were designed to test for failure, first by crushing and then by sliding. Although a maximum load factor of 3 on crushing was desired, it could not be achieved because the method of applying gravity loading to the valley and foundation of the dam created stress concentrations in which the ultimate strength of the concrete would have been exceeded. In order to assess failure by sliding, the systems for applying gravity loading to the dam, foundation and valley, and the hydrostatic loading to the dam and grout curtain, were designed so that they could be applied separately. Uplift effects due to seepage underneath the dam were simulated by reducing the gravity loading on the foundation and valley.

It was, of course, important that the materials used in the model should have similar properties to those of the prototype and that the stresses developed in the model should be approximately equal to those in the prototype. Consequently, the appropriate ultimate strengths and elastic moduli were satisfied by adopting microconcrete for the monoliths of the dam and cementitious materials for the briquettes of the valley. The joints between the briquettes were filled with a mixture of plaster, talc and sand, to give the required coefficients of friction.

The briquettes were parallelepipeds, the surfaces of which were parallel to the two dominating families of faults, and to the schistosity. Also represented in the model by joints in the briquettes were the main fault planes, seven on the left bank, five on the right bank and one transversal to the course of the river.
2.2 **MODEL MATERIALS**

Site surveys indicated that the valley could be divided into three zones having different elastic properties of modulus of elasticity and ultimate strength. From previous experience gained in testing model dams at Imperial College, it was known that cementitious materials with mixtures of cork, pumice, and sand were capable of fulfilling the specifications shown in Figure 9. After extensive laboratory testing using 5x5x10 cm. specimens for the ultimate strength and 5x5x20 cm. specimens for the modulus of elasticity, the mixes given in Figure 10 were found to be suitable.

The mechanical properties of the three mixes are presented in Figures 11 - 13, and those for the dam in Figure 14. They show that a scatter of properties was obtained, which is typical for cementitious materials of this type, and that the properties varied little with age, with the exception of Mix D, which was the only mix with cork granules. In this case, although the modulus of elasticity remained constant, the ultimate strength increased with age.

Once the mixes for the valley had been established, attention was turned to the materials for the schistosity and fault planes. As these planes were reproduced in the model with the correct coefficient of friction and without any effective thickness, it was automatically assumed that the modulus of elasticity of the rock included the compressibility of the joint material, and that there was no need to provide a different modulus in the joint itself.

The correct coefficients of friction were obtained using 'push-out' tests on specimens made from the valley mixes, P, G and D, with frictional areas measuring 10x10 cm. The faces in contact with the joint material were coated with emulsion paint and, for the larger coefficients of friction, very fine sand. Various mixtures of talc, plaster, and Keene's cement were then tried until the mixtures given in Figure 10 were found to be satisfactory. Several specimens using these joint materials were
then made and tested at various times over a period of approximately one year. Figure 15 shows the scatter of the results obtained, and Figure 16 some of the horizontal load-displacement characteristics of the joint materials from which the peak and residual strengths can be calculated.

2.3 DESIGN AND CONSTRUCTION OF MODEL

The model was built to a scale of 1:200 and covered a zone of ground approximately 485 m. parallel to the axis of the dam and 610 m. transversal to it. It was constructed on the strong floor of the laboratory, a 91.5 cm. thick prestressed concrete slab with 60 tonne anchor points on a 61 cm. square grid. Concrete blocks prestressed to the strong floor on three sides of the valley site formed an open-sided box in which the valley was constructed. The open side of the box faced upstream.

As the gravity load was to be applied to the dam and foundation from below, the valley was raised almost 80 cm. from the floor and constructed on a 25 mm. thick mild steel base plate, supported on concrete blocks and steel columns. The assembled rig was lightly anchored to the floor.

To a minimum depth of 40 m. below the surface, the valley consisted of precast briquettes made from the mixes P, G and D. The briquettes were parallelepipeds whose surfaces were defined by the two dominating families of faults and the planes of schistosity. The briquettes, whose shape and size are defined in Figure 17, were cast in special collapsible timber moulds, where each was hand tamped and covered with damp sacking until the following day. They were then removed from the moulds, put under damp sacking for a week, and finally left to cure in the temperature controlled laboratory. As all the planes defining the briquettes' shape had coefficients of friction of 35°, the faces of the briquettes were painted with emulsion and sprinkled with fine sand.
The perspex geological model shown in Plate 1 was built in order to understand the possible modes of failure and as an aid to construction. Each sheet of perspex corresponds to a schistosity plane, and on them are marked the thirteen major fault planes and the zones of the three different rock properties, P, G and D. Plate 2 is a view of the model from upstream and shows Faults Eloisa and X which formed the lower limit of the briquette construction on the right bank. In the central zone, corresponding to the riverbed, the lower limit was defined by a horizontal plane at level 700 m., and on the left bank, by a series of planes representing the major family of faults, D2, which were positioned so that all points on the planes, downstream, below, and 30 m. upstream of the dam, would be at least 40 m. below the valley surface. The upstream limit of the model was defined by Fault 31 and the grout curtain.

Below these limits, a low strength, light aggregate, no-fines, unreinforced concrete was cast directly onto the steel plate, after a number of shear connectors had been welded or bolted to the steel plate. The elastic modulus of the concrete was approximately the same as that of the rock. Shuttering boards were positioned between the surrounding prestressed concrete block walls and the in-situ lightweight concrete during casting. The boards were subsequently removed and the resulting openings were filled with an expanding cement grout after shrinkage of the lightweight concrete had taken place. A banana-shaped hole was left immediately below the foundation of the dam during casting, because the calculations for the positions of the internal gravity loading points for the dam had not been completed. A similar hole had already been cut in the steel base plate. Plate 3 shows the shape of the valley which formed the lower limit of the briquette construction; the saw-tooth effect represents the series of planes defined by the family D2, and the dark coloured plane, Fault X. All surfaces were finished with a thin layer of concrete which, when dry, was painted with emulsion and sprinkled, where necessary, with sand.
To help with the laying of the briquettes, grids were drawn on the appropriate surfaces representing the intersection of family D1 and the schistosity planes with family D2, and family D2 and the schistosity planes with Fault X. A perspex sheet on which the valley contours were drawn and the major faults located, was placed over the model, and chains, cut to the appropriate length, were suspended from this sheet in order to obtain the correct valley shape. Further sheets of perspex on which only the fault planes were drawn, were placed at the upstream and downstream ends of the model. The combination of grids, perspex sheets and chains provided enough information for the briquettes to be laid.

While the valley was being constructed, the timber mould for the symmetrical, three-centred, doubly-curved arch dam was being built. The mould consisted of three parts: a base board, an upstream mould and a downstream mould. Each mould was made from blockboard panels spaced 5 cm. apart. The panels were cut to the appropriate radii of the downstream and upstream faces of the dam, separated by wooden blocks, and screwed and nailed together. Thin strips of plywood were tacked diagonally to the panels and blocks to form the faces, which were lightly sanded and varnished. As the dam was to be cast in seven monoliths, each monolith representing three monoliths of the prototype, grooves had to be cut in each face. The grooves were located by calculating the distances along the surface of the dam, from the centre-line of the dam to the relevant construction joints, at 5 m. intervals. The distances were then scaled and measured along the face of each mould. As the orientation of the grooves varied with height, the grooves were hand cut and several thin pieces of plywood inserted into them to form division plates. Finally, the division plates were varnished.

The assembled mould is shown in Plate 4 in the casting position with the thin crest of the dam at the bottom. The first dam to be cast was made of fine cork granules and araldite, and its seven monoliths were cut into fifty-six blocks of approximately equal weight and, therefore, volume.
The centre of gravity of each block was found by experiment and its position recorded and marked on a sheet of plywood. The positions were checked by a computer program which generated the horizontal cross-section of the prototype dam and socket at metre intervals, divided each monolith in half, and proceeded to calculate the areas and centres of gravity for each section. It finally printed out for each level considered, the coordinates of the points at which the construction joints intersected the upstream and downstream faces, the arclength of each point from the centre line of the dam, the area and centroid of the cross-section of each monolith, a running total of the volume and moments of area about all three cartesian axes for each monolith, and the total volume of the dam. Using this information, the theoretical positions of the centres of gravity were calculated. The two methods compared favourably.

Plate 5 shows one of the two methods used to apply gravity loading to the dam. After reassembling the mould and greasing its surfaces to prevent water in the microconcrete being absorbed by the wood, 5 cm. diameter steel discs were suspended on round headed 7 mm. diameter prestressing wire rods. Polythene tubing was pushed onto the rods which were then passed through the template made by drilling holes at the centroidal positions marked on the sheet of plywood. Some of the positions, however, had to be paired off and slightly rotated in order to prevent the discs inhibiting each other. The rods were suspended from the template so that the top of each disc was at a level corresponding to the highest level of its associated block. Finally, the template and rods were placed over the timber mould, lowered to the correct position and fixed. The monoliths were cast in the inverted position with the rods protruding from the base of the dam. Plate 5 shows the monoliths after their removal from the timber mould.

The second method of applying gravity loading to the dam was by hydraulic jacks. The previous method could not be used to load the blocks at the crest, because it would have meant the wire rods leaving,
and possibly re-entering, the downstream face of the dam at angles of
incidence leading to a large area of surface damage. Consequently,
the blocks at the crest were loaded externally from above by hydraulic
jacks.

After the dam had been cast, thirteen more holes were
drilled in the template with their centres approximately 15 cm. apart.
They represented the foundation gravity loading positions. The tem-
plate was then located above the banana-shaped hole in the lightweight
concrete base of the valley, and a duplicate template was placed in
a matching position below the steel base plate. Polythene tubes with
stiffening rods were strung between the two templates and the hole was
filled with lightweight concrete. When the concrete had set, the tubes
and rods were removed.

Briquettes were used throughout the remaining valley structure
above the lightweight concrete base. In addition to the Faults Eloisa
and X, where the fault faces were partly formed by the concrete base
and the schistosity planes, and fault families D1 and D2, formed by
the briquettes themselves, other major faults were included in the con-
struction. These were Faults 2, 5, 11, 12, 13, K and X1 on the left
bank, Faults 16, 24, 31 and 32 on the right bank, and Fault 16 trans-
versal to the course of the river. Each briquette was fitted and
bedded in place with the correct plaster mix or mixes. When a major
fault passed through a briquette, the briquette was correctly aligned and
and the position of the fault located by sighting through the perspex sheets
at the upstream and downstream ends of the model. After the briquette
had been marked, it was cut with either a tungsten carbide hand saw or
a mechanically driven circular saw. The pieces of the briquette were
then bonded together with the relevant plaster mix and allowed to cure
before the complete briquette was bedded into the valley. Approximately
300 briquettes were used to complete the valley construction.

With the valley complete, attention was focussed on completing
the foundation and inserting the dam into the valley. By drilling upwards
from below the base plate, the holes for the gravity loading rods were extended through the briquettes to the surface, where the foundation holes were enlarged in order to accommodate the 5 cm. circular discs of the foundation gravity loading system, Plate 6. The loading system for the foundation, which will be described in the next section, was then assembled and full gravity loading was applied to it. Settlement of the valley blocks below the dam was therefore ensured and, to some extent, the conditions on site reproduced. With the aid of the down-stream mould, the monoliths and abutments, which had been cast separately in their own timber moulds, were placed and bedded in the valley, Plates 7 - 8. Strain gauge rosettes and linear gauges were placed in complementary positions on the upstream and downstream faces of the dam. Finally, tapes were placed over the construction joints between the monoliths of the dam, and the joints grouted by gravity feed from the crest of the dam, with half of the gravity loading applied to the dam and full gravity loading to the foundation and valley. This was done in an attempt to reproduce the stresses set up in the prototype by gravity.

2.4 DESIGN AND CONSTRUCTION OF TEST GEAR

As failure by sliding was to be investigated, it was essential that gravity loading on the foundation, dam and valley, and hydrostatic loading on the dam and grout curtain, could all be applied separately. The gravity loading on the valley had also to be designed to produce the correct total loads on Faults Eloisa and X. These conditions made it necessary to control the hydraulic jacks used in the various systems by several loading cabinets with their own pressure source.

To simulate the gravity load of the dam it was necessary to distribute the number of loading points throughout the dam in such a manner that undue stress concentrations would not exist around the loading points and, at a short distance away from the loading points, the stresses set up in the dam would be independent of the manner in
which the loading was applied. As explained previously, this was
achieved by casting steel rods within the monoliths, which were
anchored to the dam so that they passed through the centroids of
equal volume, and which extended to a linkage below the steel base
plate. The rods passed through plastic tubes in the dam and holes
in the valley, which provided sufficient clearance around the rods
for the model to adopt the prescribed definition of rupture: a maximum
admissible displacement of the prototype valley of 0.5 m. Below the
base plate the rods were paired off and connected by wedge type grips
to spreader beams, which were coupled by long steel bolts to a second
layer of spreader beams, each of which was connected by a bolt to a
20 tonne hollow ram. Thus the same load was applied to each tension
rod. The rams reacted against a mild steel plate which transferred
the load to a second steel plate bolted to the strong floor of the laboratory,
Plate 8.

Gravity loading at the crest of the dam and on the abutments
was applied by hydraulic jacks acting on steel plates and reacting against
an overhead reaction frame. The reaction frame consisted of four
I-beams supported at each end by an I-beam placed on top of the concrete
blocks at the sides of the model. Resting on top of the four I-beams
were two more beams, one at each end, running in an upstream-down-
stream direction. From the top of these beams, 60 tonne bolts ran
through them, down between the four I-beams, past the supporting beams
and through the concrete blocks to the underside of the floor. There
were ten bolts in all, five on each side. The bolts were post-tensioned
to provide sufficient prestress in the reaction frame to counter the
upward thrust of the hydraulic jacks reacting on the thick steel plates
clamped to the underside of the four I-beams. The system is shown
in Plate 12.

Reacting against the frame were hydraulic jacks of various
capacities. The majority of these provided the gravity loading for
the right bank of the valley. Special care was taken for this bank, to
ensure that the loading system adopted did not produce stress concentrations at the loading points which would cause local failures of the material. Extension tubes fitted to the jacks permitted a constant valley loading to be maintained during lateral displacement of the valley along a fault plane. The extension tubes acted on a triangular steel plate which distributed half of the load directly to two pedestals, and the other half, via a spreader beam, to two more pedestals. Where possible, each briquette on the right bank was fitted with a pedestal which was fixed to it by four steel shear connectors and bonded to it with Certite, a polyester resin, Plate 9.

On the left bank of the valley, gravity loading was confined to the abutment area. Four hydraulic jacks were used to provide sufficient gravity load to stabilise the abutment. The central area immediately downstream of the dam was also subjected to gravity loading. In these two areas, hydraulic jacks acting upon individual pressure plates were used.

In the previous section it was stated that thirteen wire rods with discs were used to apply gravity loading to the foundation. The wire rods in the central and right bank zones were connected by wedge type grips to lever arms to which hydraulic jacks were attached. The jacks reacted against the steel base plate of the model. On the left bank, however, the load was applied by a screw threaded into the lever, which reacted against the steel plate. The gravity loading on the central and right bank foundation areas was, therefore, easily adjusted hydraulically, while on the left bank it remained at a level equivalent to normal gravity load.

Thirty-five hydraulic jacks of various capacities linked to a common pressure source were used to apply hydrostatic load to the dam. The load was distributed to the dam by 25 mm. thick steel pads with superimposed curved surfaces. The curved surfaces were cut from a sheet of glass fibre made to the shape of the upstream face of the timber mould. A resin mortar bonded the surface to the steel pad. Two
orthogonal layers of flat rubber strips, the first on the curved surfaces and the second on the upstream face of the dam, distributed the load to the dam. The hydraulic jacks, fitted with extension tubes, acted at the centre of pressure of each plate and reacted, via adjustable swivel blocks mounted on a steel plate, against concrete blocks prestressed to the strong floor of the laboratory, Plate 10. The top row of pads was welded to the steel pads through which gravity loading was applied to the crest of the dam.

A computer program was written to determine the number of pads, the size and centre of pressure of each pad, and the position and orientation of each swivel block. This was achieved by dividing the upstream face of the prototype dam into horizontal strips 50 cm. wide, and subdividing the strips into 50x50 cm squares. Each square was then divided into two triangles, and the resultant hydrostatic load acting on each triangle was calculated. The component forces of the resultant for each triangle were then used to calculate the total load acting on each horizontal strip and the total load acting on the dam. With this information and the sizes of hydraulic jack available, the program divided the surface into four horizontal bands, each band corresponding to a set of pads, Plate 10. The coordinates of the centre of pressure, the magnitude and direction cosines of the resultant force, and the size of each plate in each set, were calculated. Finally, the direction cosines were used to locate the positions and orientations of the swivel blocks on the reaction plate.

About 38% of the total hydrostatic force was applied directly to the valley. The majority of this loading was applied by ten hydraulic jacks to the grout curtain on the right bank which formed, along with Fault 31, the upstream limit of the model. The remainder of this hydrostatic load was applied by three hydraulic jacks to the region where Fault 31 undercut the central monolith of the dam, Plate 8. The load was applied in the same manner as for the dam, and the hydraulic jacks were connected to the same pressure source. A valve was,
however, inserted in the oil pipeline serving the jacks to enable the
effect of hydrostatic load on the grout curtain and Fault 31 to be
examined.

When the jacks and extension tubes for the grout curtain
were being placed in position, it was found that some of them could
not be assembled in the correct positions because they were too big.
Consequently, some of the swivel blocks were moved to positions
immediately below their true positions, and others were moved down-
wards and to the side. In all cases, however, they were moved so
that they would have more unfavourable effect.

Uplift effects were simulated by reducing the gravity loading
on the foundation and valley.

2.5 INSTRUMENTATION

Strains in the dam and abutments were measured by electric
resistance strain gauges of 10 mm. gauge length. The gauges were
read by a Solartron digital data logger which read 10 gauges per second
and recorded them on paper and punched tape. The punched tape was
used for computer analysis. Displacements of the dam and valley were
measured by differential transformer transducers and dial gauges. The
points at which strains and displacements were measured are shown in
Figures 18 - 19.

The dial gauges and transducers were clamped to a special
Y-frame so that all the displacements would be measured in relation to
the frame datum. Upstream of the model, the Y-frame was mounted
on the strong floor by a ball seating on the left bank and a roller bearing,
which only permitted movement normal to the axis of the valley, on the
right bank. Downstream of the model, it was mounted on the reaction
blocks with a bearing which offered only vertical support.
A Seismotron, a very sensitive hearing aid, was connected to an oscilloscope and used in an attempt to detect the onset of cracking. It proved, however, to be more of a novelty than a success.

2.6 EXPERIMENTAL PROCEDURE

Preliminary loading of the model was designed to ensure that the valley, the dam monoliths and the foundation, all represented as accurately as possible the conditions on site at the end of construction. Gravity loading was therefore applied to the foundation of the dam before the monoliths of the dam were bedded into the valley; the construction joints between the monoliths were grouted with half of the gravity loading applied to the monoliths and full gravity loading applied to the foundation and valley.

The objective of the preliminary testing was to obtain the behaviour of the model dam under full scale loading. It was likely however, that by applying full gravity load to the dam without hydrostatic load, some serious overstressing could occur in the dam due to the stress concentrations bound up with the method of applying gravity loading. It was also undesirable to apply full hydrostatic load without first having full gravity load on the dam because, besides not reproducing a realistic condition of the prototype dam, it was necessary to ensure that only tolerable stress levels were developed. In the preliminary tests, the loading sequence was therefore to apply combinations of gravity and hydrostatic loading to the dam, in such proportions and in such increments, that the total deformations and stresses were well within acceptable limits.

There were approximately forty different load cases in the preliminary testing of the model and, so far as could be ascertained, the dam remained uncracked throughout; nor was any significant non-linear behaviour detected. Inspection of the dam was almost impossible because the hydrostatic pads obliterated the upstream face, and hydraulic jacks and the deflection frame prevented access from downstream, Plate 11.
During each load case, Faults Eloisa and X were prevented from moving by a steel plate butted against the concrete reaction blocks at the downstream end of the model. From the readings taken, it was observed that the model behaved in a manner which enabled the stresses and deformations at full gravity load and full hydrostatic load to be obtained by extrapolation from partial loading.

During all tests, the integrity of each loading system on unloading was ensured by maintaining a small residual pressure, equivalent to five per cent of the full load pressure, in the system. The gravity load on the foundation was kept permanently at its full scale value except, of course, for the overload tests. At night, this was achieved by using a load maintainer.

2.6.1 Loading Series A

Combinations of loading were applied to enable the stresses and displacements due to full hydrostatic load to be obtained. They were measured with and without hydrostatic loading on the grout curtain and, in both cases, Faults Eloisa and X were prevented from moving downstream.

2.6.2 Loading Series B

Having ascertained the stresses and displacements of the dam under hydrostatic loading, tests were planned to study the stability of the dam on the rock mass, particularly across Faults Eloisa and X. The geological model in Plate 2 shows Faults Eloisa and X intersecting to form a slender pyramid of rock below Fault X, and a wedge of rock above Fault X and between the two faults which increases in size as Fault Eloisa rises to the surface. In these tests it was therefore impossible to have the correct stress on Faults Eloisa and X simultaneously because the gravity loading for the wedge between the faults could not be applied separately.
The stability of Fault Eloisa under hydrostatic loading was investigated by preventing movement on Fault X and applying hydrostatic loading to the dam and grout curtain, full gravity loading to the dam and foundation, and the correct gravity loading on Fault Eloisa. Seven loading cabinets were needed to apply the load, Plate 12.

The stability of Fault X under hydrostatic loading was investigated by allowing Fault X to move and applying hydrostatic loading to the dam and grout curtain, gravity loading to the dam and foundation, and sufficient gravity load to produce the correct stress situation on Fault X. Consequently, Fault Eloisa sustained an excessive gravity loading which prevented it from sliding. Only six loading cabinets were required to apply the load.

The purpose of each loading cabinet is given in Figure 20.

2.6.3 **Loading Series C**

The third series of tests was designed to study the stability of the dam and rock mass under uplift conditions. Uplift was represented, albeit inadequately, by reducing the pressure in the hydraulic jacks applying the load to the foundation and the valley surface. With this representation, the stability of Faults Eloisa and X was studied in a sequence of tests similar to those described in Loading Series B.

2.6.4 **Loading Series D**

Having established that there were no indications of distress or instability in the dam, foundation or valley under full scale prototype loading, with and without uplift, it was important to establish at what load factor failure would occur by crushing. In these tests movement on Faults Eloisa and X was permitted, and the stability on these faults, for the correct stress condition on Fault X, in multiples of the full scale loading, was obtained; Fault Eloisa was thus overloaded during the tests and, to some extent, prevented from moving.

Combinations of full scale loading and various degrees of uplift on the valley, ranging from zero uplift to full uplift, were tested.
and then increased monotonically beyond the full scale value until a load factor of 2.4 was reached. At this load factor, testing was stopped because the local stresses around the loading points in the valley and foundation became very close to the ultimate strength of the valley material. It was considered more important to explore the overall stability against sliding rather than achieve a multitude of local failures.

This series of tests, designed for testing failure by crushing, was therefore limited to a load factor of 2.4.

2.6.5 **Loading Series E**

This series of tests was designed to achieve failure in the model by sliding. Once again, movement on Faults Eloisa and X was permitted and the correct stress condition was produced on Fault X.

In the first instance hydrostatic and gravity loading, with full uplift on the valley, were applied until twice the prototype condition was reached. With the gravity loading kept at twice full scale, the hydrostatic loading on the dam and grout curtain was increased to a maximum of 2.9 times the full scale value, at which point the reaction blocks for the hydrostatic load began to overturn. As the model was stable at this loading, the gravity and hydrostatic loading were reduced to the full scale value and, maintaining the full uplift value, the hydrostatic loading was increased to 2.9 times the full scale value. On this occasion also there was no failure. Failure was eventually induced by reducing the gravity loading on the valley to half and then a quarter of the full uplift value. First failure occurred with the valley sliding on Fault X.

The loading sequence used to induce failure was repeated. No significant movement took place until the gravity loading on the valley had been reduced to half the full uplift value, when final failure occurred with a major slide on Fault X and some movement on Fault
Eloisa. The test was stopped when the movement on Fault X corresponded to about 5 m. on the full scale.

A summary of all the relevant load cases is given in Figures 21 - 22.

2.7 RESULTS AND DISCUSSION

Stresses and displacements were obtained from the strains and displacements measured at the locations shown in Figures 18 - 19. The results are presented in terms of prototype behaviour, the displacements having been derived by taking into account the scale of the model.

2.7.1 Results of Loading Series A

The displacements and stresses in the dam under full hydrostatic load on the dam, and full hydrostatic load on the dam and grout curtain, are presented in Figures 23 - 29. The results were extrapolated from the load change from 0.3 times hydrostatic load to full hydrostatic load, since it had been established that the model was behaving linearly over this range.

The reasonably consistent behaviour of the model under hydrostatic load during the first three series of tests is indicated by Figures 23 - 24, which show the radial displacements of the crown cantilever and at the levels 817 and 867, for four different load cases. The small residual shapes shown in the figures also justify the consistency. The effect of hydrostatic load on the grout curtain was to increase the downstream movement on the right bank, with the result that the symmetrical displacement pattern produced by hydrostatic load acting on the dam only, became slightly distorted with a maximum displacement of approximately 110 mm. off-centre towards the right bank. The grout curtain load had little effect on the crest displacement, but it did increase the displacement at the base of the crown cantilever from 35 mm. to almost 50 mm., and the movement of the right abutment from 22 mm. to 33 mm.
The hoop stresses at levels 817 and 867 are shown in Figure 25, and of particular interest are the high hoop stresses near the right abutment. A significant bending moment is apparent at the crest on the right bank side of the centre-line. It does not, however, occur at level 817 and so it may be attributed to some instability in the loading arrangements which occurred in the area.

The steel pads through which gravity load was applied to the crest of the dam were welded to the steel pads through which hydrostatic load was applied. The lines of action of the hydraulic jacks applying the gravity load generally passed through points outside the crest of the dam, with the result that the plate tended to slip off the crest. Consequently, hydrostatic load was applied to prevent the plates from slipping. This, however, did not completely cure the problem because the plates did not return to their original positions when they were unloaded. Extendible props were therefore installed to prevent the plates from slipping, and before any strains or displacements were recorded, a check was made to see that the crest was not prevented from moving, by making sure that the props were loose.

Hydrostatic load on the grout curtain produced a slight increase in stress on the right bank and in the hoop stresses on the downstream face of the dam, Figures 25 - 26. The maximum hoop stress is 34 kg/cm² and occurs at level 842. A typical stress pattern was obtained for the vertical stresses on the crown cantilever due to hydrostatic loading, Figure 27, though the degree of vertical bending was much less than expected. This may be due to the undercutting of the crown cantilever by Fault 31, which is almost certain to have caused the curve for the upstream face to tail off near the base and give a maximum tensile stress of only 7 kg/cm² at the base; the compressive stress at the base was approximately 30 kg/cm².
The principal stresses on the upstream and downstream faces due to hydrostatic loading are shown in Figures 28 - 29. Typical arching action may be seen on the downstream face; on the upstream face it is not so apparent, and it is possible that the general stress distribution and magnitudes may have been affected by the method of applying load and by some of the construction joints not being grouted perfectly. Hydrostatic load on the grout curtain does not alter the principal stress directions but it does tend to decrease the maximum principal stresses and increase the minimum principal stresses in the lower regions of the dam and on the right bank; the tendency was therefore towards more horizontal arching and less vertical and horizontal bending.

2.7.2 Results of Loading Series B

The results for this series of tests, designed to study the effect of hydrostatic load on the dam and grout curtain on the stability of the dam on Faults Eloisa and X, are presented in Figures 30 - 36. The stresses are very similar to those for Loading Series A for hydrostatic loading on the dam and grout curtain, as are the displacements for the cases in which the stability on Fault X was tested with Fault Eloisa overloaded. However, there is an increase in radial displacements on the right bank of about 7% for the cases where sliding on Fault Eloisa was being tested. This is not surprising because Fault Eloisa is very close to the surface in this region, Plate 2.

2.7.3 Results of Loading Series C

The first study in this series was of the behaviour of the dam under full load with various conditions of uplift on Fault Eloisa with Fault X prevented from moving, Figures 37 - 42. The radial displacements of the crown cantilever and at levels 817 and 867, for uplift conditions ranging from zero uplift to full uplift, are given in Figures 37 - 38. It can be seen that full uplift on the foundation and valley causes an increase in the displacement at the crest of the crown
section from 72 mm. to 88 mm., and at the base from 35 mm. to 46 mm. The pattern of curves in Figure 38 shows that the movement on the right bank was less than that on the left bank at levels 817 and 867, and that the dam appeared to rotate about the ends of the arches.

The displacements due to hydrostatic load only are shown for purposes of comparison. The extra movement on the left bank may be attributed to insufficient gravity loading being applied to the left bank abutment area; consequently the briquettes were not properly bedded down. The effect of the gravity load of the dam was therefore to compact the briquettes on the downstream side of the dam, with the result that due to the orientation of the family of faults D2, there was further radial movement. The overall effect of gravity loading was therefore to cause downstream movement of the dam on the left bank; on the right bank, however, its effect was to cause upstream movement of the dam, movement which may possibly have been considerably influenced by the inclination of Fault Eloisa.

Figures 39 - 42 show that the effect on stresses of uplift on Fault Eloisa is very small. Provided the significant bending moment at the crest, attributed to an instability in the loading arrangements, is ignored, the hoop stresses indicate that there is no appreciable horizontal bending action and that a greater load is distributed to the right abutment than to the left abutment. The principal stresses show reasonable arching action on both faces of the dam. The behaviour, however, is not symmetrical.

The second study in this series was of the behaviour of the dam under full load with various conditions of uplift on Fault X with Fault Eloisa overloaded. Figures 43 - 48 show a similar behaviour to the first study but with a larger increase in the radial displacements of the dam.

The results of both studies indicate that no serious stress levels were reached.
2.7.4 Results of Loading Series D

In the light of the results for the previous loading series, the testing of Fault Eloisa with Fault X prevented from moving, was abandoned. The tests in this series were designed to explore the extent to which the model could sustain overload conditions without failure by crushing. All loads were therefore increased monotonically beyond their full scale values.

The displacements of the dam and valley for various load factors are shown in Figures 49 - 54, and although the results for load factors up to and including 2.0 are presented, a maximum load factor of 2.4 was achieved. At this load, however, a local failure occurred below one of the hydraulic jacks applying gravity loading to the valley in order to stabilise the left abutment. Unfortunately it took place before any strains or displacements could be recorded, and the test was not repeated for fear of damaging the valley. As the foundation loading system was also causing difficulties at high load factors, it was decided not to increase the load above its full scale value in any further test. This was not a serious restriction because the foundation load represented approximately 7% of the total gravity load on the right bank.

The results show that the dam did not behave symmetrically and that while the abutments remained relatively still, the central monoliths of the dam moved significantly downstream. The displacement histories shown in Figure 54 suggest that the model behaved linearly and that failure was not imminent.

2.7.5 Results of Loading Series E

The final series of tests was designed to produce failure in the model by sliding, the idea being to alter the resultant load path in the model by increasing the hydrostatic load while the gravity load remained constant. In order to eliminate the effect of creep, which was bound to occur at high loads, only fifteen dial gauges were recorded. The curves in Figures 55 - 86 showing the radial displacements of the dam are necessarily smoother than those for all the previous loading series.
Figures 55 - 62 show the results for Load Case 65 in which a monotonic increase in gravity and hydrostatic load to twice full load was followed by an increase in the load factor from 2.0 to 2.4. As in previous tests, there was a considerable movement of the central monoliths of the dam, and more movement on the left bank than on the right bank. Figures 55 - 56, however, indicate that the increase in hydrostatic load produced slightly more movement on the right bank than on the left bank. A reasonably linear behaviour of the dam is indicated by the displacement histories shown in Figure 56. The sudden change in shape does not indicate a sudden change to inelastic behaviour, but simply shows that the displacements of the dam rapidly increase when the hydrostatic load is increased. This is confirmed by the residual displacements showing no appreciable permanent set.

The hoop stresses at levels 817 and 867, Figure 57, show significant bending moments on the right bank and a mean stress at the right abutment approximately five times greater than that at the left abutment. The maximum compressive stress of $170 \text{ kg/cm}^2$ on the downstream face is less than the ultimate crushing of the microconcrete of $297 \text{ kg/cm}^2$. Probably due to Fault 31 undercutting the dam, there is no tensile stress in the vertical direction on the upstream face of the crown cantilever. The strain histories of Figure 58 confirm the deductions made from the displacement histories and show that the dam did not suffer any permanent damage. Figures 60 - 62 show the principal stresses for full load, twice full load, and for a load condition of 2.4 times hydrostatic loading and 2.0 times gravity loading; they show typical arching action, high compressive stresses at the right abutment and near the base on the left bank, and no notable changes in principal stress directions.

Failure was reached in Load Case 66. Figures 63 - 77 present the results for the various loading combinations listed in Figure 65 which were undertaken before failure was induced by
reducing the gravity load on the foundation and valley below their 100% uplift values, while keeping the hydrostatic load factor at 2.9. The large residual displacements in Figures 63 - 65 signify failure and indicate that the right abutment moved at least 300 mm. downstream. The left abutment, however, was relatively unaffected.

The displacement curves in Figure 63 show that when gravity load was held constant and the hydrostatic load increased, the base of the crown cantilever and the left abutment remained comparatively still, while the crest and crown cantilever took the brunt of the load, and bulged. The right bank side of the dam therefore appears to have hinged on the left bank side of the dam, the amount of rotation being influenced by the hydrostatic load on the grout curtain: the bigger the ratio of hydrostatic load factor to the gravity load factor, the greater the influence. The displacement histories in Figures 64 - 65 illustrate the rapid increase in downstream movement produced by an increase in hydrostatic load, and show that when the dam was unloaded from loading condition I to loading condition K, its stiffness did not change; also, when failure was induced by reducing the gravity load on the valley, the dam remained still. Failure of the model, consisting of a significant movement on Fault X, must therefore have been quite sudden.

The stresses in Figures 66 - 69 reveal that the gravity load on the valley had little effect on the stresses in the dam provided that there was sufficient to keep the dam stable. This can be seen in the figures by comparing the top series of curves for twice gravity loading on the valley, with those in the bottom series for full gravity loading. Once again, there are significant hoop stresses at the right abutment. The strain histories in Figure 70 are in sympathy with the displacement histories, but whereas the residual displacements were very large, the residual strains are not. This suggests that the dam did not suffer serious damage in spite of the considerable movement of the right bank. There is a general orderliness about the principal stresses presented
in Figures 71 - 77. They indicate that the behaviour of the dam under hydrostatic load was almost symmetrical and that it supported the hydrostatic load by an arching action. Figures 74 - 76 show a general clockwise rotation of the principal stress directions on the upstream face as the hydrostatic load was increased and the gravity load on the valley reduced. Thus an increase in hoop compression took place on the left bank which suggests that there was a reduction in shear resistance where the dam joins the lower region of the left bank. The 'residual stresses' in Figure 77 must not be regarded as actual stresses but rather as an indication that the dam suffered permanent deformation as well as cracking. They were in this case generally small, except where there were high compressive stresses near the base of the dam on the left bank, already a suspect zone.

The second failure and final rupture of the model was produced in Load Case 67 and achieved by repeating Load Case 66. A comparison of Figures 63 and 78 shows that the crest displacements for full and twice full load were much greater in the second failure test than in the first, while the displacements immediately before failure were approximately the same. This phenomenon may possibly be attributed to the joint material for Fault X, having exceeded its peak strength in the first failure test, only having its residual strength to resist the load: this seems very likely because a sudden movement on Fault X, which did in fact occur in the first failure test, is suggested by the load-displacement characteristics of the joint materials in Figure 16.

The suggestion that the dam did not suffer any serious damage in the first failure test is confirmed by a comparison of the stresses in Figures 66 - 69 and Figures 79 - 80, and the strain histories in Figures 70 and 81. However, the residual strains do show that the dam was severely damaged after the second failure. The principal stresses, Figures 82 - 85, show that the general behaviour of the dam before and after the first failure was similar,
but with the crest taking slightly more load and the right bank slightly less load than before failure. The principal stresses in Figure 85 depict the behaviour of the dam immediately before it failed due to a major movement on Fault X. It shows high compressive stresses and a significant bending moment at the right abutment, and the development of tensile stresses and high compressive stresses low down on the left bank. The 'residual stresses' in Figure 86 confirm that the dam failed.

The model after failure is shown in Plates 13 - 16. The movement of the valley on Fault X is indicated by the distance between the two short parallel lines at the bottom of Plate 13. The continuous lines between the rows of pedestals show that the whole of the right bank moved bodily downstream. The significant movement on Fault X of 20 - 30 mm. in the model is shown with the main crack formation on the dam on Plate 14. A comparison of the crack formation with the 'residual stresses' in Figure 86 shows that the principal directions, for strain gauge rosettes near cracks, are approximately normal to and parallel to the nearby crack. Plate 15 shows a close-up view of the movement on the right bank and Plate 16 shows the shear failures which occurred in the construction joints on the right bank due to the excessive movement on Fault X.

2.8 SUMMARY

An investigation into the behaviour of the El Atazar Dam has been made by the construction and testing of a model to a scale of 1:200. The scale of the model was chosen having regard to the testing facilities available and to the size of valley which was considered necessary to dissipate the dam reactions. The extent of the valley upstream was limited by assuming Fault 31 and the grout curtain on the right bank to be no-tension surfaces. To a depth of approximately 40 m. below the surface, the model valley consisted of briquettes which were shaped to conform with the two dominating families of faults and
the planes of schistosity, and cut so that the major faults could be modelled. There are, of course, geological weaknesses in the rock mass which were not reproduced in the model. Below a depth of 40 m. the valley was considered to be sufficiently stable to allow it to be represented by a mass of lightweight concrete.

The model was designed to study particularly the conditions of failure on Faults Eloisa and X. In order to do this several loading cabinets were used to produce various combinations of gravity load on the dam, valley and foundation, and hydrostatic load on the dam and grout curtain. The effect of uplift on the valley was simulated by a reduction in gravity loading, but no reduction in the frictional properties was allowed for in the joints between briquettes, as might occur on site due to seepage.

The model was tested for failure by crushing, and failure by sliding. The model was found to be stable under a crushing load factor of 2.4 times full load. Failure had to be induced by reducing the gravity loading on the valley to unrealistic levels because the hydrostatic loading system was designed to resist a load factor of 3.0. It occurred at a load factor of 2.9 on hydrostatic loading on the dam and grout curtain, full scale gravity loading on the dam, and less than full scale gravity loading on the valley and foundation with full uplift. This represents a more onerous loading condition than is likely to happen in the prototype and indicates that there must be an appreciable alteration in the resultant load path in the valley before failure occurs.

No definitive assessment of the stresses and displacements has been made except for stating whether they are tensile, compressive or near the ultimate stress. This was a deliberate policy because further discussion of the model behaviour will be deferred until Chapter 6 in the light of the theoretical results presented in Chapter 5.
CHAPTER 3: MODEL II

3.1 INTRODUCTION

The second model of the El Atazar Dam included some of the modifications which were made to the original design after a large part of the excavation for the dam had been carried out, during which serious difficulties had arisen, owing to the lack of stability of the slopes. The purpose of the second model was therefore to determine the effect of the strengthening measures on the safety of the dam.

In order to make a direct comparison with the first model so that the strengthening could be assessed, the second model should have been virtually the same as the first. However, as a more accurate representation of the valley was considered to be essential, the valley was made weaker by using briquettes with half the linear dimensions of those in the first model which meant an eightfold increase in the number of briquettes used to construct the valley.

Model II was built to a scale of 1:200 and included the following strengthening measures:

a) shear keys
b) a reinforced concrete ring beam anchored by cables deep into the valley
c) strengthening of the right abutment with horizontal piles
d) a mid-height spillway
e) a stilling basin

The small number of hydraulic jacks used to apply gravity load to the left bank in the tests on Model I, produced high stress concentrations under the loading pads, some failure of the model material and, according to the results, insufficient gravity load to bed down the briquettes. Consequently, a more accurate representation of gravity loading on this bank was achieved in Model II by loading it in the same way as the right bank.
In the first model, the outward movement of the right abutment which occurred with failure, was considered to be restricted by the close proximity of the reaction blocks at the side of the model. The valley was extended and additional briquettes were therefore used in Model II to produce a more accurate representation of the valley strength at this abutment.

The testing arrangements in Model II were re-designed to permit a load factor of 2.5 on the gravity load on the dam and valley, and a load factor of 5.0 on the hydrostatic load on the dam and grout curtain. In view of the local failures which occurred below the foundation gravity loading discs in Model I, the foundation load in the second model was kept at its full scale value.

3.2 MODEL MATERIALS

The three mixes P, G and D used for the briquettes of the first model were slightly altered for the second model following a series of tests which showed that both the modulus of elasticity and ultimate strength would have been too low for the smaller briquettes. This was probably due to a change in the method of casting; the large briquettes were cast in timber moulds and hand tamped, while the smaller briquettes, owing to the number required, were cast in steel moulds and placed on a vibrating table. At first, the vibration was not very successful because the cork and pumice rose to the surface. However, this was overcome by giving less vibration.

The materials and mixes used in Model I for the joints between the briquettes and the monoliths of the dam were also used in Model II. The shear keys, ring beam, piles, spillway and stilling basin were constructed in microconcrete using the same mix as the dam.

The average mechanical properties of the joint materials and the dam and valley mixes are given in Figure 87.
3.3 DESIGN AND CONSTRUCTION OF MODEL

The model was built to a scale of 1:200 and covered a slightly larger area of ground than the first model, due to extending the briquette construction onto the reaction blocks on the right bank side of the model. This allowed a more realistic amount of outward movement to occur at the right abutment before failure, and therefore provided a better representation of the valley strength at this abutment.

The valley, as before, was raised above the floor of the laboratory, constructed on a thick mild steel base plate supported on concrete blocks and, to a minimum depth of 40 m. below the surface, consisted of precast briquettes having half the linear dimensions of those used in the first model. Below the briquettes, the valley consisted of light aggregate concrete which was cast directly onto the steel plate after a number of shear connectors had been welded, or bolted, to the steel plate. Plate 17 shows the valley foundation which formed the lower limit of the briquette construction. The series of planes defined by the family D2, which created the saw-tooth effect in Plate 3, were not cast for the second model at the outset because of the difficulties they caused during the laying of the briquettes. Instead, they were cast as and when the briquette construction on the left bank demanded it.

Sheets of perspex, on which the major fault planes were marked, were placed at the upstream and downstream ends of the model. Further sheets of perspex, on which the valley contours were drawn, were supported over the model by scaffolding. A timber board was placed, in accordance with the strike and dip of the schistosity plane used to define the shape of the briquettes, at the downstream end of the wedge of rock created by the intersections of Fault Eloisa, Fault X and the horizontal plane at level 700 m., defining the lower limit of the briquette construction in the central zone of the model. The perspex sheets and the timber board provided the datum which was necessary to guarantee a good start to, and a rapid development of, the briquette construction, Plate 18.
Briquettes were used throughout the remaining valley structure above the light aggregate concrete base. The best method of construction was thought to be a wall by wall procedure, which involved working upstream and then downstream from the timber board providing the datum. This was found, however, to be impracticable due to a tendency for the wall to topple because of the dip of the schistosity planes and the plasticity of the joint material. Consequently, the briquettes were generally laid layer by layer.

The procedure used in the first model for locating and cutting fault planes, was also adopted for the second model. With an eightfold increase in the number of briquettes, bringing the number to approximately 2500, there was a definite need for an improvement in the briquette cutting facilities; the result was the cutting machine shown in Plate 19.

The shear keys, piles, mid-height spillway and stilling basin were all precast in timber moulds. To prevent them from being damaged during the construction of the valley, either concrete or timber mock-ups were used to enable the surrounding briquettes to be cut, and drilled if necessary, to the correct shape. In order to insert a shear key, the surrounding briquettes were cut, the briquettes below the shear key were bedded in the valley, and the joint material was allowed to set. The shear key was then bonded to the briquettes below it with Certite which, after it had set, was also used to bond the briquettes above it, to the shear key. A similar process was used to install the two horizontal piles designed to strengthen the right abutment, Plate 20.

While the valley was being constructed, the downstream mould for the dam was altered to include a socket on the right bank into which the ring beam would be cast. The dam was cast, as before, upside down, with the thin crest of the dam at the bottom and the gravity loading rods protruding from the top.
The procedure for inserting the dam into the valley was slightly different from that used in the first model. Before, the holes for the gravity loading rods for the dam and foundation were extended through the briquettes to the surface by drilling upwards from below the base plate. When this was tried for a second time, however, the drill began to dislodge the briquettes as it approached the surface, and this was obviously due to their small size. Consequently, the briquettes were replaced and a drilling rig was built to enable the holes to be drilled from above.

After assembling the foundation gravity loading system of the dam, and applying full scale load to the foundation, the monoliths and precast left abutment were assembled on a mortar base so that gaps, representing construction joints, were left between the monoliths. This time, however, the right abutment was cast in-situ around the projecting ends of the horizontal piles. When the mortar base had cured, the dam and valley in the immediate vicinity of the monoliths were reduced in temperature to 5 - 6°C, Plate 21. The construction joints and the gap between the reaction blocks and the left bank of the model were then grouted, and when the grout had set, the temperature was increased to the normal laboratory temperature. This procedure was adopted in the hope that a small amount of pre-stressing would be induced in the dam which would, in turn, ensure that the construction joints were grouted satisfactorily. It was impossible to tell whether this was entirely successful, but the construction joints looked considerably better than those of Model I.

When the grouting had cured, the stilling basin was bonded to the valley and the ring beam on the downstream side of the right bank was cast in-situ, Plate 22. The beam contained the same percentage area of reinforcement as the prototype, and eight holes, which were later extended by drilling to the upstream limit of the model, Fault 31. To represent the anchoring of the beam to the valley by cables, a wire was passed through each hole and connected at its upper end to a steel block resin bonded to the upper surface of the beam. At its lower end,
the wire passed over a series of pulleys to a cylinder containing enough lead shot to produce the correct amount of anchorage. In order to study the effect of anchoring the beam to the valley, the cylinders were placed on a tray which could be lowered or raised so that the wires did, or did not, support the weight of the cylinders.

The mid-height spillway was idealised as a rectangular beam having a cross-sectional area proportional to the amount of concrete in the cross-sectional area of the prototype. In the model, the spillway was embedded into the valley using Certite, with its upper surface just above the valley surface.

With the valley and dam construction complete, strain gauges were placed in complementary positions on the upstream and downstream faces of the dam, and attention was turned to the construction of the test gear.

3.4 DESIGN AND CONSTRUCTION OF TEST GEAR

The loading rig was designed to withstand a gravity load factor of 2.5 and a hydrostatic load factor of 5.0.

As in the first model, gravity loading was applied to the main body of the dam by passing tension rods through the foundation to spreader beams and linkages operated by hydraulic jacks below the base plate of the model. The gravity loading at the crest of the dam and on the abutments was applied by hydraulic jacks acting on steel plates and reacting against an overhead reaction frame.

The method of applying gravity load to the foundation was altered because of the difficulties created in the first model by stress concentrations around the gravity loading points. It was therefore thought preferable not to vary the load on the foundation, but to use the same method of applying the load to both left and right banks. Each of the thirteen gravity loading rods was passed through a short steel beam and connected to its underside by a wedge type grip. The steel beam was welded at each end to a steel tube containing a stiff spring. The springs were compressed and held in position by passing steel pins
through holes in the sides of the tube. The position of the holes was, of course, governed by the spring stiffness and the force required in the rod to produce the correct foundation load. The springs reacted against the underside of the steel base plate.

The results of Model I suggested that insufficient gravity load was applied to bed down properly the briquettes on the left bank. The gravity loading system used for the right bank in the first model was therefore extended to the left bank in the second, Plate 23. The gravity loading on the valley was applied by means of hydraulic jacks fitted with extension tubes. Each extension tube acted on a triangular steel plate which distributed half the load directly to two pedestals, and the other half, via a spreader beam, to two more pedestals. On the right bank, each pedestal was welded to a steel plate which distributed the load to four briquettes, the plate being fixed to the briquettes by four steel shear connectors, one in each briquette, and bonded to them with Certite. On the left bank, in the region just downstream of the dam, the method was the same as on the right bank except that each pedestal served two briquettes and each briquette had two shear connectors; further downstream, each pedestal served one briquette and had two shear connectors. The pedestals were so arranged that by using 10 tonne hydraulic jacks on the left bank and 20 tonne hydraulic jacks on the right bank, and linking them all to the same pressure source, the correct stress distribution was obtained at a level of approximately 40 m. below the valley surface.

Hydrostatic loading on the dam and grout curtain was applied by means of hydraulic jacks acting on thick steel pads. Each pad had a shaped glass fibre surface which distributed the load to the dam. Between the glass fibre surface on the loading pad and the upstream surface of the dam, there were two layers of strip rubber, at right angles to each other, which transmitted the load uniformly to the surface of the dam. The hydraulic jacks reacted against a steel backing plate bolted to concrete blocks which were prestressed to the strong floor of the laboratory.
The extension of the right bank gravity loading system to the left bank, created such a large increase in the amount of load which had to be applied to the valley surface that the overhead reaction frame had to be completely redesigned. The conglomeration of valley gravity loading jacks shown in Plate 24 reacted against three thick steel plates. The plates were clamped, side by side, to the underside of four deep, high yield steel I-beams, by bolts which passed between the I-beams and through beams placed on top of, and at right angles to, the I-beams. The bolts were post-tensioned so that the beams and plates would provide some torsional stiffness to the four I-beams. Slip between the plates and the I-beams was prevented by placing steel blocks against the lower flanges of the beams and welding them to the plate. At their ends, the I-beams were placed between four pairs of solid steel beams, the two lower pairs of which rested on steel columns. Inside each column was a 60 tonne bolt which extended from the underside of the laboratory strong floor to the upper pairs of solid beams on top of the four I-beams. Twenty bolts were used to resist the upward thrust of the valley gravity loading jacks, and each was post-tensioned to a load of 40 tonnes in order to provide the 800 tonnes required for a gravity load factor of 2.5. Plate 25 shows the overhead reaction frame and the ties, which were connected to the ends of the outer set of columns, to counteract the horizontal thrust produced by the hydraulic jacks going out of plumb when the model failed.

The reaction blocks at the sides and downstream end of the model were prestressed to the laboratory strong floor by 60 tonne bolts. In Model I, sufficient bolts were used to prestress the reaction blocks at the upstream and downstream ends of the model to enable the hydrostatic load on the dam and grout curtain to be increased to a load factor of 2.9. At this load factor, the reaction blocks at the downstream end were on the point of slipping, and those at the upstream end began to overturn. To enable a load factor of 5.0 to be applied, the number of bolts and reaction blocks at the downstream end was doubled. At the upstream end, however, the only way the reaction blocks could be
prevented from overturning was by either increasing the number of bolts, or increasing the bolt loads. As there were insufficient anchor points in the strong floor, the number of bolts could not be increased. Consequently, special maraging steel bolts were made with an ultimate strength of 130 tonnes which, for safety, were heat treated after machining, inspected for cracks, and loaded to 110 tonnes in an Amsler testing machine to determine their stress-strain characteristics.

Four loading cabinets were used so that different load combinations could be applied to the model. Gravity load could be applied separately to the valley, the crest of the dam and the body of the dam. The fourth loading cabinet was used to apply hydrostatic load to the dam and grout curtain.

3.5 INSTRUMENTATION

The strains on the upstream and downstream surfaces of the dam were measured by electric resistance strain gauges of 10 mm. gauge length. The displacements of the dam, abutments and valley were measured by transducers. Figures 88 - 91 show the points at which the strains and displacements were measured and their corresponding channel numbers. Figure 92 shows the positions of the transducers on the valley in relation to the hydraulic jacks which applied gravity load to the valley.

For each loading condition, in order to detect a faulty gauge or transducer, or a sudden change in behaviour of the model, two successive scans were made of the strain gauges and transducers by a Compulog Data Logger scanning at 15 channels per second. Those channels for which the two readings differed by more than 5 units were immediately printed out by the logger. The first indication that the valley was beginning to move was provided by the logger when it began to print out all the channel numbers. In order to reduce the printing time so that further readings could be taken during failure, the tolerance
of 5 units was increased to a large number. The results for each test were recorded on punched tape which was used for computer analysis.

The transducers measuring the radial displacements of the dam and the vertical displacements of the abutments, were mounted on a special frame. All the other transducers were strapped to angle-irons and fixed to either the valley surface or the top of the abutments, Plate 28. Each transducer made contact with a ball-bearing which, for a transducer mounted on the special frame was attached to the model, and vice versa.

The special frame was mounted on a ball seating, a roller bearing which only permitted movement normal to the axis of the valley, and a bearing which offered only vertical support. The ball seating was placed on the upstream end of the reaction blocks along the side of the left bank, the roller bearing was placed in a similar position on the right bank, and the remaining bearing was placed on the reaction blocks at the downstream end of the model.

3. 6 EXPERIMENTAL PROCEDURE

In an effort to reproduce the conditions on site at the end of construction, gravity loading was applied to the foundation of the dam before the monoliths of the dam were bedded into the valley. The dam and the adjoining part of the valley were cooled to 5 - 6°C, and the construction joints between the monoliths grouted. The temperature was then gradually increased to the normal laboratory temperature, in the hope that a small amount of prestressing would be induced in the dam to ensure that the construction joints were grouted satisfactorily.

A total of 57 loading cases was undertaken. They can be divided conveniently into the following three groups:

a) Load Cases 1 - 33 - Preliminary Tests
b) Load Cases 34 - 47 - Tests for Failure by Crushing
c) Load Cases 48 - 57 - Tests for Failure by Sliding.
Due to the difficulties encountered with the foundation gravity loading system in the first model, the foundation load was kept at its full scale value throughout all the tests in the second. Also, in the light of the first model results, no special arrangements were made to study the model behaviour with both Faults Eloisa and X prevented from moving, nor to study the possibility of sliding on Fault Eloisa with Fault X prevented from moving. Instead, both the left and right banks were completely free to move downstream and sufficient gravity loading was applied to the surface of the valley to produce the correct stress distribution at a depth of 40 m. below the surface. On the right bank, this corresponded with Fault X.

A summary of all the load cases is given in Figures 93 - 95.

3.6.1 Preliminary Tests

The objective of the preliminary tests was to obtain the behaviour of the model dam under full scale loading without unduly overstressing the dam.

Initially, the gravity and hydrostatic loads were gradually increased together to half their full scale values to ensure that the model and the test gear were performing satisfactorily. Following this, the loads were increased to their full scale values, and later, the gravity and hydrostatic loads were increased alternately in increments of 0.25 times their full scale values, so that the stresses due to gravity load and to hydrostatic load could be separated.

3.6.2 Tests for Failure by Crushing

The tests in this series were designed to explore the extent to which the model could sustain overload conditions without failure by crushing. All loads were therefore increased proportionally beyond their full scale values. The design of the overhead reaction frame, however, limited the load factor to a maximum of 2.5.
3.6.3 Tests for Failure by Sliding

This series of tests, designed to alter the resultant load path and achieve failure in the model by sliding, can be divided into the following three major tests:

a) the gravity and hydrostatic loads were increased to a load factor of 2.0 and then the hydrostatic load factor was increased to 4.0. Owing to a loss of pressure in the hydraulic system, the test was interrupted by inserting a test in which

b) the gravity and hydrostatic loads were increased to a load factor of 1.75, and then the hydrostatic load factor was increased to 3.5. As the model was stable under this loading, the gravity load factor was gradually decreased to 1.0.

c) the first major test (a) was repeated and followed by a gradual reduction in the gravity load factor until failure occurred.

3.7 RESULTS AND DISCUSSION

The stresses and displacements were obtained from the strains and displacements measured at the locations shown in Figures 88 - 90. The results are presented in terms of prototype behaviour:

3.7.1 Results of the Preliminary Tests

Figures 96 - 103 present the load factor-channel reading characteristics of a selected number of strain gauges during the preliminary tests designed to determine the behaviour of the dam under full scale loading. One would expect them to show slight changes in the behaviour of the model, during the early stages of loading, due to the settlement of the briquettes in the lower regions of the dam under gravity load, and in the upper regions under hydrostatic load. After a few repeats of the loading sequence, however,
the model should have established itself into a reasonably linear and consistent mode of response.

Figure 96 shows clearly the difference between the expected and unexpected types of behaviour. The results presented in Figures 96 - 101 show that for the majority of strain gauges on the left bank side of the dam, quite considerable changes of behaviour occurred before full scale loading of the model had been achieved. The largest changes took place in the cross-gauges of the rosettes at the bottom of the downstream face. The behaviour of the gauges well away from the foundation was better, as was the behaviour of those on the upstream face.

The characteristics of the strain gauges on the upstream and downstream faces of the right bank side of the dam are good, Figures 102 - 103. They show less change and a reasonably linear response; also, as there were no significant changes in the gradients of the graphs, there was no loss in stiffness due to repeated loading of the structure.

The general picture is therefore that the behaviour on the upstream face is more linear than on the downstream face, and that the behaviour on the right bank is considerably more linear than on the left bank. The unexpected behaviour of the rosettes near the foundation on the left bank appears to be connected with a discontinuity in the behaviour of the dam and valley interface. Some of the gauges indicate a change from a tensile strain to a compressive strain as the load factor was increased. There may be four reasons for this: settlement of the dam on its foundations, the effects caused by Fault 31 undercutting the centre monolith of the dam, non-linear effects associated with the structural behaviour at the dam and valley interface on the left bank, and local stress concentrations due to the two methods of simulating the gravity load of the dam.

The unusual behaviour led to an investigation into the separate effects of gravity and hydrostatic load. The effects were achieved by increasing the gravity load and hydrostatic load alternately in 0.25
increments until full scale loading had been applied, calculating the strains for each increment, and summing the four relevant sets of strains for each type of load.

Figure 104 shows the hoop and vertical stresses due to gravity load. The fluctuations in the hoop stresses at level 867, and to a lesser extent at level 842, and the strange vertical stress patterns in the upper half of the dam, are probably the result of local bending effects created by the crest gravity loading system. Except near the crest, the mean hoop stresses are very small, indicating that bending rather than arching, is the predominant structural action in the horizontal plane. Neither action, however, is really significant. The vertical stresses are also out of character in the lower regions of the dam where the mean vertical stress is reasonably constant except for a slight increase at the base of the crown cantilever; the most plausible cause of the irregularities is the reduction in cross-sectional area of the base resulting from the central monolith overhanging Fault 31. Some of the irregularities in the curves may also be due to stress concentrations around the internal gravity loading discs.

Figure 105 shows the principal stresses due to gravity load. The principal stress directions, except on the right bank side of the downstream face where the ring beam must have some influence, suggest that cantilever action predominates. The behaviour is not symmetrical.

The hoop and vertical stresses due to hydrostatic load are given in Figure 106. The irregularities in the curves representing the hoop stresses at levels 867 and 842 are most likely due to the hydrostatic and gravity loading plates at the crest being welded together and causing some interaction between the two types of loading. The hoop stresses at these two levels, however, show only minimal horizontal bending at the ends of the arches in comparison with those at levels 817 and 792. This is not entirely unexpected since the length to thickness ratios of the arches rapidly increase with height: for the levels
792, 817, 842 and 867, the ratios are approximately 9, 13, 24 and 58.
At the crest, the mean hoop stress drops off at the left abutment with
the result that the mean hoop stress at the right abutment is higher.
Lower down, the mean hoop stress is reasonably constant throughout
the length of the arch.

The vertical stresses show the typical cross-over pattern
one would expect due to hydrostatic load. A comparison of the stresses
for the left bank, crown and right bank cantilevers, shows that there
was more significant bending action at higher levels on the left bank
than on the right bank, while at the base of the cantilevers the situation
was the opposite. This, and the slightly greater tailing off in the curve
for the upstream face of the left bank cantilever, suggest that the foundation
on the left bank was more flexible than on the right bank.

Figure 107 shows the principal stresses due to hydrostatic
load. A typical arching action is evident on both the upstream and
downstream faces. The principal stress directions on the upstream face
appear to be less systematic, and it is possible that they may have been
affected by the method of applying hydrostatic load. The highest stresses
occur on the left bank at the bottom of the downstream face, where three
of the minimum principal stresses seem unreasonable in comparison
with their counterparts on the right bank.

The special frame carrying the transducers was at this stage
mounted on the laboratory floor which, quite obviously, deflected under
the enormous gravity load acting on the valley. The floor, however,
deflected more than expected and caused the frame to move during the
tests. All the displacements recorded during the preliminary tests
have therefore been ignored.

In conclusion, it is apparent that the behaviour of the dam was
affected by the behaviour at the left bank dam and valley interface.
Its effect was to increase the flexibility of the foundation and produce
some unusually high stresses at the bottom of the downstream face.
3.7.2 Results of the Tests for Failure by Crushing

From the strains recorded during the preliminary tests, and an inspection of the dam which was somewhat limited because the valley gravity loading system made the dam almost inaccessible, it was evident that no serious stress levels had been reached in the build up to full scale loading. The tests in this series were therefore designed to explore the extent to which all the loads could be increased proportionally beyond the full scale value.

Although the results for load factors up to and including 2.0 are presented in Figures 108 - 123, a maximum load factor of 2.1 was achieved. At this load factor, however, testing was stopped because local crushing failures began to occur around the loading pedestals on the left bank of the valley.

The load factor-channel reading characteristics in Figures 108 - 115 resemble those of the preliminary tests in which the gauges on the right bank side of the dam behaved as expected, and the majority of those on the left bank side did not. The change in behaviour of strain gauge no. 200 on the upstream face at the base of the right bank cantilever probably occurred because a hydraulic jack, applying gravity load to the valley on the downstream side of the dam, jumped out of its seatings at a load factor of 2.0, crashed onto the valley and caused Load Case 46 to be abandoned.

Figures 116 - 120 record the hoop and vertical stresses in the dam for full scale loading and load factors of 1.25, 1.5, 1.75 and 2.0. The first thing that is noticeable about the curves is that their general shapes do not alter with increasing load: this immediately indicates that the dam suffered no serious damage during this series of tests, although its behaviour was not necessarily linear.

The hoop stresses in Figures 116 - 117 show the local bending effects at the crest caused by the gravity loading system, and the more significant horizontal bending action in the lower regions of the dam where
the arches are much thicker and shorter. In the upper regions of the dam, the mean hoop stress on the right bank is greater than on the left bank, and the difference between them increases with increasing load. At the three lower levels, the erratic behaviour of the hoop stresses at the left bank end of the arch, probably originates from non-linear behaviour at the dam and valley interface. The hoop and vertical stresses in Figures 118 - 120 confirm the results of the preliminary tests by showing the effects of Fault 31 undercutting the central monolith, the more flexible left bank, and the crest gravity loading system.

The principal stresses for load factors of 1.0, 1.5 and 2.0 are presented in Figures 121 - 123. They show that as the load was increased, although there were no significant changes in the principal stress directions on the upstream face, there were quite large clockwise rotations and small counter-clockwise rotations, on the left and right bank sides of the downstream face respectively. This indicates that the load path was gradually changing to one where a greater percentage of the load was being carried by the upper half of the dam in horizontal arching action, and suggests that the foundation was becoming more flexible. The results show extremely large compressive stresses on the left bank at the bottom of the downstream face. However, as they exceed 321 kg/cm\(^2\) the ultimate strength of the concrete in uniaxial compression, and no favourable biaxial stress system is present, they must not be regarded as actual stresses, but rather as products of Young's Modulus and the strain. They are possibly evidence of cracking.

During this series of tests, the displacements were ignored due to movement of the special frame during testing. It is also evident that by the end of the series some kind of damage had occurred along the left bank dam and valley interface. Nevertheless, the tests do show that an overall failure of the model was not imminent.
3.7.3 Results of the Tests for Failure by Sliding

The final series of tests was designed to induce a sliding failure by altering the resultant load path in the model. The results, which include the displacements for the first time, are presented in Figures 124 - 157; they conclude with two diagrams showing the locations of cracks in the dam and valley after failure, Figures 158 - 159.

The first major test consisted of increasing the gravity and hydrostatic load factors to 2.0 and then increasing the hydrostatic load factor to 4.0. The results of the test are given in Figures 124 - 133 and show, because of their consistency, no signs of impending failure. On the right bank, however, there does appear to be a discontinuity in the hoop stresses on the downstream face at level 867, Figure 124. Figure 116 shows that an unusually high hoop stress always existed in the locality, and Figure 132 shows an unusual, but compatible, upstream movement of the crest in the same region. Further reference to the cracks observed in the dam after failure had occurred, Figure 158, shows a vertical crack in one of the construction joints on the right bank. This suggests that the construction joint began to open up at a fairly early stage in the testing programme but, from the hoop stresses and displacements, it does not appear to have extended much below level 842. It was possibly caused by a hydraulic jack of the crest gravity loading system being slightly upstream of its correct position.

The results also show that, as the hydrostatic load factor was increased, the amount of horizontal bending and the mean hoop stresses on the right bank increased. The behaviour of the left bank is difficult to assess due to the behaviour at the valley interface, but because of a decrease in the mean hoop stress at level 792 and no significant increases at level 817, it appears that the right bank side of the dam supported a greater proportion of the load. Once again, the vertical stresses in Figures 126 - 128 show the development of more significant vertical bending action on the right bank than on the left bank. This is not at all surprising since the left bank is already considered to be more flexible,
whereas on the right bank, the hydrostatic load on the grout curtain resists the tendency which the briquettes have to topple upstream due to the orientation of the schistosity planes. The directions of the principal stresses, Figures 129 - 131, show no marked differences. On the downstream face, however, as the hydrostatic load factor was raised above 2.0, the compressive stresses at the valley interface on the left bank decreased, while those on the right bank increased. This suggests that the load was being shed from the left bank to the right bank by a greater arching action than one might expect, and it is confirmed by the near horizontal minimum principal stress directions on the downstream face.

The radial displacements shown in Figures 132 - 133 show a bulge in the central zone of the dam which reflects the large hoop stresses in Figure 127. The radial displacements of the cantilevers show that the base of the dam moved first upstream, then downstream, which probably occurred because Fault 31 undercut the dam and the schistosity planes dipped downstream. The crest, however, moved progressively upstream.

The second major test was undertaken because the first showed no signs of failure. It consisted of increasing the gravity and hydrostatic load factors to 1.75, then increasing the hydrostatic load factor to 3.5, and finally, reducing the gravity load factor to 1.0.

A comparison of the results, Figures 134 - 143, with those of the first major test, Figures 124 - 133, shows that the general behaviour is the same and that the gravity load has virtually no effect except to provide the dead-weight to prevent sliding. Figures 142 - 143 show that the right bank moved more than the left, and that the decrease in gravity load produced a considerable increase in the radial displacements throughout the dam. On the crown cantilever, the increase was approximately 55 mm. at the base and 97 mm. at the crest. Since similar movements were also recorded by the left and right bank cantilevers, it appears that the valley was also beginning to move and that the peak
strength of the joint materials may have almost been reached.

The third major test consisted of increasing the gravity and hydrostatic load factors to 2.0, then increasing the hydrostatic load factor to 4.0, and reducing the gravity load factor to slightly less than 1.0, when failure occurred with a slide on the right bank on Fault X. When sliding began, the pressures in the gravity and hydrostatic loading systems dropped in an attempt to re-establish equilibrium. It was during this attempt that the last set of strain readings was taken with the load factor on the valley gravity loading at 0.8 and that on a hydrostatic loading at 3.6.

The loss of several gauges during the test does not prevent the hoop and vertical stresses in Figures 144 - 148 from showing that there was a larger change in stress when the gravity load factor was reduced from 1.5 to 1.0 than from 2.0 to 1.5. It was during the reduction from 1.5 to 1.0 that a hairline crack developed between the upstream face of the left abutment and the briquettes. The hoop stresses show that during failure there was greater horizontal bending action at the left bank than at the right bank, thus inferring that the right bank yielded and the dam pivoted about the left bank. Except at level 792, the hoop stresses show no large tensile stresses and it would therefore seem reasonable to assume that the top half of the dam remained intact. The minimum principal stresses were always less than the ultimate strength, Figures 149 - 152. However, the tensile stresses at the valley interface may be evidence of cracking. The reduction in gravity load produced only slight changes in the directions of the principal stresses, large changes in the maximum principal stresses associated with vertical bending and, for the reduction from 1.5 to 1.0, changes in the minimum principal stresses which indicated that some of the load being taken by the left bank was redistributed to the right bank. The maximum principal stresses in Figure 152 show that nearly all of the downstream face was in a state of vertical tension and nearly all of the upstream face was in compression, indicating that during failure the dam bulged under the hydrostatic load. At the base of the dam on the downstream side, tensile
stresses existed from the right bank cantilever to the left abutment; also, the direction of the minimum principal stresses near the base tended to lie parallel with the foundation line. While the tensile stresses are possible evidence of cracking, the directions suggest a reduction in shear resistance at the valley interface with a consequent increase in hoop compression in the upper portion of the dam. The hoop stresses in Figure 144 certainly confirm the increase in hoop compression.

The radial displacements of the arches in Figure 154 agree with the hoop stresses in showing a larger increase when the gravity load factor was reduced from 1.5 to 1.0 than from 2.0 to 1.5. They also show that, although the left bank side of the dam moved the most, the overall movements of the left and right banks were only appreciably different at level 817. The validity of the results for the right bank at level 817 are questionable, because of the consistency of the results at the other four levels. The radial displacements of the cantilevers in Figure 155 show a progressive downstream movement, and an increase in displacement of approximately 850 mm. at the base of each cantilever when failure occurred. The unusual shape of the curves for the right bank cantilever are probably due to the crest gravity loading system.

The displacements recorded in Figure 156 show that as the gravity load was reduced the abutments moved upwards and outwards. This movement was accompanied by a downstream movement at the left abutment and a rather unexpected upstream movement of the right abutment. The latter however, was probably because the hydrostatic load on the grout curtain rotated the abutment. The valley immediately downstream of the dam also moved outwards. All the previous movements agree with the requirements for failure, that the abutments must move upwards and outwards.

The displacements of the valley surface were generally insignificant until failure occurred, when they suddenly increased. Figure 157
shows that the region below a line drawn from the right abutment to the top of the downstream end of the right bank, moved approximately 1 metre. As the movement above this line was only approximately 275 mm., the right bank appears to have separated into two sections. The results also show an upstream movement of the right bank at the downstream end of the model, where there was insufficient gravity load applied to the valley surface to prevent the large movement of the right bank causing a row of briquettes to move up the schistosity plane formed by the row of briquettes in front.

After the third major test, the gravity and hydrostatic load factors were increased to 0.5, and the hydrostatic load factor was then increased to 2.8 to produce the ultimate failure of the model. The right bank began to move at a hydrostatic load factor of 2.5. However, it stabilised itself and remained stable until the load factor reached 2.8, when the downstream ends of both the left and right banks moved, the right abutment cracked, and the hydrostatic load decreased as a major slide on Fault X occurred.

After the ultimate failure of the model, the test gear was dismantled so that the model could be examined. Plate 26 shows that the dam and its foundations moved together downstream on the horizontal plane at level 700. In doing so, cracks developed along the valley interface on the left bank, which must have reduced the shear resistance at the interface and increased the hoop compression at the crest, to produce the failure of the left abutment, Plate 27. An extensive horizontal crack formed in the left bank side of the dam above the internal gravity loading points, at the boundary separating the internal and external gravity loading systems for the dam. The crack, which is shown in Plate 28, reflects the tensile maximum principal stresses on the downstream face recorded in Figure 153. On the right bank, failure occurred with a major slide on Fault X, and cracks developed only in the outer regions of the abutment block, Plate 29. The cracks observed in the dam after failure are recorded in Figure 158; they indicate that two construction joints on the left bank opened, probably due to the dam rotating about the left bank,
and one on the right bank opened, which the results suggest happened before any attempt was made to induce failure by reducing the gravity load factor.

The removal of the left abutment and dam enabled the foundation to be inspected. On the left bank, a crack was found immediately below the abutment, together with some dry pack along the downstream edge of the foundation which had been crushed to powder, and some cracks in the dry pack in the lower region of the bank, Plate 30. The cracks extended from the left bank, across the central zone, to the right bank where the dry pack had crumbled along the downstream edge. The crushed dry pack on the left bank reflects the high compressive stresses obtained along the edge of the downstream face of the dam. The influence of the crumbled dry pack on the right bank was probably offset by the presence of the ring beam.

After removing the dry pack under the dam, cracks were found below the right abutment along Fault 16. There were also cracks in the firmly embedded briquettes of the left bank, running approximately parallel with the surfaces of the dam. The briquettes below the central monoliths were found to be loose, possibly having been loosened during the removal of the monoliths.

The ring beam and mid-height spillway on the right bank were removed intact. To inspect the piles in the right abutment, the surrounding briquettes were removed and the abutment was cut with a saw, Plate 31. The downstream pile was fractured at its junction with the abutment at 90° to the pile axis but there were no cracks between the pile and the valley. A fracture in the upstream pile in the same position, but at 45° to the pile axis, provided a link between a crack in the valley on the downstream side of the pile and a crack on the upstream side of the abutment. Together, they suggest that as the valley moved on Fault X, the briquettes downstream and upstream of the abutment separated, leaving the upstream pile behind. After removing the loading pedestals and the top layer of briquettes, a large open crack was found running up the bank.
from the end of the downstream pile to the downstream end of the model, Plate 32. A similar, but smaller crack was also found higher up the bank. The largest of the three shear keys in Fault Eloisa cracked, but since the crack was near a foundation gravity loading point, it was probably not significant. The shear keys were otherwise removed intact.

The removal of the loading pedestals and the top layer of briquettes on the left bank revealed that Fault 16 had opened, Plate 33. The crack shown in Figure 159 running from the left abutment to intersect Fault 16 at a point well downstream of the dam, was not present at this level. However, a continuation of the crack was apparent below and behind the site of the left abutment. The removal of a second layer of briquettes revealed the same crack pattern as the first layer, except for a few small cracks near the stilling basin. The left bank wall of the stilling basin was fractured. Excavations below the left abutment showed that the shear key in Faults 11, 12 and 13 was not damaged. Unfortunately, three holes were drilled through the shear key in Fault K for the gravity loading rods of the dam. It is therefore impossible to say whether the cracks in the key were due to differential movement between the briquettes and the light aggregate concrete base of the valley, or to the tightening of the gravity loading rods against the key when the valley moved. A gravity loading rod also passed through, and slightly damaged, the shear key in Fault 2. The major damage to the key, however, consisted of a continuation of the cracks running approximately parallel with the surfaces of the dam.

The locations of the major cracks in the valley surface are presented in Figure 159. The pattern suggests that the region between Fault 16 and the main crack formation on the right bank, moved bodily downstream when failure occurred, and in doing so, produced the large open cracks along Fault 16 and the lower of the two cracks on the right bank.
3.8 SUMMARY

A second model of the El Atazar Dam has been constructed to a scale of 1:200 and tested to failure. The model included various strengthening measures which were omitted from the first model. The valley was built using smaller briquettes and gravity loading was extended to the left bank.

In the light of the results of the first model, the second model was designed to study the conditions of failure on Fault X only, and the effect of uplift on the valley was ignored. The model was tested, however, in the same manner, first for failure by crushing and then for failure by sliding. It was found to be stable under a crushing load factor of 2.1, but, due to local failures below the loading pedestals on the valley, its stability under higher load factors could not be investigated. Failure took place with a major slide on Fault X when the hydrostatic load factor was 4.0 and the gravity load factor was just less than 1.0. An appreciable alteration in the resultant load path in the valley was therefore necessary to produce failure.

The results show that the behaviour of the dam was distorted because some kind of damage had been sustained, either to the strain gauges or to the model itself, along the left bank dam and valley interface. The exact nature of this damage will be discussed in Chapter 6, following the theoretical work of Chapter 5.
CHAPTER 4: COMPUTER PROGRAM

4.1 INTRODUCTION

The two-dimensional finite element model for analysing jointed rock behaviour developed by Goodman, Taylor and Brekke\(^{(15)}\) has been extended by Mahtab and Goodman\(^{(16)}\) to three dimensions. Their model overcomes the disadvantages of the extensively reported graphical and vector methods, which are limiting equilibrium analyses considering neither the deformability of the solid nor that of the structural weaknesses. They are also unable to model the non-linear load-deformation behaviour of a jointed rock mass.

The three-dimensional finite element model was therefore adopted because it allows the blocks of rock and the joints to behave as two distinct sets of physical elements. It is, however, inevitable that any reasonable three-dimensional representation of a doubly-curved arch dam in an asymmetric valley containing several fault planes, will require the solution of several hundred, if not thousands, of simultaneous equations. The economic solution of such a large number of equations became the fundamental problem.

The importance attached to the solution of symmetric positive-definite equations, as met in the finite element approach to structural analysis, is endorsed by the interest shown by numerous investigators. Their aim has generally been to modify the standard algorithms by taking advantage of the sparseness and symmetry of the matrix of coefficients, in order to reduce the number of calculations. Taking advantage of symmetry is straightforward, but recognising and implementing the distribution of the non-zero coefficients is not.

Irons\(^{(17)}\) and Melosh and Bamford\(^{(18)}\) have written papers on the frontal solution technique which takes advantage of matrix sparsity and symmetry, and is capable of handling efficiently problems with large variations in matrix sparsity. The technique is very suitable for analyses in which two-dimensional or three-dimensional elements with other than corner nodes are used, and since the use of such elements was envisaged,
the technique was adopted. Irons gives a comprehensive description and Fortran listing of a frontal solution program in his paper; it is from this that the general structural analysis computer system, now to be described, has been developed.

4.2 GENERAL DESCRIPTION OF PROGRAM

The general purpose finite element program has been developed in order to solve all types of two and three dimensional structures. In its present form only plane stress, plane strain, plate bending with or without shear deformation, thin and thick shell problems, two-dimensional field problems, and three-dimensional solid problems can be solved. It can, however, be quickly modified to include elements other than those shown in Figures 164 - 167 for analysing further types of problem. The program is written in Fortran and is intended for use on the CDC 6000 series of computers using either the FUN or FTN compiler. A special version also exists for use on the CDC 7600 computer. The program is designed so that it can operate within the Instant Turnround limits of 24K words and 18 seconds central processing time on the Imperial College CDC 6400 computer. In order to fulfill these requirements, the program is overlayed and stored on disc, and arrays stored dynamically.

The program is overlayed into seven parts, one of which is optional. Each part is activated in turn by the main program, which includes the two cards controlling the amount of computer storage required by the user in order to solve his specific problem. The first three overlays are concerned with processing the input data, the fourth with sorting the data and calculating all the element stiffness matrices, the fifth assembles the overall stiffness matrix and solves each load case by either the frontal technique or Choleski solution, and the sixth uses the displacements to calculate the stresses for each element and the reactions to earth at each support. The seventh overlay is optional and is a program supplied by the user to enable him to process the displacements and stresses according to his own requirements. A flow chart and summary of the main events in the program are shown in Figure 160.
4.3 OVERLAY (0, 0) - MAIN PROGRAM

The dynamic storage maps of Figures 161 - 163 show how the longworking vector ELPA is used during each stage of the program. As variables can only be linked from one overlay to the next by placing them in blank or labelled COMMON blocks, it was considered expedient to place ELPA in blank COMMON in view of the fact that its dimension would need alteration according to the size of problem to be analysed. Only two cards, therefore, require changing in order to increase the size of ELPA and hence the size of the problem. In most subroutines ELPA acquires, among other names, the variable name NLPA, permitting the vector to store both real and integer numbers. This is possible because real and integer variables on the CDC 6000 and 7000 series computers have 64 bits per word.

The main program, OVERLAY (0, 0), consists of a short routine which includes the two cards controlling the size of ELPA, a subroutine which executes each of the seven primary overlays in turn, and several subroutines which are required by some or all of the primary overlays. The main program remains in the central memory of the computer throughout execution.

4.4 OVERLAY (1, 0) - DATA PROCESSING 1

The data input required by the program has been planned so that its meaning should be self-evident. It may contain data for any number of independent problems, each problem consisting of a set of input items. In general, an input item comprises a header statement followed by one or more data statements and terminated by a blank card. As a special case, however, it may be only a header statement. A typical set of input items for a computer run appears on the next page.
PROBLEM
STRUCTURE
OPTIONS
NODAL POINT COORDINATES
HEXI8 ELEMENTS
ELASTIC PROPERTIES
SUPPORT CONDITIONS
LOAD CASE
COMBINED LOAD CASE
DATA FOR DUMMY OVERLAY

PROBLEM

PROBLEM

END

Overlay (1, 0) is the first of three overlays which process the input data. It deals with the first five items listed above. The input item PROBLEM identifies a new problem while the type of structure is specified by means of the STRUCTURE card.

The purpose of OPTIONS is to enable the user to have some control of the program, albeit a limited one. Facilities exist to reduce the amount of data checking and the number of lines of output, and to specify the frontal technique to enable the automatic check for determining the method of solution to be omitted. These three facilities are especially useful when solving very large problems, as considerable savings in execution time can be made.
A further two options are concerned with numerical integration and its application to the calculation of stiffness matrices for isoparametric elements. Bates\textsuperscript{(19)} repeated the work of Zienkiewicz, Taylor and Too\textsuperscript{(20)} which proposed a uniform reduction in the order of numerical integration for all strain components, in order to overcome the parasitic shear problem associated with the curvilinear plate and shell elements available in the computer program. Pawsey and Clough\textsuperscript{(21)}, however, suggested a selective reduction applicable only to the transverse shear strain components. Zienkiewicz et al found that their technique led to a remarkable, and quite unexpected, general improvement in element properties. Not only did it overcome the parasitic shear problem, but it also improved the convergence characteristics of the element in both thick and thin plate and shell situations. As tests on isoparametric plane stress elements showed similar improvements, the recommendation\textsuperscript{(22)} was forwarded that most numerically integrated elements can, in most circumstances, benefit from a reduction in the accuracy of integration.

The recommendation is implemented in the computer program by the use of the finer integration mesh and full integration options. The computer assumes that the user requires explicit integration in the thickness direction, $\zeta$, and a reduced integration mesh with respect to the $\xi$ and $\eta$ directions. The tables on the following page indicate the types of mesh which are available for calculating the element stiffness matrix by Gaussian quadrature formulae. The 14 and 27 point rules suggested by Irons\textsuperscript{(23)} for solid elements are replacements for the $3\times3\times3$ and $4\times4\times4$ product Gauss rules.

The remaining options permit the user to:
<table>
<thead>
<tr>
<th>Type of curvilinear element</th>
<th>Numerical integration w. r. t. $\xi, \eta$ with explicit integration w. r. t. $\xi$</th>
<th>Numerical integration w. r. t. $\xi, \eta$* with full integration w. r. t. $\xi$</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>reduced integration mesh</td>
<td>finer * integration mesh</td>
</tr>
<tr>
<td>Quadratic</td>
<td>2x2</td>
<td>2x3</td>
</tr>
<tr>
<td>Cubic</td>
<td>3x3</td>
<td>4x4</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Type of solid element</th>
<th>Reduced integration mesh</th>
<th>Finer * integration mesh</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear</td>
<td>2x2x2</td>
<td>14 pt. rule</td>
</tr>
<tr>
<td>Quadratic</td>
<td>14 pt. rule</td>
<td>27 pt. rule</td>
</tr>
<tr>
<td>Cubic</td>
<td>14 pt. rule</td>
<td>27 pt. rule</td>
</tr>
</tbody>
</table>

* Options
a) Stop execution immediately after all the data input cards have been read and checked, and the method of solution determined. If the data is correct the problem can be re-run with the low level data checking option specified in order to reduce execution time. Execution, however, is always terminated by errors in the data or NLPZ, the dimension of ELPA being too small.

b) Specify that a dummy overlay is being supplied so that the displacements and stresses can be processed according to the user's own wishes.

c) Request the calculation and output of the average nodal point stresses.

d) Specify orthotropic materials for the curvilinear plate and shell elements by supplying the angle of orthotropy at each nodal point, in either the global or local coordinate system.

e) Specify a nodal variation in thickness for the curvilinear plate elements.

f) Request the output of the element-by-element progression of the wavefront, when solution is by the frontal technique.

g) Request the output of the long vector ELPA/NLPA at the end of data processing.

h) Request the output of the data supplied to the element stiffness matrix subroutines, and the stiffness and stress matrices for each element. These facilities are very useful when a new element is being added to the system.

The necessity for automatic mesh and element generation schemes is immediately obvious to any user of the finite element method. They not only help to reduce the time spent on the tedious preparation and checking of data, but also narrow the likelihood of human error by enabling the user to concentrate on a relatively small amount of data.
The main feature of the nodal point generation facilities is the Zienkiewicz and Phillips' scheme\(^{(24)}\) for curvilinear mapping of parabolic quadrilaterals. The scheme allows a unique coordinate mapping of curvilinear and Cartesian coordinates by using the shape functions of the 8 node isoparametric quadrilateral plane stress element. The scheme, however, is only possible if the region in which the mesh is to be generated can be described adequately by the coordinates of the four corner nodes and the four mid-side nodes, and the number of divisions required along the edges. If equal spacing is required along an edge, the mid-side nodes must be at the mid-point, otherwise the mesh will be graded. The quadrilateral can be quite severely distorted, even to the point of making two sides lie on the same line so as to make a triangle. Corner angles, however, must not be greater than 180° as non-uniqueness of mapping may result.

If the coordinates of a mid-side node are not specified, the program assumes the node to be the mid-point of the straight line joining the two appropriate corner nodes. If no mid-side nodes are specified, the mesh is generated on the basis of a quadrilateral with straight sides with equal subdivision of each side. For this case, the shape functions of the 4 node isoparametric quadrilateral plane stress element are used.

Nodal point numbers and their coordinates can be specified in the program in the following ways:

a) The nodal point number and its coordinates are punched on a data card.

b) Straight or circular, single-line generation in which the first and last nodes of the line, and the increment to be added to the first and subsequent nodes, are specified. Provided that the coordinates of the end nodes have been specified, or calculated by previous generation, the line is divided into equal subdivisions. Circular generation is
achieved by declaring the nodal point number and coordinates of the centre of the circle. Generation is clockwise unless specified as anti-clockwise.

c) Multi-line generation which is a stack of single-line generations.

d) Quadrilateral generation.

The number of single-line, multi-line and quadrilateral generations is unlimited, and if the coordinates of a nodal point are redefined, the previous coordinates are overwritten.

The types of element currently available are given with their keyword and a brief description of their characteristics in Figures 164 - 167. The list is rather limited and restrictive, but it has been made dynamic in the sense that new and improved elements can be added quickly to the system as the need arises.

Each element is defined by a number and a set of nodal points. The order in which the elements are specified, or generated, is very important when solution is by the frontal technique, since the order defines the way in which the wavefront passes through the structure. For greater flexibility, a subroutine is required which would enable the user to specify the solution order; unfortunately this facility does not yet exist. For the Choleski method, however, the order is immaterial because the complete overall stiffness matrix is stored in the central memory of the computer.

Element sets are used in the program in order to reduce the number of locations required in ELPA to store the element information. Each element header card creates a new set and consists of an element keyword and either the word ELEMENTS, when each element is specified separately, or GENERATION, when they are to be generated.

A good, powerful, element generation scheme was needed for the program. As time was limited, the element generation scheme of ASKA\(^{(25)}\) was adopted, but in a different format. It is based on the ASKA topological
variable and its valuable first and second levels of repetition, and can be considered capable of generating either a row of elements, a group of rows, or several groups of rows. The data input format is as follows:

<table>
<thead>
<tr>
<th>R</th>
<th>k</th>
<th>R</th>
<th>. . .</th>
<th>P</th>
<th>r</th>
</tr>
</thead>
<tbody>
<tr>
<td>RI</td>
<td>Δk</td>
<td>ΔR</td>
<td>. . .</td>
<td>ΔP</td>
<td>r</td>
</tr>
<tr>
<td>RII</td>
<td>ΔΔk</td>
<td>ΔΔR</td>
<td>. . .</td>
<td>ΔΔP</td>
<td>r</td>
</tr>
<tr>
<td>C</td>
<td>Δk</td>
<td>ΔP</td>
<td>. . .</td>
<td>ΔP</td>
<td>A r</td>
</tr>
<tr>
<td>CI</td>
<td>ΔΔk</td>
<td>ΔΔP</td>
<td>. . .</td>
<td>ΔΔP</td>
<td>A r</td>
</tr>
<tr>
<td>L</td>
<td>Δk</td>
<td>ΔP</td>
<td>. . .</td>
<td>ΔP</td>
<td>ΔΔr</td>
</tr>
<tr>
<td>LI</td>
<td>ΔΔk</td>
<td>ΔΔP</td>
<td>. . .</td>
<td>ΔΔP</td>
<td>A r</td>
</tr>
<tr>
<td>LII</td>
<td>ΔΔΔk</td>
<td>ΔΔΔP</td>
<td>. . .</td>
<td>ΔΔΔP</td>
<td>A r</td>
</tr>
<tr>
<td>LCI</td>
<td>ΔΔΔk</td>
<td>ΔΔΔP</td>
<td>. . .</td>
<td>ΔΔΔP</td>
<td>A r</td>
</tr>
</tbody>
</table>

where R, RI, RII are the basic topological variable identifiers
C, CI are the first level of repetition identifiers
L, LI, LII, LCI are the second level of repetition identifiers
k is the starting value or first element number
P \_i is the starting value of the i\textsuperscript{th} nodal point
number of the element
r is the total number of elements to be generated in the basic statement
c is the number of first level repetitions, including the first unmodified execution of the basic statement
l is the number of repetitions to be made of the basic and first level statements, including the first unmodified pass.

The remaining terms are probably best explained by the flow chart in Figure 168 which shows, to those familiar with FORTRAN, that the scheme is a set of nested DO-loops.
At the end of Overlay (1, 0) the coordinates have been stored at the end of the long vector ELPA in descending numerical order, the element numbers stored immediately before the coordinates in the sets and order in which they were supplied, and the string of element nodal point numbers, corresponding to the element order, at the front of ELPA. After all the input data has been processed, however, the string of nodes is moved along the vector to leave an unused area of ELPA.

4.5 OVERLAY (2, 0) - DATA PROCESSING 2

Overlay (2, 0) processes the input data defining the elastic properties for each element and the support conditions of those nodes which have at least one degree of freedom constrained. The data supplied by the user, in accordance with the horizontal looping facility\(^{(26)}\), is stored in ELPA by working towards the front of the vector from the label NLPEO in Figure 161.

The horizontal loop facility expresses the series,

\[ i, i+k, i+2k, i+3k, i+4k, \ldots, j \]

where \( i, j \) and \( k \) are positive integers, as

\[ i, j, k \]

As a special case, the series may consist of only \( i \).

The mechanical properties of the material are used to describe the ELASTIC PROPERTIES of each element. Although the elastic modulii, Poisson's ratio, shear modulus, the element thickness and the angles of orthotropy are generally used to define the element properties, a facility exists which enables the rigidities of the element to be specified.

Each degree of freedom of a node under SUPPORT CONDITIONS must be specified as free, rigid or elastic, with all freedoms being referred to the global cartesian coordinate system of the structure.
A free degree of freedom is self-explanatory; a rigid one has a specified displacement, and an elastic one a specified spring stiffness.

4.6 **OVERLAY (3, 0) - DATA PROCESSING 3**

This, the last of the data processing overlays, handles the data associated with the loading on the structure, produces a summary of all the input data and determines the method of solution.

The LOAD CASE header statement initiates a new load case, the number of which is limited only by the size of the computer, as the program is dynamic in this respect. The data for each load case is supplied under the input items

- **CONCENTRATED LOADS**
- **SURFACE PRESSURES**
- **UNIFORMLY DISTRIBUTED LOADS**
- **CONSTANT BODY FORCES**
- **BODY FORCE POTENTIALS**

and the horizontal looping facility. An explanation of the last two input items is given by Zienkiewicz (27), the others being self-evident to the structural engineer.

The user can, by using the input item **COMBINED LOAD CASE**, factor any one or more of the load cases supplied by him, and combine them to form a new load case. The program does not, however, solve an extra load case but simply factors the relevant solutions of displacements and adds them together immediately before calculating the element stresses. The number of combined load cases is unlimited.

The data card following the last combined load case item specifies that it is the END of all the data or the beginning of a new PROBLEM. Both, however, direct the computer to output a data summary and, provided that there are no data errors, to select the method of solution.
The subroutine in which the method of solution is determined was the most difficult part of the program to write. Various reasons caused it to be rewritten several times, the last being caused by the need for random access storage devices for the back-substitution process in the frontal solution.

Unless the user has requested the frontal solution, the computer uses the string of element nodal points to calculate the total number of degrees of freedom for the structure and the semi-bandwidth of the overall stiffness matrix. Each time the semi-bandwidth is modified, the modified value is used to calculate the amount of storage required by the Choleski method of solution. The check is performed after each modification because the frontal solution, unlike the Choleski solution, does not rely on a node numbering system which attempts to minimise the spread of node numbers for each element. If the storage required is found to be greater than NLPZ, a flag is set to indicate that solution must be by the frontal technique.

Having determined the method of solution, each number in the string of element nodes is coded so that it:

- contains the nodal point number
- contains its destination in the list of current active nodes (ref. Section 4.10)
- indicates whether it is making its first, last, first and last, or intermediate appearance
- gives the number of appearances on its first appearance.

As this information is produced, the maximum number, MNODZ, of the list of current active variables, MNODS, and a slightly conservative estimate of the maximum size, M10, of the equation buffer length, EQU, are calculated, Figure 162.

The process for deciding whether or not the frontal solution can be used is complicated. The main effort is concentrated in reducing the number of random access records to a minimum and attempting to
solve all the load cases in the first solution phase. The back-
substitution phase, however, may sometimes prevent this and the
re-solution phase has to be used. A brief outline of the process
follows.

The length, M10, of the equation buffer is set initially to
the length required for storing the equations of a single node when the
number of current active nodes is greatest. With this value, the
maximum number of random access records required to transfer all
the completed equations to disc, and the number of load cases which
can be processed by the first solution and back-substitution phases,
are calculated. The re-solution phase increases the number of
random access locations; having determined whether this facility is
required, the calculation is repeated with an increased length of the
equation buffer until the minimum number of records is found by
obtaining the same value for two successive repeats. If the size of
ELPA does not allow it, the length of the equation buffer can be
increased, at the expense of processing fewer load cases in the first
solution and back-substitution phases. There is, however, an escape
clause if NLPZ should prove too small for solution to occur. Although
use of the re-solution phase increases the number of random access
records, it can accommodate more load cases than either of the other
two phases.

Overlay (3, 0) concludes by calculating the labels which define
the manner in which ELPA is subdivided in the pre-solution, solution
and post-solution processes, and determining whether the size of ELPA
permits these to be performed. The computer writes out the number
of locations required in ELPA for each phase of the program. If ELPA
is too small, an error message is written stating the minimum value of
NLPZ for which all phases are possible, and execution is terminated.
4.7 **OVERLAY (4, 0) - PRE-SOLUTION PROCESS**

The unused area at the beginning of ELPA, left by the data processing overlay, is used in the pre-solution process to store the individual element information and the list of current active nodes, Figure 161. The individual element information consists initially of the element number, element type, number of nodes and various other parameters, the geometry, elastic properties, loading data, destination vector, and support conditions for each degree of freedom, stating whether it is free, rigid, or elastically constrained. It also contains, after being transferred to the element stiffness matrix subroutine, the element stiffness matrix and the load vectors, or right hand sides, for all the load cases.

The pre-solution process is straightforward. For each element in turn, searches are made of the various items of data stored in ELPA between the labels NIZZ and NLPZ, and the relevant information is stored in the individual element information vector, as shown in Figure 161. The information is written on tape 2 and then transferred to the element stiffness matrix subroutine with the areas for storing the element stiffness matrix and right hand sides initialised to zero. The element stiffness matrix subroutine calculates the stress matrix for each node of the element and writes them on tape 2. As the number of locations required by a stress matrix is usually less than that required for storing the element stiffness matrix, the space allocated for the latter is used for both purposes. Having calculated the stress matrices, the computer re-initialises the area used to store the stress matrix and calculates the stiffness matrix and element right hand sides. Control is then returned to the calling subroutine to allow the element right hand sides to be modified in accordance with the support conditions, the concentrated loads for those nodal points making their last appearance to be added to the element right hand sides, and for the element record and the stiffness matrix to be written on tape 1.

The pre-solution process ends by writing the combined load case data, stored between labels, NLPFO and NLPLO, to tape 3.
4.8 **OVERLAY (5, 0) - SOLUTION**

This primary overlay loads either the secondary Overlay (5, 1), the Choleski solution, or the secondary Overlay (5, 2), the frontal solution.

4.8.1 **Overlay (5, 1) - Choleski Solution**

Choleski's triangular decomposition method factorizes the overall stiffness matrix, $K$, in the form $K = LU$ where $L$ and $U$ are the lower and upper triangular matrices. As matrix $K$ is symmetrical, it can be arranged that $U = L^T$ and $K = LL^T$ enabling $L$ to be expressed in the general terms

$$
1_{11} = \sqrt{k_{11}} \\
1_{ii} = \sqrt{k_{ii} - \sum_{k=1}^{k=i-1} l_{ik}^2} \\
1_{ij} = \frac{k_{ij} - \sum_{k=1}^{k=j-1} l_{ik} l_{jk}}{l_{jj}}
$$

for $i > j$

These terms show that no term in $K$ is used more than once and that its one use is to obtain the $1$ term in the same position. This allows the storage locations in $K$ to be overwritten by the values of $1$ as each is obtained, and avoids the need for additional storage for $L$ and $L^T$. The difficulty which would be caused by a square root of a negative number does not arise because the matrix $K$ is positive-definite.

The solution process is:

$$
P = K\delta = LL^T\delta = Lx
$$

where $P$ and $\delta$ are the load and displacement vectors, and $x = L^T\delta$. The unknowns $x$ can be solved by forward substitution using the expressions

$$
x_1 = \frac{P_1}{1_{11}} \quad \text{and} \quad x_i = \frac{(P_i - \sum_{j=1}^{j=i-1} x_j 1_{ij})}{1_{ii}} \quad \text{for} \quad i > 1
$$
The values of $x$ are obtained in order and can be overwritten on $P$, as each of the original values stored there is used only once. The back-substitution step $x = L^T \delta$ uses the expressions

$$
\delta_n = x_n / l_{nn} \quad \text{and} \quad \delta_i = (x_i - \sum_{j=i+1}^{n} l_{ii} \delta_j) / l_{ii} \quad \text{for} \quad i < n
$$

where $n$ is the number of unknown displacements. The displacements are found in reverse order and can be overwritten on $x$ as they are calculated.

The subdivision of ELPA for the Choleski method of solution is shown in Fig. 162. The method is used only when the complete overall stiffness matrix can be stored in the central memory of the computer, and at least one load case can be handled by the forward and back-substitution stages. This is usually possible when the number of nodal points and the semi-bandwidth of the overall stiffness matrix are small. The latter is achieved by numbering the nodal points so that the spread of numbers for each element is a minimum.

The subroutine begins by rewinding the element tape, tape 1, and reading the element record and stiffness matrix for each element and adding their respective contributions to the overall stiffness matrix and right hand sides. If re-solution is necessary, the element record is written on tape 4. For some types of element, the element stiffness matrix is in a segmented form in order to conserve locations. Factorisation of the overall stiffness matrix, in which full advantage is made of its symmetrical and banded nature, is followed by the forward and back-substitution phases. Finally, the displacements which have overwritten the overall right hand sides, are written on tape 3.

Re-solution is necessary if all the load cases cannot be processed at the same time. This entails rewinding tape 4 and repeating the process until the displacements for the last load case have been determined. The overall stiffness matrix, however, is not reassembled or factorised.
4.8.2  Overlay (5, 2) - Frontal Solution

The frontal solution of Irons (17) is a computer algorithm for solving a large set of simultaneous equations, the coefficient matrix of which is symmetrical, positive-definite and banded. The method of solution is essentially Gaussian elimination without partial or complete pivoting. The subdivisions of ELPA for the three phases of the frontal solution are shown in Figure 162.

In order to solve the matrix equation $K\delta = P$ by Gauss elimination, the coefficients of row $s$ which constitute equation $e_s$, are used in the expressions

$$K_{ij}^* = K_{ij} - \frac{K_{is}K_{sj}}{K_{ss}} \quad \text{and} \quad P_i^* = P_i - \frac{K_{is}P_s}{K_{ss}}$$

to eliminate $\delta_s$. The elimination reduces matrix $K$ to an upper triangular matrix. The terms $K_{ij}$ of the overall stiffness matrix, which is symmetrical, and $P_i$ of the overall right hand sides, are the sum of the element contributions. By inspection, the expressions show that $\delta_s$ can be eliminated provided $P_s$ and the terms $K_{si} = K_{is}'$, $K_{sj}$ and $K_{ss}$ of equation $e_s$ are fully summed. Consequently, it does not matter in what order the additions and subtractions to $K_{ij}$ and $P_i$, corresponding to element contributions and modifications resulting from use of the above formulae, are made.

If the overall stiffness matrix is banded, several terms $K_{si}$ and $K_{sj}$ are zero. Columns outside the bandwidth and, due to symmetry, their corresponding rows, remain unmodified when $\delta_s$ is eliminated. It is, therefore, prudent to store the square submatrix containing only those $K_{ij}$ which are to be modified. The currently active variables $\delta_a$, with which these are associated, exclude those variables $\delta_p$ which have been eliminated, and those variables $\delta_f$ which have zero coefficients in all the equations $e_s$ so far encountered, so that $K_{sf}$ are still zero. As the square submatrix is symmetrical, only the lower triangular matrix is required in core to effect forward elimination.
The elimination phase for the first solution alternates between assembly of the overall stiffness matrix and right hand sides, and elimination. It is best explained by an example which will also introduce the terms used in Figure 162.

<table>
<thead>
<tr>
<th>Element Number</th>
<th>Element Nodes</th>
<th>Element Destination Vector</th>
<th>Composition of MNODS (Position of nodes in Grandpa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4 5 2 1</td>
<td>1 2 3 4</td>
<td>4 5 2 1*</td>
</tr>
<tr>
<td>2</td>
<td>7 8 5 4</td>
<td>4 5 2 1*</td>
<td>4 5 2 7 8</td>
</tr>
<tr>
<td>3</td>
<td>5 6 3 2</td>
<td>2 1 4 3</td>
<td>6 5 2 3 8</td>
</tr>
<tr>
<td>4</td>
<td>8 9 6 5</td>
<td>5 3 1 2</td>
<td>6 5 9 8</td>
</tr>
</tbody>
</table>

* Nodes eliminated after assembly of element contributions.

The simple structure shown above consists of four elements, each with four nodes and, let us suppose, one degree of freedom per node in order that a node may be considered as a variable to be eliminated. Assuming that the elements were defined in numerical order in the first data processing overlay, the third data processing overlay would have used this order to calculate the dimensions of the various subdivisions of ELPA, and the values of all the frontal solution labels, NB, NPAZ, L2, L3, .... shown in Figure 162.
The phase begins by rewinding tape 1 and reading from it the first element record. The element record and the equation buffer, which is in fact empty, are written immediately on tape 4, and the element stiffness matrix is read from tape 1. The element right hand sides and stiffness matrix are assembled in the current load cases and current overall stiffness matrix, called by Irons, 'grandpa's right hand sides' and 'grandpa's coefficients' respectively. As they are empty, the current active nodes, MNODS, will contain the nodes of the first element in its first four locations, and the destination vector for the element will contain the positions in MNODS of its nodal points. The table shows that node 1 has appeared for the last time since it is not used to define any other element. Its equation in the current overall stiffness matrix must therefore be complete and ready for elimination. As the destination in MNODS of node 1 is 4, the coefficients of row 4 are used to eliminate $\delta_4$ and form, with its associated right hand sides, the complete equation $e_4$ which is stored in the equation buffer EQU. In storing the coefficients, the element stiffness matrix is overwritten, this being possible because it has already been assembled in the current overall stiffness matrix. Finally, row 4 in the overall stiffness matrix and right hand sides is zeroed.

The element record for the second element is read from tape 1, and written on tape 4 with the equation buffer containing the equations associated with the variables eliminated after assembling the first element contributions. The equation buffer is now ready to receive the element stiffness matrix of the second element, which is read from tape 1 and added to the overall stiffness matrix. As nodes 4 and 5 occupy rows 1 and 2, the new nodes 7 and 8 must occupy new rows. Row 4, however, is free since node 1 has already made its last appearance. The overall stiffness matrix now contains five variables. Nodes 4 and 7, however, have appeared for the last time and rows 1 and 4 in the overall stiffness matrix and right hand sides can be eliminated. Node 4 is eliminated first, followed by node 7, with the usual zeroing of rows.
The third element record is read from tape 1 and the customary writing on tape 4, reading and assembly of element right hand sides and stiffness matrix, are performed. The new nodes 6 and 3 are stored in positions 1 and 4 of MNODS. The degrees of freedom, $\delta_2$ and $\delta_3$, of nodes 2 and 3 are eliminated, and the associated coefficients in rows 3 and 4 of the overall stiffness matrix and right hand sides are stored in the equation buffer and then zeroed.

The fourth element introduces node 9, which is stored in row 3 of the overall stiffness matrix. Since no more elements exist, all the nodes can be eliminated and the equations written on tape 4.

For example, the five locations needed in MNODS represent the maximum number of currently active nodes required on core at any one time. The active nodes are 5, 2 and 8 before the third element is introduced, 6, 5, 2, 3 and 8 after assembling the third element, and 6, 5 and 8 after eliminating nodes 2 and 3 prior to the introduction of the last element. The active nodes, or front, change as each new element is introduced, the front dividing the structure into two independent parts, the first consisting of all the eliminated nodes and the second of all the nodes to be activated. The two parts may be considered as substructures linked together by the front at the given moment of time.

The forward elimination sequence:

1) read element record from tape 1; RREC (1-MAXVAR)
2) write element record and equation buffer on tape 4; RREC(1-L1)
3) read element stiffness matrix from tape 1; EQU(1-MAXELT)

is not used by Irons (17). It was introduced when the random access facilities of the computer were found to be an absolute necessity for back-substitution and re-solution. The sequence also saves computer storage by allowing the element stiffness matrix and the equation buffer to occupy the same locations.
Back-substitution commences when all the variables have been eliminated. Tape 4 contains all the eliminated equations which must be processed in the reverse order in which they were created. The action BACKSPACE-READ-BACKSPACE was originally used to read the records in reverse order, but proved to be costly in both peripheral and central processing computer time. The trouble was rooted in the BACKSPACE command; the computer rewound the tape and skipped each record until the correct one was reached, instead of moving back over one record. Random access facilities overcame the problem and Figure 169 justifies their use for the two-dimensional studies reported in Section 5.2.5.

The back-substitution phase begins by zeroing ELPA between the labels NB and NRUNO. It then proceeds to read the records on tape 4 in reverse order and calculate the displacements using the equations

\[ \delta_n = \frac{P_n^*}{K_{nn}^*} \quad \text{and} \quad \delta_i = \left( P_i^* - \sum_{j=i+1}^{j=n} K_{ij}^* \delta_j \right) / K_{ii}^* \quad \text{for } i < n \]

where \( n \) is the total number of unknown displacements. During elimination, each equation was accompanied on tape by the current size of the overall stiffness matrix, a coded form of the variable being eliminated stating its nodal point number and destination in the solution vector, and the destination of the variable in the vector of running displacements. In back-substitution this information is recovered, and as each variable is calculated it is placed in the running displacement vector and the solution vector. The composition of the running displacement vector reflects the composition of the current overall stiffness matrix at the corresponding time during the elimination phase. The summation in the above equation is therefore achieved simply by temporarily setting to zero the equation coefficient corresponding to the leading diagonal term of the overall stiffness matrix, and taking a scalar product of the equation coefficients and the running displacements.
When all the records on tape 4 have been read in reverse order and all the displacements calculated and stored in the solution vector, the back-substitution phase ends by writing the solution vector for each load case on tape 3.

Re-solution is required when the elimination phase for the first solution or the back-substitution phase cannot accept all the load cases supplied by the user. The program is designed to re-solve without repeating the assembly and reduction of the overall stiffness matrix. As the coefficients required by the elimination formulae can be retrieved from the existing equations on tape 4, the locations used by the current overall stiffness matrix are released to allow the phase to accept many more right hand sides than in the original solution. In re-solution, tape 1 containing the element records and stiffness matrices is ignored, and the element records on tape 4 are used to update the right hand sides of the equations.

Two other facilities suggested by Irons are included in the program. The first concerns very large elements like the 32 node isoparametric solid element which requires 4656 locations to store the element stiffness matrix. To introduce the element stiffness matrix as a long vector in ELPA during the solution stage would waste valuable core storage. In Overlay (3, 0), a test is therefore made to see if the core storage allocated by the user requires the vector to be broken into shorter records, when it is written on tape 1 in the pre-solution problem. The length of these records, MAXELT, is determined automatically by the computer.

The second facility is the refined diagonal decay criterion of roundoff damage used by Irons. During the first solution, the values through which each leading diagonal term passes are squared and accumulated into an extra overall right hand side. The criterion is the ratio of the square root of this accumulation to the diagonal's final value. If it is very large, the structure is either a mechanism or inadequately restrained, and the solution is terminated. If it is
large, or the pivot negative, a warning is printed but solution continues. The warning indicates to the user which nodal variable caused the trouble, and perhaps, helps him to correct it.

4.9 OVERLAY (6, 0) - POST-SOLUTION PROCESS

The post-solution process calculates the displacements for the combined load cases, the stresses for each element, the reactions to earth and, if requested, the average nodal point stresses. Figure 163 shows the two ways in which ELPA is subdivided during the process.

The first subdivision is used to manipulate and extend tape 3 so that it not only contains the combined load case data and the displacement vector for each load case, but also the displacement vector for each combined load case.

The second subdivision is used to calculate the stresses and reactions to earth. For each displacement vector on tape 3, the element stress tape, tape 2, is rewound, and the reactions to earth and average stress vectors are set to zero. For each element in turn, the individual element information record is read from tape 2, the displacements for each of its nodes are located in RHS and stored in DISP, and if the node is a-support node and is appearing for the first time, its number of appearances is stored in MNODS, its nodal point number in NREACT, and its reactions calculated and stored in REACT.

Execution is then transferred to the stress matrix subroutine for the type of structure concerned. Here, the stress matrix for each node of the element, DB, is recovered from tape 2 and used to calculate the nodal point stresses. The stresses are stored in STRESS and are accumulated into AVSTR if the average nodal point stress option has been used. If the dummy overlay option has been specified, the element nodal points, coordinates, displacements, stresses and various parameters, are written on tape 7. Further, if the element output has not been suppressed, the stresses are printed. When, however, several types
of element are used in the same problem and output by elements is required, tapes 1 and 3 are used alternately to store information, so that all elements of the same type have their results printed together. On return to the calling subroutine, the average nodal point stresses are calculated for those nodes of the element making their last appearance by locating their number of appearances in MNODS.

The overlay concludes by printing the displacement vector, the average nodal point stresses and the reactions to earth.

4.10 OVERLAY (7, 0) - DUMMY OVERLAY

The use of the dummy overlay option allows the user to write his own overlay program in which he can recover the element records and stiffness matrices on tape 1, the individual element information and stress matrices on tape 2, the displacement vectors on tape 3, the element records, and for the frontal solution only, the eliminated equations, on tape 4, and the coordinates, displacements and stresses for each element on tape 7. He can, therefore, process the results according to his own wishes.

For the theoretical work of Chapter 5, this facility was used to recover the element information on tape 7, to transform the global stresses and displacements of the upstream and downstream surfaces of the dam into local normal, radial and tangential stresses and displacements, to calculate the principal stresses on the surfaces, and to present the results in easily recognizable groups.

Its use, however, is far less restrictive than it might appear from this example. It could be used to extend the problem into a non-linear state by updating the element records and stiffness matrices on tape 2, and re-executing the solution and post-solution processes.
4.11 SUMMARY

A computer program has been written as an attempt to lay the foundation for a general purpose system for the Civil Engineering Department of Imperial College. Although it has been programmed for the CDC 6400/6600/7600 computers and needs a greater variety of elements, it is versatile enough for such deficiencies to be overcome without too much effort.

Solution is by the frontal technique, for which the order of the elements is relevant and the nodal point numbering is not, or the Choleski method when the vector ELPA is large enough to allow the overall stiffness matrix to be stored in the central memory of the computer. While the frontal solution has proved to be very useful in solving the three-dimensional arch dam and valley problem reported in the next chapter, its advantages, if any, over a good band algorithm using backing storage, for which nodal point numbering is relevant and the element order is not, are still in doubt.

Several options have been incorporated into the system to give the user some control of the program. The most powerful of these is the dummy overlay facility which enables him to change his problem from linear elastic to a non-linear state.
CHAPTER 5: THEORETICAL STUDY

5.1 INTRODUCTION

This study applied the finite element program described in the previous chapter to two types of analysis of the El Atazar Dam.

The first type was a two-dimensional investigation concerned with determining the extent to which it is necessary to include the foundation of the dam in order to achieve no significant alterations in the stress distribution immediately below the dam.

The second type of analysis consisted of a three-dimensional analysis of the dam and its valley using various elements in the library of three-dimensional elements, Figures 164 - 165. Three main analyses have been made, each with a variety of loading conditions. The first analysis was of the shell of the dam rigidly supported at its periphery, the second was of the shell and socket supported by a rigid valley, and the third was of the dam supported on an idealised elastic foundation in which some of the major faults had been represented. The third analysis was undertaken in order to study the influence of the faults on the stability of the dam. It consisted of examining the normal and shear stresses in those finite elements modelling the faults, and then assessing the likelihood of a sliding failure.

As the studies were made assuming linear elasticity and no adjustments were made for the opening and closing of joints due to the development and redistribution of tensile stresses, they must all be considered as a first approximation only.

5.2 TWO-DIMENSIONAL ANALYSIS

One of the most significant developments in the numerical solution of solid structures was the introduction of isoparametric finite elements, many of which have a high degree of accuracy\(^{(28)}\). A modification of this approach which results in a further improvement in accuracy has been presented by Wilson\(^{(29)}\). It is with this modification that the QDMI4 element used in this two-dimensional study is associated.
The method introduces incompatible displacement modes at the element level in order to improve the element accuracy. These unknowns are eliminated by requiring that the total strain energy within the element is at a minimum. Convergence of the solution is thus assured.

5.2.1 Source of Errors in the QDM4 Element

It is well known that one of the main causes of inaccuracies in lower order finite elements is due to their inability to represent certain simple stress gradients. Such elements are generally known as being 'too stiff'. The phenomenon is clearly illustrated by subjecting a rectangular QDM4 element to a pure bending stress. The exact displacements for this type of loading are illustrated in Figure 170 and are given by:

\[
\begin{align*}
  u &= \alpha_1 xy \\
  v &= \frac{\alpha_1}{2} (a^2 - x^2) + \alpha_2 (b^2 - y^2)
\end{align*}
\]

For the simple rectangular element, the only displacement activated by this type of loading is

\[
  u = \beta_1 xy
\]

The form of the error in the solution is therefore

\[
  v = \beta_2 (a^2 - x^2) + \beta_3 (b^2 - y^2)
\]

Attempts to reduce these errors have involved selecting integration formulae which disregard the shear strain energy. However, this technique can produce a stiffness matrix which has directional properties. The approach adopted by Wilson to minimize the errors was to add extra displacement modes to the elements which have the same form as the errors in the simple displacement approximation, and which generally violate inter-element compatibility. The magnitudes of the modes are selected by requiring that the total strain energy of the element be at a minimum.
5.2.2 Derivation of the Two-Dimensional QDM4 Element

For the simple quadrilateral element QDM4 shown in Figure 166, the local and global coordinate systems are related by

\[ \begin{align*}
    x &= N_1 x_1 + N_2 x_2 + N_3 x_3 + N_4 x_4 \\
    y &= N_1 y_1 + N_2 y_2 + N_3 y_3 + N_4 y_4 
\end{align*} \]

where the shape functions \( N_i \) are given in terms of the local coordinate system:

\[ N_i = \frac{1}{4} \left( 1 + \xi_i \right) \left( 1 + \eta_i \right) \]

where \( \xi_i \), \( \eta_i \) are the local coordinates of nodal point \( i \). In isoparametric elements, the same shape functions are used in the displacement approximation in order to ensure rigid body displacement modes:

\[ \begin{align*}
    u &= N_1 u_1 + N_2 u_2 + N_3 u_3 + N_4 u_4 \\
    v &= N_1 v_1 + N_2 v_2 + N_3 v_3 + N_4 v_4 
\end{align*} \]

In determining the element stiffness and stress matrices, volume integrals involving the shape functions arise. The element stiffness matrix can be written as

\[ K_e = \int_{\text{volume}} B^T DB \, dV \]

where the strain matrix \( B \) involves appropriate derivatives of the displacements with respect to the coordinate directions since it is defined as

\[ \xi = B \delta \]

where \( \xi \) are the strains and \( \delta \) the global displacements of the element. In two-dimensional analysis, since the strain-displacement equations are

\[ \begin{align*}
    \epsilon_{xx} &= \frac{1}{2} \left( \frac{\partial^2 u}{\partial x^2} + \frac{\partial^2 v}{\partial y \partial x} \right) \\
    \epsilon_{yy} &= \frac{1}{2} \left( \frac{\partial^2 v}{\partial y^2} + \frac{\partial^2 u}{\partial x \partial y} \right) \\
    \gamma_{xy} &= \frac{1}{2} \left( \frac{\partial^2 u}{\partial y \partial x} + \frac{\partial^2 v}{\partial x \partial y} \right) 
\end{align*} \]
\[
\begin{align*}
\mathbf{e} = \begin{bmatrix}
\varepsilon_x \\
\varepsilon_y \\
\varepsilon_z 
\end{bmatrix} = \begin{bmatrix}
\frac{\partial u}{\partial x} \\
\frac{\partial v}{\partial y} \\
\frac{\partial u}{\partial y} + \frac{\partial v}{\partial x}
\end{bmatrix} 
\end{align*}
\]

It is obvious that the chain rule of partial differentiation must be used to express global derivatives in terms of local derivatives:

\[
\begin{bmatrix}
\frac{\partial}{\partial \xi} \\
\frac{\partial}{\partial \eta}
\end{bmatrix} = \begin{bmatrix}
\frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} \\
\frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta}
\end{bmatrix} \begin{bmatrix}
\frac{\partial}{\partial x} \\
\frac{\partial}{\partial y}
\end{bmatrix} = J \begin{bmatrix}
\frac{\partial}{\partial x} \\
\frac{\partial}{\partial y}
\end{bmatrix}
\]

The Jacobean matrix \( J \), which is evaluated by using equation (1),

\[
J = \begin{bmatrix}
\frac{\partial x}{\partial \xi} & \frac{\partial y}{\partial \xi} \\
\frac{\partial x}{\partial \eta} & \frac{\partial y}{\partial \eta}
\end{bmatrix} = \begin{bmatrix}
\frac{\partial N_1}{\partial \xi} & \frac{\partial N_2}{\partial \xi} & \frac{\partial N_3}{\partial \xi} & \frac{\partial N_4}{\partial \xi} \\
\frac{\partial N_1}{\partial \eta} & \frac{\partial N_2}{\partial \eta} & \frac{\partial N_3}{\partial \eta} & \frac{\partial N_4}{\partial \eta}
\end{bmatrix} \begin{bmatrix}
x_1 & y_1 \\
x_2 & y_2 \\
x_3 & y_3 \\
x_4 & y_4
\end{bmatrix}
\]

is inverted to obtain the global derivatives of the shape functions \( N_i \)

\[
\begin{bmatrix}
\frac{\partial N_1}{\partial x} \\
\frac{\partial N_1}{\partial y}
\end{bmatrix} = J^{-1} \begin{bmatrix}
\frac{\partial N_1}{\partial \xi} \\
\frac{\partial N_1}{\partial \eta}
\end{bmatrix} = \begin{bmatrix}
J_{11} & J_{12} \\
J_{21} & J_{22}
\end{bmatrix} \begin{bmatrix}
\frac{\partial N_1}{\partial \xi} \\
\frac{\partial N_1}{\partial \eta}
\end{bmatrix} = \begin{bmatrix}
J_{11} \frac{\partial N_1}{\partial \xi} + J_{12} \frac{\partial N_1}{\partial \eta} \\
J_{21} \frac{\partial N_1}{\partial \xi} + J_{22} \frac{\partial N_1}{\partial \eta}
\end{bmatrix} = \begin{bmatrix}
a_1 \\
b_1
\end{bmatrix}
\]

From equation (2), the global derivatives of the displacement \( u \) are

\[
\begin{bmatrix}
\frac{\partial u}{\partial x} \\
\frac{\partial u}{\partial y}
\end{bmatrix} = \begin{bmatrix}
\frac{\partial N_1}{\partial x} & \frac{\partial N_2}{\partial x} & \frac{\partial N_3}{\partial x} & \frac{\partial N_4}{\partial x} \\
\frac{\partial N_1}{\partial y} & \frac{\partial N_2}{\partial y} & \frac{\partial N_3}{\partial y} & \frac{\partial N_4}{\partial y}
\end{bmatrix} \begin{bmatrix}
u_1 \\
u_2 \\
u_3 \\
u_4
\end{bmatrix} = \begin{bmatrix}
a_1 & a_2 & a_3 & a_4 \\
b_1 & b_2 & b_3 & b_4
\end{bmatrix} \begin{bmatrix}
u_1 \\
u_2 \\
u_3 \\
u_4
\end{bmatrix}
\]
Similarly, the global derivatives of the displacement $v$ are

$$\begin{bmatrix}
\frac{\partial v}{\partial x}
\frac{\partial v}{\partial y}
\end{bmatrix} = \begin{bmatrix}
a_1 & a_2 & a_3 & a_4
b_1 & b_2 & b_3 & b_4
\end{bmatrix} \begin{bmatrix}
v_1 
v_2 
v_3 
v_4
\end{bmatrix}$$

In substituting these global derivatives in equation (5), it is evident that equation (4) becomes

$$\mathbf{\varepsilon} = \mathbf{B}\delta = \begin{bmatrix}
B_1 & B_2 & B_3 & B_4
\end{bmatrix} \begin{bmatrix}
\delta_1 
\delta_2 
\delta_3 
\delta_4
\end{bmatrix}$$

where the submatrix $B_i$ of the strain matrix $B$, and the displacements of node $i$ are

$$B_i = \begin{bmatrix}
a_i & 0 
0 & b_i 
b_i & a_i
\end{bmatrix} \quad \text{and} \quad \delta_i = \begin{bmatrix}
u_i 
v_i
\end{bmatrix}$$

The elasticity matrix $D$ for a two-dimensional element is

$$D = \begin{bmatrix}
d_{11} & d_{12} & d_{13} 
d_{21} & d_{22} & d_{23} 
d_{31} & d_{32} & d_{33}
\end{bmatrix} = \begin{bmatrix}
d_{11} & d_{12} & 0 
d_{21} & d_{22} & 0 
0 & 0 & d_{33}
\end{bmatrix}$$

for isotropy.

The stresses and strains are related by the equation

$$\mathbf{\sigma} = \mathbf{D}\varepsilon = \mathbf{D}\mathbf{B}\delta = \mathbf{S}\delta$$

and $S$ is known as the stress matrix. The matrix $S$ can be subdivided
so that advantage can be taken of the sparse nature of $B$ and $D$.

Thus,

$$S = \begin{bmatrix} S_1 & S_2 & S_3 & S_4 \end{bmatrix}$$

where

$$S_j = DB_j = \begin{bmatrix} d_{11} & d_{12} & d_{13} \\ d_{21} & d_{22} & d_{23} \\ d_{31} & d_{32} & d_{33} \end{bmatrix} \begin{bmatrix} a_j & 0 \\ 0 & b_j \\ b_j & a_j \end{bmatrix}$$

or

$$\begin{bmatrix} S_{11} & S_{12} \\ S_{21} & S_{22} \\ S_{31} & S_{32} \end{bmatrix} = \begin{bmatrix} d_{11}a_j + d_{13}b_j, & d_{12}b_j + d_{13}a_j \\ d_{21}a_j + d_{23}b_j, & d_{22}b_j + d_{23}a_j \\ d_{31}a_j + d_{33}b_j, & d_{23}b_j + d_{33}a_j \end{bmatrix} = \begin{bmatrix} d_{11}a_j, & d_{12}b_j \\ d_{21}a_j, & d_{22}b_j \\ d_{33}b_j, & d_{33}a_j \end{bmatrix}$$

for isotropy.

In determining the stiffness matrix, the integration is best carried out in terms of the local coordinates. If the element thickness is $t$, equation (3) can be written as

$$K^e = \int \int \int B^T DB |J| t \, d\xi \, d\eta$$

and in submatrix form as

$$K^e_{ij} = \int \int \int B^T_i DB_j |J| t \, d\xi \, d\eta$$

$$= \int \int \int B^T_i S_j |J| t \, d\xi \, d\eta$$

$$= \int \int \left[ a_i S_{11} + b_i S_{31}, a_i S_{12} + b_i S_{32} \right] |J| t \, d\xi \, d\eta$$
Although many alternative integration formulae are available, the direct application of the one-dimensional Gauss quadrature rules are generally used to yield

\[ K^E = \sum_m \sum_n H_m H_n G(\xi_m, \eta_n) \]

where \( G(\xi_m, \eta_n) = B^T DB |J| t \)

is evaluated at the Gauss sampling points \((\xi_m, \eta_n)\) and \(H_m, H_n\) are the corresponding weight coefficients at this point.

5.2.3 Addition of Incompatible Modes to QDM4 Element to form QDMP4 Element

The basic method used to derive the stiffness matrix for a QDM4 element is also used when internal degrees of freedom are added at the element level. The displacements for the QDMP4 element take the following form:

\[
\begin{align*}
  u &= N_1 u_1 + N_2 u_2 + N_3 u_3 + N_4 u_4 + \alpha_1 N_5 + \alpha_2 N_6 \\
  v &= N_1 v_1 + N_2 v_2 + N_3 v_3 + N_4 v_4 + \beta_1 N_5 + \beta_2 N_6
\end{align*}
\]

The shape functions \(N_5\) and \(N_6\) must be zero at the four nodes and, since their amplitudes \(\alpha_i\) and \(\beta_i\) are additional degrees of freedom, the resulting stiffness matrix will be 12x12. By minimising the strain energy within the element with respect to \(\alpha_i\) and \(\beta_i\), four additional equations can be generated to enable the four extra degrees of freedom to be eliminated and an 8x8 stiffness matrix developed. The procedure is equivalent to the static condensation procedure.

The extra shape functions \(N_5\) and \(N_6\) are selected to be of the same form as the bending errors given in Section 5.2.1. They are given by

\[
\begin{align*}
  N_5 &= (1 - \xi^2) \\
  N_6 &= (1 - \eta^2)
\end{align*}
\]

and the corresponding incompatible modes are shown in Figure 170.
5.2.4. Static Condensation

The static condensation method is general: it can be used to reduce the number of degrees of freedom at the element stiffness matrix level or the overall stiffness matrix level. In many cases, it is similar to the substructure technique and to the frontal solution method described in Section 4.8.2, both of which appear to be an application of Gauss elimination.

By eliminating unwanted degrees of freedom from the element stiffness matrix, the size and bandwidth of the overall stiffness matrix can be reduced. If this is done, the procedure is best extended to the condensation of the stress matrices so that the calculation of the eliminated degrees of freedom will not be required in the subsequent evaluation of the element stresses.

The static condensation procedure can be written in matrix form as

\[
\begin{bmatrix}
K_{rr} & K_{re} \\
K_{er} & K_{ee}
\end{bmatrix}
\begin{bmatrix}
\delta_r \\
\delta_e
\end{bmatrix}
= 
\begin{bmatrix}
P_r \\
P_e
\end{bmatrix}
\]

where \( \delta_e \) represent the degrees of freedom to be eliminated and \( \delta_r \) are the degrees of freedom associated with the reduced stiffness matrix. Solving the second submatrix equation for \( \delta_e \) yields

\[ \delta_e = R - \delta_r \]

in which

\[ R = K_{ee}^{-1} P_e \]
\[ C = K_{ee}^{-1} K_{er} \]

Substituting for \( \delta_e \) in the first submatrix equation gives

\[ K^* \delta_r = P^* \]

where

\[ K^* = K_{rr} - K_{re} C \]
\[ P^* = P_r - K_{re} R \]
and the term $K_{re} C$ represents the modification in stiffness due to the release of the degrees of freedom $\delta_e$, and $K_{re} R$ represents the forces carried over from the $\delta_e$ to the $\delta_r$ degrees of freedom.

The stresses in the element may be written as

$$\sigma = \begin{bmatrix} S_r & S_e \end{bmatrix} \begin{bmatrix} \delta_r \\ \delta_e \end{bmatrix} + \sigma_o$$

where $\sigma_o$ are the initial stresses. Substituting for $\delta_e$ results in

$$\sigma = S^* \delta_r + \sigma_o$$

where $S^* = S_r - S_e C$

$$\sigma_o^* = \sigma_o + S_e R$$

Although the matrix equations illustrate static condensation mathematically, the inversion and multiplication of matrices within a computer program is generally inefficient. The degrees of freedom $\delta_e$ can be eliminated, however, by Gauss elimination. In the computer program, in the subroutine for calculating the stiffness matrix and stress matrices for a QDM14 element, the coefficients to be eliminated are stored after the coefficients which are to be modified. This saves a rearrangement of the reduced coefficients as the 12x12 matrix is reduced to 8x8. The last four rows of the matrix are eliminated in reverse order.

Solving for $\delta_n$ gives

$$\delta_n = R_n - \sum_{j=1}^{n-1} C_{nj} \delta_j$$

where

$$C_{nj} = \frac{K_{nj}^*}{K_{nn}}$$

and

$$R_n = \frac{p_n^*}{K_{nn}}$$
On eliminating $\delta_n$, the remaining equations become

$$K_{ij}^{**} = K_{ij}^* - K_{in}^* C_{nj}$$

$$P_i^{**} = P_i^* - K_{in}^* R_n$$

and

$$S_{ij}^{**} = S_{ij}^* - S_{in}^* C_{nj}$$

$$\sigma_{oi}^{**} = \sigma_{oi}^* + S_{in}^* R_n$$

where $m$ is the number of stress components and * indicates repeated modification of the coefficient for each value of $n$.

While the above is general, the equivalent nodal point forces for the QDMI4 element are restricted to the four corner nodes by using only the first four shape functions to define the distributed loads acting on the element. As $P_e = 0$, therefore $P_n = R_n = 0$ and the load vector $P_r$ and the initial stresses $\sigma_o$ remain unchanged in eliminating the four extra degrees of freedom.

To save storage, in the computer program the 12x12 element stiffness matrix is calculated and, as each degree of freedom is eliminated, the values of $C_{nj}$ are stored; the reduced stiffness matrix is then saved on disc. The 3x12 stress matrix for each corner of the element is calculated in turn. Each is stored in the locations used to calculate the stiffness matrix and reduced to a 3x8 matrix using the stored coefficients $C_{nj}$. The reduced form of the matrix is saved on disc.

5.2.5 Analysis of the Dam

In order to study the interaction between the dam and its foundation, the dam was represented by the cross-section of the crown cantilever and by plane stress elements, and the foundation by plane strain elements. Using the idealisations shown in Figure 171, the dam was first analysed assuming a rigid foundation. It was then analysed with its foundation, the size of which was gradually increased from an $MxN$
mesh of 4x4 to a mesh of 16x20.

Although the dam was subjected to a variety of load cases, only the results for combined gravity and hydrostatic load are presented in Figures 172-173. The shear and vertical stresses in Figure 172 show that only the stresses in the lower third of the dam are affected when the foundation, or valley is included in the analysis. Further, a comparison of the results for the 4x4 and 16x20 meshes suggests that the effect of foundation deformability can be estimated by analysing only a relatively small part of the foundation with the dam. This may not, however, be generally true: in the analysis, the dam and foundation had the same elastic modulus, and the foundation was homogeneous.

It is evident that the stresses at level 750 are erroneous. The shear force and bending moment at the section are statically invariant, yet the analysis shows a change with the inclusion of the elastic foundation. This indicates errors in the finite element method probably related to the idealisation and sudden change in geometry at the base of the dam, and to the type of element used.

The rotation of the base of the dam which occurs when the dam is analysed with its foundation accounts for the pronounced increase in the crest displacement, Figure 173. The downstream movement at the base is much smaller.

If the interface between the dam and valley is considered as a discontinuity, it appears from this study that only a relatively small part of the valley beyond any discontinuity need be included in an analysis designed to investigate the interaction between a dam, its valley, and any major geological features represented in the valley.

As a secondary study, the analyses were used to examine the use of the random access facilities on the CDC 6400 computer for the back-substitution phase of the frontal solution technique, Section 4.8.2. The results show that their use is fully justified, Figure 169.
5.3 **THREE-DIMENSIONAL ANALYSIS**

Although the results of this analysis are presented in a logical sequence, they were not undertaken in the same systematic way.

The shell of the dam, rigidly supported at its periphery, was first analysed using HEX20 elements, the standard quadratic isoparametric brick element. To confirm the results, a second analysis was made using HEX32 elements and agreement was achieved. Both idealisations were then extended to include the socket on which the dam was founded, and analysed assuming the socket to be supported by a rigid valley. Once again, the results agreed. Bates\(^{(19)}\) also confirmed the results using the QDS12 curved, super-parametric element for analysing thick and thin shells.

At this stage the HEX32 analysis was abandoned, and the HEX20 idealisation was extended further to include the valley and some of its major faults. This necessitated expanding the joint element of Mahtab and Goodman\(^{(16)}\) from its PLJ8 form to the PLJ16 form containing mid-side nodes. However, it was found that these elements, in conjunction with the idealisation shown in Figure 184, could not be used because a computer with a working area of approximately 120K locations was required by forward elimination in the frontal solution; the computers available were too small. Fortunately, Wilson's work on introducing incompatible displacement modes was discovered. This solved the difficulty by eliminating the need for mid-side nodes and reducing the required working area to 29K locations.

The analyses of the dam and socket were repeated using the 8 node brick element, HEX8, and its modified form containing incompatible modes, HEXI8. Finally, a variety of load cases were undertaken for the dam and its valley using the original idealisation.
5.3.1. Incompatible Modes for Three-Dimensional Brick Elements

The method described in Section 5.2.3. for introducing incompatible modes in order to improve the bending properties can also be used in three-dimensions. The arbitrary 8 node brick element HEX8 was improved by adding the incompatible shape functions

\[ N_9 = (1 - \xi^2) \]
\[ N_{10} = (1 - \eta^2) \]
\[ N_{11} = (1 - \zeta^2) \]

to form the HEXI8 element. Similarly, the 6 node wedge element PEN6 was improved by adding

\[ N_7 = 4(L_1 L_2 + L_2 L_3 + L_3 L_1) \]
\[ N_8 = (1 - \zeta^2) \]

to form the PENI6 element.

5.3.2 Joint Elements

The quadrilateral joint element developed by Mahtab and Goodman\(^{(16)}\) is represented by two coincident plane faces whose relative behaviour is determined by two unit shear stiffnesses, \(k_x\) and \(k_y\) in the \(x'\) and \(y'\) directions, and a unit normal stiffness \(k_z\). The unit shear stiffnesses \(k_x\) and \(k_y\) are defined as the forces per unit area required to produce unit displacements in the directions corresponding to the local axes \(x'\) and \(y'\) respectively. The unit normal stiffness is defined similarly. The right handed set of local coordinate axes \(x', y', z'\) are chosen so that \(x'\) and \(y'\) are consistent with the dip and strike used in the geological description of the fault.

Since the joint element is plane, the relative displacements between the top and bottom surfaces are a function only of the in-plane coordinates \(x', y'\). Assuming a linear variation of displacement across
the opposite faces, the relative displacements between the two surfaces are given by:

\[ f = N \delta' \]

in which

\[ N = \{-B_1 - B_2 - B_3 - B_4 B_1 B_2 B_3 B_4 \} \]

\[ \delta' = \{ \delta'_1 \delta'_2 \delta'_3 \delta'_4 \delta'_5 \delta'_6 \delta'_7 \delta'_8 \} \]

and

\[ B_i = \begin{bmatrix} N_i & 0 & 0 \\ 0 & N_i & 0 \\ 0 & 0 & N_i \end{bmatrix} \quad \text{and} \quad \delta'_i = \begin{bmatrix} u'_i \\ v'_i \\ w'_i \end{bmatrix} \]

The shape function \( N_i \) is expressed in terms of the natural coordinates \((\xi_i, \eta_i)\) of the corner node \( i \),

\[ N_i = \frac{1}{4} (1 + \xi_i \xi_i) (1 + \eta_i \eta_i) \]

and \( \delta'_5 \) to \( \delta'_8 \) and \( \delta'_1 \) to \( \delta'_4 \) represent the displacements of the top and bottom corner nodal points defining the quadrilateral element.

The stresses and displacements are related only by the unit shear and normal stiffnesses. Thus,

\[ p = k'f \]

where

\[ k' = \begin{bmatrix} k'_x & 0 & 0 \\ 0 & k'_y & 0 \\ 0 & 0 & k'_z \end{bmatrix} \]

The potential energy stored in the joint element is given by

\[ \phi = \int \int_{\frac{1}{2}} f^T p \, dx'. \, dy' \]

which, on substituting for \( p \) and \( f \), becomes

\[ \phi = \int \int_{\frac{1}{2}} \delta'^T N'^T k'N \delta' \, dx'. \, dy' \]
Following standard procedure, the potential energy is minimised with respect to the nodal displacements. Thus,
\[ \frac{\partial \phi}{\partial \delta'} = \int \int N^T k' N \, dx', dy' \cdot \delta' = 0 \]
and the stiffness matrix is obtained as
\[ K' = \int \int N^T k' N \, dx', dy' \]
As \( N \) is in terms of the natural coordinates, the integral is more easily evaluated by a change of variables. If
\[ \begin{align*}
x' &= N_1 x_1' + N_2 x_2' + N_3 x_3' + N_4 x_4' \\
y' &= N_1 y_1' + N_2 y_2' + N_3 y_3' + N_4 y_4'
\end{align*} \]
and
\[ J = \begin{bmatrix} \frac{\partial x'}{\partial \xi} & \frac{\partial y'}{\partial \xi} \\
\frac{\partial x'}{\partial \eta} & \frac{\partial y'}{\partial \eta} \end{bmatrix} \]
then
\[ dx' \cdot dy' = |J| \, d\xi \cdot d\eta \]
and the integral for the stiffness matrix can be expressed as
\[ K = \int \int N^T k' N \, |J| \, d\xi \cdot d\eta \]
and evaluated using Gaussian quadrature.

The contribution of the joint element to the overall stiffness matrix is obtained by transformation from the local to the global coordinate system. It can be shown that the stiffness of the element in global coordinates is given by
\[ K = \int \int N^T k N \, |J| \, d\xi \cdot d\eta \]
where
\[ k = \lambda^T k' \lambda \]
and \( \lambda \) is the direction cosine matrix of the angles formed between the
local and global sets of axes.

It is apparent from the definition of $k'$ that the shear and normal stiffnesses of the joint element are independent. This is not the case if joints are represented by continuum elements whose thickness is small in comparison with their lateral extent. Although terms could be included in $k'$ to account for joint dilatation during shear, this, and other non-linear behaviour is probably best modelled using a combined initial stress and strain approach\(^{(30)}\).

5.3.3 Analysis of the Shell of the Dam

In this analysis, advantage was taken of the symmetry of the shell, and only half of the shell was studied. The shell was considered to be rigidly supported at its periphery. It was analysed for gravity and hydrostatic loading, and their combined effects. Gravity load was assumed to act on the completed dam as a whole, rather than on individual monoliths which is the alternative analytical assumption. The effect of the assumption used is to transfer gravity load from the crown cantilever to the sides of the valley by vertical arching.

In the first case, the shell was divided into 21 HEX20 elements as shown in the radial development of the dam, Figure 174. At the boundary, some of the elements had two faces delineating the periphery, consequently creating wedge-shaped elements. All six stress components at the nodes along the intersection of each pair of faces, however, were zero. This was probably because 13 of the 20 nodes defining the element were on the periphery and had zero displacements to comply with the rigid support conditions: these zero displacements, in conjunction with the shape of the element producing a sparse stress matrix, produced the singular stresses. To overcome these problems, the wedge-shaped PEN15 element, a degenerate form of the HEX20 element, was programmed.
In the interests of economy, the analyses were repeated using a different quadrature rule each time to calculate the element stiffness matrices. The 14 and 27 point rules suggested by Irons\(^{(23)}\), and the 3x3x3 and 4x4x4 Gauss quadrature rules, were utilised. No significant differences were found in the results. As expected, however, the time taken to calculate the element stiffness matrices was found to be proportional to the number of integration points. It was therefore decided that in any future analysis using HEX20 and PEN15 elements, the 14 point integration rule would be employed.

To confirm the results obtained using the HEX20 and PEN15 elements, the shell was analysed using 6 HEX32 elements. The analysis was repeated using the same integration rules to calculate the element stiffness matrices. The results this time showed that the 14 point rule was unsuitable, and that the 27 point rule was marginally better than the 3x3x3 Gauss rule when compared with the 4x4x4 Gauss rule.

Stresses were calculated at the nodal points defining the element. Clough\(^{(31)}\) stated that, for elements with nodes along the edges, the position of all nodes is arbitrary in the global coordinate system provided the topology of the cube is retained. Initially, the mid-side nodes of the HEX20 elements were not placed at the mid-points of the edges. The resulting nodal point stresses were found to be erroneous, as were the displacements, but not to the same degree. In view of this, the coordinates of the mid-side nodes were re-calculated as the mid-point of the appropriate edge of the element. The improvement in the stresses was quite remarkable. A similar modification was made for the HEX32 idealisation and the improvement was even more striking. Not all nodes, therefore, can be placed arbitrarily.

The stresses due to combined gravity and hydrostatic loading are summarised in Figure 175. Of particular interest are the tensions predicted on the upstream face, especially along the dam and valley interface. The importance of these stresses will be discussed later.
5.3.4 Analysis of the Shell and Socket

Before analysing the complete dam supported on a realistically modelled elastic foundation, the shell and socket of the dam were analysed assuming the socket to be supported by a rigid valley. The division of the dam and socket into 56 elements is shown in Figure 174.

The original analysis used HEX20 and PEN15 elements, but when it was extended to include the valley, solution of the overall stiffness matrix by the frontal technique was found to be impossible due to insufficient core storage on all the available computers. Fortunately, the problem was cured by the addition of incompatible displacement modes to the HEX8 and PEN6 elements to form the HEXI8 and PENI6 elements. As a result, this analysis became a comparison of the ability of the HEX8, HEXI8 and HEX20 elements, in conjunction with their associated wedge element, to predict the behaviour of the dam under gravity and hydrostatic load. An analysis was also made using 24 HEX32 elements, but as the results generally agreed with those obtained using the HEX20 and PEN15 elements, they have not been presented.

The results of the analyses are presented in Figures 176-183. Under gravity loading, the displacements and stresses in the dam are so small that the differences between the three types of analysis are generally insignificant, Figures 176-177. The results, however, do show that the hoop stresses throughout the dam are small, especially in the lower regions of the dam where it is thick, Figure 176. Further, both the hoop and vertical stresses suggest that there is no significant bending action in either the horizontal or vertical direction. Of particular interest is the fact that no notable tensile stresses are predicted, although they might have been expected at the base of the crown cantilever. The radial displacements in Figure 177 show that, while the upper half of the dam moves downstream, the lower half moves upstream. This is
not entirely unexpected; the phenomenon can be predicted by dividing the cross-section of the crown cantilever in Figure 171 in half, estimating the two centres of gravity, and choosing the directions in which each half would overturn. Because the lower half of the dam is much thicker, and therefore stiffer, its upstream movement will be overshadowed by the downstream movement of the upper half, so that the net movement at the crest will be downstream.

Hydrostatic load was applied to the dam as a body force potential, assuming a linear pore pressure gradient through the thickness of the dam, varying from the full water pressure on the upstream face to zero on the downstream face. The stresses and displacements due to hydrostatic load, given in Figures 178-179, therefore include the effects of uplift.

Both the stresses and displacements obtained using the HEXI8 and PENI6 elements show good agreement with those obtained using HEX20 and PEN15 elements. This is quite remarkable because the latter types of element, with their mid-side nodes, were able to model the curvature of the dam; the former types could not, because of their imposed linear geometry. The results for the HEX8 and PEN6 elements show all the signs of being too stiff.

The hoop and vertical stresses in Figure 178 indicate that horizontal and vertical bending action increase with depth, a characteristic which probably follows from the shortening and rapid thickening of the arches with depth. The stresses therefore suggest that the water load is borne in the upper regions of the dam mainly by horizontal arching, and in the lower regions by bending action, which produces tensile stresses on the upstream face. The displacements in Figure 179 are not quite symmetrical due to the slightly steeper slope of the left bank.

The results for combined gravity and hydrostatic load in Figures 180-181 are very similar to those for hydrostatic load, because gravity load produces relatively low stresses and small displacements.
Figures 182-183 show the principal stresses obtained using the HEX20 and PEN15 elements, and the HEXI8 and PENI6 elements. The magnitudes and directions of the principal stresses for the two analyses are in good agreement for the main body of the dam. Near the base of the central monoliths and along the dam and socket interface, agreement is maintained in their directions, but not in their magnitudes. The differences may occur because the HEXI8 and PENI6 elements are less able than the HEX20 and PEN15 elements, to accept the discontinuity in the geometry at the interface. Both analyses, however, predict tensile stresses on the upstream face of the dam at the interface. The largest occurs low down on the left bank and is of sufficient magnitude to initiate tensile cracking.

<table>
<thead>
<tr>
<th>Program section or parameter</th>
<th>HEX20 PEN15</th>
<th>HEXI8 PEN16</th>
<th>HEX8 PEN6</th>
</tr>
</thead>
<tbody>
<tr>
<td>Data processing</td>
<td>13</td>
<td>2</td>
<td>3</td>
</tr>
<tr>
<td>Pre-solution</td>
<td>85</td>
<td>29</td>
<td>13</td>
</tr>
<tr>
<td>Frontal solution</td>
<td>163</td>
<td>11</td>
<td>12</td>
</tr>
<tr>
<td>Post-solution</td>
<td>42</td>
<td>10</td>
<td>11</td>
</tr>
<tr>
<td>Dummy overlay</td>
<td>19</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>Total time (secs)</td>
<td>322</td>
<td>58</td>
<td>46</td>
</tr>
<tr>
<td>Degrees of freedom</td>
<td>1338</td>
<td>390</td>
<td>390</td>
</tr>
<tr>
<td>Maximum semi-bandwidth</td>
<td>135</td>
<td>54</td>
<td>54</td>
</tr>
<tr>
<td>Length of ELPA required</td>
<td>14948</td>
<td>2529</td>
<td>2529</td>
</tr>
<tr>
<td>Core required on CDC 6400</td>
<td>31620</td>
<td>19712</td>
<td>19209</td>
</tr>
<tr>
<td>Cost in computer units</td>
<td>6.336</td>
<td>0.929</td>
<td>0.694</td>
</tr>
<tr>
<td>% costs</td>
<td>100</td>
<td>14.7</td>
<td>10.9</td>
</tr>
</tbody>
</table>

The behaviour of the HEXI8 and PENI6 elements is unquestionably good, being superior to the HEX8 and PEN6 elements, and only slightly inferior to the HEX20 and PEN15 elements. Their use in preference to the latter elements therefore hinges on the computer costs recorded in the table above.
The table shows enormous savings over the HEX20 and PEN15 analysis for all stages of the computer program. The extra time spent in the data processing is used in handling the mid-side nodes. Extra time is needed in the pre-solution to calculate the much larger element stiffness matrices, in the frontal solution to solve a larger stiffness matrix, and in the post-solution and dummy overlay sections to process the extra nodal points. The length of the working vector ELPA is governed in all three cases by the locations required to store the maximum size of the current overall stiffness matrix in the forward elimination phase of the frontal solution. The table clearly indicates that the use of mid-side nodes increases the maximum semi-bandwidth, and hence the size of ELPA. Their use draws greatly on the computer resources, as illustrated by the costs which are based on the time and core requirements of the job.

As the percentage costs in the table show, there is a saving in cost of approximately 85% in using the HEX18 and PEN16 elements, and as the agreement between the HEX18 and PEN16, and the HEX20 and PEN15 analyses is good, it seems most satisfactory to use the more economical elements.

5.3.5 Analysis of the Dam and Valley

The idealisation used to study the influence of the major faults on the stability of the dam is shown in Figures 184-185. The same mesh was used to represent the dam and socket in this analysis as was used in the previous analysis. To accommodate the dam and socket, the valley surface upstream of the dam was made slightly lower than the downstream surface. As in the model tests, the upstream limit of the finite element idealisation was defined by Fault 31; however, the fault was moved slightly upstream of its true position in which it undercuts the central monoliths of the dam, to facilitate the idealisation. The remaining limits were chosen taking into consideration the results of the two-dimensional analysis.
The valley was considered to be isotropic and was divided, as closely as was feasible, into the three zones of the model represented by the mixes P, G and D. The mechanical properties of the HEXI8 and PENI6 solid elements within these zones were the average experimental values obtained for Model I.

Originally it was hoped to represent all the fault planes constructed in the first model, however this was found to be impracticable. On the left bank, Faults 11, 12, 13 and Faults 2, 5, K, X were replaced by two representative planes. In order to include these planes in the idealisation, a pseudo-plane 30 m. below and parallel to the upstream valley surface was introduced to complete a circuit of PLR joint elements. Difficulties in connecting solid and joint elements were thus avoided.

On the right bank, Fault X was modelled by a plane which lay approximately 45 m. below the upstream valley surface, whose strike was parallel to the centre-line of the dam and whose dip was 37°. An exact representation of Fault Eloisa was unviable. As a compromise, the fault was simulated by a plane which surfaced just downstream of the dam and whose slopes \( \frac{dz}{dy} \) and \( \frac{dz}{dx} \) were equal to those of Fault Eloisa and the right bank valley surface respectively. This ensured a reasonable simulation of any upstream movement of the dam on Fault Eloisa produced by its self-weight, due to the fact that the slope \( \frac{dz}{dy} \) dips upstream. Between Faults Eloisa and X, planes representing Family D2 and running the full length of the valley were made normal to Fault X. These planes produced eight blocks surrounded by joint elements, except at the valley surface and at the upstream and downstream limits of the idealised valley. The unit shear stiffnesses \( k_x' \) and \( k_y' \) of the joint elements were assumed equal; the value for each geological discontinuity in the computer model was obtained from the load-displacement characteristics of the appropriate joint material used in Model I, Figure 16. As a realistic value of the unit normal stiffness \( k_z' \) was not available, analyses were undertaken for various values of the stiffness ratio \( R = \frac{k_z'}{k_y'} \).
The boundary conditions imposed resembled as closely as possible the restraints in the experimental models: the base was completely fixed, no outward movement of the sides was allowed and, at the downstream end, no downstream movement was allowed except for those nodal points on the right bank above Fault X. No boundary conditions were imposed on Fault 31, the upstream limit of the model.

Only four basic load cases were considered:

a) self-weight of the dam assuming a specific gravity of 2.4
b) self-weight of the valley assuming a specific gravity of 2.6
c) hydrostatic load on the dam and socket
d) hydrostatic load on the grout curtain.

In order to include the effects of uplift, hydrostatic load was applied to the dam and socket assuming a linear pore pressure distribution through the dam. However, due to the system of drainage galleries in and below the prototype dam, and the extensive grouting below and radiating upstream from its base, it was considered that the pore pressure in the foundation would probably fall so rapidly that the hydrostatic load on the grout curtain in the computer model, would be adequately represented by a number of concentrated point loads acting on Fault 31. As in the experimental models, only that part of the grout curtain on the right bank above Fault X was considered.

For joint stiffness ratios of $R = 1, 10, 100$ and $1000$, various combinations of the four basic load cases were considered in order to study the effect of $R$ on:

1) the stresses in the dam
2) the self-weight of the valley as a stabilising force
3) the hydrostatic load on the grout curtain as a destabilising force
4) the stability of the dam under overload conditions.
The first study showed that the stresses in the dam were independent of $R$. By comparing the results of the rigid valley analysis with those of the elastic valley analysis, Figures 176-183 and Figures 186-194, the effects of foundation deformability can be assessed. Under gravity load, the stresses for the crown cantilever indicate an increase in horizontal bending action and a decrease in vertical bending action. The vertical stresses do not show a reversal in bending moment: this may be so, but it is more likely due to an inadequate number of elements being used to model the changes in geometry at the dam and valley interface. A comparison of Figures 176 and 186 shows that the vertical stresses at the base of the upstream face appear to support this hypothesis.

Although the displacements due to gravity load in Figure 187 are small, they do show an unsymmetrical behaviour and a pattern which is completely different to those for the rigid valley analysis, Figure 177. There is a definite tendency for the dam to move upstream, especially on the right bank. Near the right abutment, where Fault Eloisa is near the surface, the resultant movement is in an upstream direction while near the base it is not, probably because Fault Eloisa is very much deeper. The difference between the crown cantilever displacement curves is reduced by rotating the curve for the elastic valley analysis about the base. It suggests, therefore, that the vertical displacement at the base of the upstream face should be greater than that at the downstream face, and the finite element results confirm it. Further, a logical argument would be: since hydrostatic load would produce a tensile stress at the base of the upstream face, the geometry of the dam is chosen to counteract the tensile stress by producing a compressive stress under the self-weight of the dam. However, the upstream foundation area is reduced by Fault 31, resulting in larger compressive stresses and vertical displacements at the base of the upstream face. The tensile vertical stress at the base of the upstream face in Figure 186 conflicts with this behaviour, further supporting the hypothesis.
The principal stresses due to gravity load in the lower regions of the dam in Figure 188 must therefore be considered cautiously. For the main body of the dam, the principal stresses indicate a dominant cantilever action, and only minimal arching stresses.

The stresses and displacements due to hydrostatic load on the dam and grout curtain are presented in Figures 189-190. Results are also given for hydrostatic load on the dam only. Hydrostatic load on the grout curtain produces only slight changes in the stress distribution in the dam. It has an increasing effect on the radial displacements with depth, a characteristic which can probably be attributed to the water pressure increasing with depth. The increases are larger on the right bank because load was only applied to the grout curtain on that bank.

A comparison of the stresses in Figures 178 and 189 and the displacements in Figures 179 and 190 shows clearly that the displacements are affected to a much larger extent by the valley deformability. The changes are most pronounced in the lower regions of the dam where the mean compressive hoop stresses are increased and the vertical stresses are reduced. Contrasting with these, are the smaller effects on the hoop stresses in the upper regions of the dam, where the only significant effects are near the abutments, and in the departure from the symmetrical behaviour predicted by the rigid valley analysis, which is not surprising because Fault Eloisa is only just below the right abutment. The effect of foundation deformability on the crown cantilever displacement curves for hydrostatic load on the dam, consists of two parts: a downstream body movement and a base rotation which produces a small additional downstream movement of the crest. The hydrostatic load on the grout curtain produces a further, but smaller, downstream body movement, and an approximately equal, but opposite, base rotation. The net effect of foundation deformability and hydrostatic load on the grout curtain is therefore a downstream body movement.

Figure 191 shows the principal stresses for hydrostatic load on the dam and grout curtain. Of significance are the principal tensions
on the upstream face, especially those at the base on the left bank large enough to initiate tensile cracking. Although some of their magnitudes are indicative of an inadequate number of elements being used in the idealisation of the dam and valley interface, they are not thought to be sufficiently erroneous to ignore their implication.

Figures 192-194 show the stresses and displacements due to combined gravity and hydrostatic load. Because the effects of gravity load are small, they show almost the same characteristics as those due to hydrostatic load. A comparison with Figures 180-183 giving the results of the rigid valley analysis, shows that the displacements and those stresses close to the dam and valley interface are significantly affected by foundation deformability, features which were also found in the two-dimensional analysis of the dam, Section 5.2.5. An inspection of the principal stress directions in Figures 183 and 194 reveals that slightly more arching action and less cantilever action can be expected to occur in an elastic valley than in a rigid valley.

Indications as to the stability of the fault planes under full prototype loading, including the self-weight of the valley, are given in Figures 195-203. As both experimental models failed with a major slide on the right bank, only the results for Faults Eloisa and X, and the nine planes representing Family D2, have been considered, Figure 184.

Figure 195 shows the joint elements representing Fault X while Figures 196-197 present the stresses and direction of maximum shear stress in the joint elements for stiffness ratios of \( R = 1 \) and 1000. The tensile normal stresses in the five deepest elements between the dam and Fault 31 indicate that the joints will open. This may be attributed to the tensile forces at the base of the upstream face, produced by the hydrostatic load on the dam, acting in conjunction with the hydrostatic load on the grout curtain. Except for these five elements, the directions of the maximum shear forces are predominantly in the dip direction. This suggests that a slide downstream on Fault X is not likely. The shear
and normal stresses at the centroid of each element were used to calculate the safety factors, defined as:

\[
\text{Safety factor} = \frac{\text{Limiting Coefficient of Friction} \times \text{Normal Stress}}{\text{Maximum Shear Stress}}
\]

However, since the directions of the maximum shear stresses are in the dip direction, the values of the safety factors are somewhat meaningless, especially well downstream of the dam. This is because the two banks of the prototype valley have already wedged themselves together under their self-weight into a state of equilibrium. A comparison of Figures 196 and 197 shows that, because the shear stresses are lower and the normal stresses are higher for \( R = 1000 \) than for \( R = 1 \), the greater the stiffness ratio \( R \), the less likely is a slide to occur on Fault X. The stresses in elements 109-116 and 117-124 are similar; it is therefore possible that the finite element idealisation would have been better if the downstream limit had been closer to the dam, enabling a finer division of elements.

The stresses on Fault Eloisa are less than those on Fault X, Figure 198. This is to be expected due to the self-weight of the valley, Fault Eloisa lying above Fault X. Further, the directions of the maximum shear stresses are not predominantly in the dip direction, and the overall safety factor appears to be independent of \( R \).

Figures 199-202 give the results for some of the planes representing Family D2. The directions of the resultant shear forces are generally in the dip direction, indicating that the blocks formed by these planes and Faults X and Eloisa, are being pushed into the valley. Any relative movement between them in a downstream direction is therefore most unlikely, which indicates that if a slide occurs on Fault X, then the blocks will slide en masse. This is reflected in the higher safety factors obtained for these planes than for Faults Eloisa and X. A comparison of the results for the elements below and immediately downstream of the dam, shows that the directions of the maximum shear stresses for Planes 1 and 3 are in a downstream direction, while for Planes 6 and 9, they are in an upstream direction. Further downstream, they are in the dip direction for all four planes. As Planes 6 and 9 are
near the right abutment, where the hydrostatic load on the grout
curtain is negligible compared to that in the vicinity of Planes 1 and
3, the results suggest that the schistosity planes, although not
represented in the analysis, will open up, and the valley upstream
and downstream of the right abutment will separate.

Figure 203 summarises the results by presenting the total
shear and normal forces acting on each joint plane: the values quoted
are those given by the computer to 6 significant figures, although of course,
their true worth is only to, perhaps, 2 significant figures. The forces
were obtained by integrating the stresses over the area of each joint
element, and summing the component forces for all the elements. As
indicated, the majority of the resultant shear force directions for
Planes 1-9 are in the dip direction, especially for \( R = 1 \). As failure
of the valley can only occur on these planes by sliding downstream, the
safety factors are not truly representative. While a similar argument
holds for Fault X, it does not do so for Fault Eloisa. If the safety
factor against sliding downstream is taken as

\[
\lambda_s = \frac{\lambda}{\cos \theta}
\]

where \( \lambda \) is the safety factor listed and \( \theta \) is the direction of the
resultant shear force, then a safety factor less than unity is obtained
for sliding in a downstream direction. The analysis therefore predicts
that failure is very likely at the prototype load if strengthening of the
valley is not undertaken.

The effect of the self-weight of the valley as a stabilising
force has been summarised by plotting the safety factors and directions
of maximum shear stress for those elements immediately downstream
of the dam, Figure 204. These elements were chosen because their
safety factors were generally lower than the corresponding elements
immediately below the dam. The results show that the stability of
Faults Eloisa and X is very much dependent on the self-weight, altering
the directions of the maximum shear stresses from the downstream direction
to the much safer dip direction, and thereby improving the safety factors against sliding downstream. A notable change in direction also takes place for Planes 1-9 for a stiffness ratio of \( R = 1 \). There are, however, only slight changes for \( R = 1000 \). Without the self-weight of the valley, the overall stability of the joint planes is extremely low, with safety factors generally less than 0.5 and reasonably independent of \( R \), Figure 205. If the direction of the appropriate resultant shear force is also considered, the safety factor against sliding downstream for Fault X is increased from 0.26 to approximately 9.5, and that for Fault Eloisa from 0.26 to approximately 0.87. The improvements are shown by a comparison of Figures 203 and 205.

Figure 206 indicates that hydrostatic load on the grout curtain has no significant destabilising effects. However, a comparison of Figures 203 and 207 shows that the direction of the resultant shear force on Fault Eloisa is altered by approximately 13\(^\circ\) by the grout curtain load. The result is a decrease in the safety factor against sliding downstream from approximately 1.46 to 0.87. Fault Eloisa is therefore made unstable by the hydrostatic load on the grout curtain.

Although analyses were undertaken for several of the overload conditions to which Model II was subjected, only one has been presented in Figures 208-209. This corresponds to the last overload condition which Model II safely sustained, a load factor of 1.0 on the gravity load on the dam and valley, and a load factor of 4.0 on the hydrostatic load on the dam and grout curtain. Figure 208 indicates that Fault Eloisa is unstable under the overload condition, the directions of the maximum shear stresses not only being in a predominantly downstream direction, but also suggesting possible movement up the plane near the right abutment, movement which must occur if the dam is to fail. Although changes occur in the safety factors and directions of the maximum shear stresses on Planes 1-9 and Fault X, they are not as significant as those for Fault Eloisa. A comparison of Figures 203 and 209 shows that the direction of the resultant shear force on Fault Eloisa has swung through approximately
to an almost downstream direction. The safety factor against sliding downstream, which is independent of the joint stiffness ratio $R$, is consequently reduced from approximately 0.87 to 0.29.

5.4 SUMMARY

Following a two-dimensional study to determine to what extent the valley should be included in a three-dimensional idealisation of the dam and its valley, an investigation into the structural interaction between the dam and its valley was undertaken using Wilson's brick element with incompatible displacement modes and Mahtab and Goodman's joint element.

The suitability of Wilson's element was confirmed by comparing the results of a rigid valley solution with those obtained using the conventional 8 and 20 node isoparametric brick elements. Their use also proved to be much cheaper in terms of computer time.

The three-dimensional analyses of the dam supported on an idealised elastic valley show that the stresses in the dam are independent of the joint stiffness ratio $R$, and that the displacements are affected to a much larger extent than the stresses by the valley deformability. The analyses also indicate that sliding is most likely to occur on Fault Eloisa rather than Fault X. This is because Fault X, being deeper, is stabilised by the self-weight of the valley. The stability of Fault Eloisa is also shown to be dependent on the hydrostatic load on the grout curtain; without it the safety factor against sliding downstream is 1.5, with it it is less than unity.

In order to investigate further the interaction between the dam and its valley, it remains to correlate these theoretical results with the experimental results of Models I and II.
6.1 INTRODUCTION

The techniques used at Imperial College for the manufacture and testing of concrete models in concrete valleys under simulated water load, have been extended in order to investigate the stability of a dam in a realistically modelled valley. Such models can be particularly useful to the designer, because they can include such features as asymmetry, irregularity of valley shape and stiffness, and may incorporate major geological features. The effects of spillways, stilling basins and general valley reinforcements, such as shear keys, may also be investigated.

Although the model may well yield a safety factor which is reasonably close to the true one, model making and loading techniques often produce local stress concentrations and, sometimes, local failures. The stresses in the model dam may not, therefore, be sufficiently accurate to act as the only reference for design. Further, tangential and vertical displacements and internal stresses are difficult to measure in a model.

It is therefore essential that any model investigation should be accompanied by a theoretical analysis. Although dynamic relaxation, integral equations and trial load methods are capable of analysing dams, the finite element method provides a more general and simpler formulation, and has the additional advantage of being able to analyse the valley, which otherwise would have to be analysed by statics. Even this method, however, can produce spurious results if a non-conforming element is used and the analyst is not aware of its limitations.

6.2 REVIEW OF EXPERIMENTAL BEHAVIOUR

The results for both models were influenced by the loading techniques, the major source of error being the external gravity load applied to the crest of the dam. However, any alternative method would have required either damaging the downstream surface of the dam, or
simulating the gravity and hydrostatic load acting on the dam by a single system. This would have complicated the load system and sacrificed the ability to vary the ratio of hydrostatic load to gravity load. The external gravity loading system produced local bending effects in the upper regions of the dam, resulting in some large fluctuations in the hoop stresses. This was deduced by investigating the individual effects of gravity and hydrostatic load; the curves for hydrostatic load were found to be smoother. Stress concentrations and local crushing below the gravity loading pedestals on the valley limited the extent to which both gravity and hydrostatic load could be increased proportionally. The limit was 2.4 in Model I and 2.1 in Model II, the reduction probably following from the smaller size of briquette used to construct the valley.

The general behaviour of both models was similar, arching action predominating in the upper regions of the dam where the arches were thin, and bending action in the lower regions where they were comparatively thick. On increasing the load factor above 1.0, load was shed from the middle and lower regions of the left bank to the right bank, where most of the extra load was carried by arching action in the upper regions of the dam. The right bank therefore supported the greater proportion of the load. Owing to a reduction in base area due to Fault 31 undercutting the central monoliths, cantilever action was inhibited and vertical compression increased at the base of the central monoliths. Hydrostatic load on the grout curtain had more significant effects on the displacements than on the stresses, the effects naturally being larger on the right bank than on the left bank. As expected, gravity load on the valley had no significant effect on the stresses in the dam, though it did help to prevent sliding on Faults Eloisa and X; it is therefore essential for the stability of the dam. This is confirmed by the way in which failure was induced in Model I, by reducing the gravity load factor below 1.0.
Both models failed with a major slide on Fault X, and the events leading up to failure were very similar. Valley movements were insignificant until failure occurred, when there was a sudden increase, indicating that the peak strength of the joint material had been reached. The sudden movement on Fault X caused the right bank side of the dam to rotate about the left bank, and the movement was so large that there was a reduction in shear resistance at the dam and valley interface on the left bank. Consequently, load was shed into the upper regions of the dam where there was an increase in the hoop compression. In Model II, the increase was sufficient to cause the left abutment block to fail. The rotation of the dam about the left bank also caused some of the construction joints to open up. The failure characteristics of Model II, which include the separation of the briquettes at the right abutment, the opening up of Fault 16, and the development of large cracks on the right bank of the valley, were more striking than those of Model I. This was probably influenced by the smaller size of briquette used to construct the valley.

The main differences in design of the two models, other than the size of the briquettes, were the addition in Model II of the strengthening measures and the application of gravity load to the whole of the left bank, the latter being a direct result of the behaviour of Model I. These differences may help to explain the major difference in behaviour, that of the dam and the valley interface on the left bank in Model II, which occurred before a load factor of 1.0 had been achieved. This will be discussed in Section 6.4.

6.3 REVIEW OF THEORETICAL BEHAVIOUR

Two main points arise from the two-dimensional analysis. The first is that erroneous stresses can occur at sudden changes in geometry, where the finite element mesh is too coarse and nonconforming elements are used. If a singularity occurs, however, then fineness of mesh is not enough. The second point is, that only a relatively small part of the valley beyond any discontinuity need be included in an elastic
analysis, in order to investigate the interaction between a dam and its valley. This applies, however, only to the more flexible higher order elements.

Two general points arise from the three-dimensional analyses of the dam supported by a rigid valley. Despite Clough's recommendation\(^{31}\), the position of the mid-side nodes in the HEX20 element is not arbitrary. The accuracy of the nodal point stresses is greatest when the nodes are at the mid-point of each side, the displacements are also sensitive to their positioning, but to a much lesser extent. The second point is, that in spite of its behaviour at sudden changes in geometry, the performance of the nonconforming Wilson element is extremely competitive in terms of computer time, in comparison with the conforming HEX20 element.

The most important fact to emerge from the three-dimensional analysis, is the prediction of tensile stresses on the upstream face at the dam and valley interface. They are sufficient to initiate cracking and thereby invalidate the elastic analysis. Any tensile stress at the upstream interface is caused by an imbalance between the hydrostatic and gravity load effects. Surprisingly, although gravity load was applied to the dam as a whole, rather than individual monoliths, the principal stresses due to gravity load show a predominant cantilever action, with only small vertical compressive stresses at the interface. It therefore appears that whatever assumption is made about the gravity load action, tensile stresses will always exist at the upstream interface.

Without hydrostatic load on the grout curtain, the behaviour of the dam is virtually symmetrical, and with hydrostatic load on the grout curtain, there are only slight changes in stress. The effect on the displacements is greater, especially on the right bank.

As the stresses and displacements due to gravity load are small, the behaviour of the dam under full load is very similar to that under hydrostatic load, when arching action predominates in the thin upper regions, and bending action in the thick lower regions.
The theoretical results show that foundation deformability affects the displacements to a much larger extent than the stresses. The increased flexibility at the dam and valley interface restricts the development of bending action. Consequently, slightly more arching and less cantilever action occurs when the dam is supported by an elastic valley than by a rigid valley.

The stresses and displacements of the dam are independent of the joint stiffness ratio \( R \). The elastic valley analyses also show that a slide downstream on Fault X is unlikely, although it is dependent on \( R \); the greater the value of \( R \), the less likely is a slide to occur. However, a slide on Fault Eloisa is most likely since the self-weight of the rock above it is not sufficient to stabilise it; the safety factor is independent of \( R \).

The stability of Faults X and Eloisa is therefore dependent on the self-weight of the valley. This is illustrated by the safety factors against sliding downstream in the table below. The table also shows that hydrostatic load on the grout curtain causes Fault Eloisa to slide.

Since Fault Eloisa is unstable under full load, it is not surprising that it is unstable under the failure load of Model II, as shown in the table on the following page. The analyses indicate that failure will occur with a slide on Fault Eloisa, with the schistosity planes in the vicinity of the right abutment opening up to separate the briquettes upstream and downstream.
of the right abutment. If a slide takes place on Fault X, it will occur with the briquettes founded on Fault X sliding en masse. The maximum shear stress directions for most of the joint elements forming Fault Eloisa, are predominantly in an upstream-downstream direction, with some showing evidence of possible outward and upward movement, the essential condition for the dam to fail.

Theory, therefore, predicts an initial failure on Fault Eloisa, although with a redistribution of the load path, due to a loss of stiffness on Fault Eloisa, it is possible that a slide will also occur on Fault X.

6.4 DISCUSSION

In Section 3.7, particular interest was focussed on the behaviour of Model II in the area of the left bank dam and valley interface, following the discovery of a non-linear behaviour of the elements of the strain gauge rosettes on both faces of the dam in this area. As a result, the history of these elements was compared with the elements of several rosettes in the body of the dam and some gauge elements on the right bank, Figures 88-95 and Figures 100-107. It should be emphasised that gauge reading drift was ignored in plotting these figures.

Examination of these figures showed that the right bank was clearly behaving in a normal way, while the left bank was, equally clearly, behaving in an unexpected manner. It is evident from the behaviour of the rosettes along the downstream interface that the unexpected behaviour began in the lower and upper regions of the left bank at an early stage in the loading programme. During Load Cases 1-33, the rosette at the
middle of the interface behaved consistently, but then the unexpected behaviour appeared at this rosette too. It can be concluded that by Load Case 47 some kind of damage had been sustained, either to the strain gauges or the model itself, along the left bank interface, while the rest of the dam was reacting to this damage rather than initiating it.

There are three possible causes of the unexpected behaviour. They are:

1. Strain gauge failure, which could have been caused by a number of agencies, including the internal moisture in the concrete of the dam, oil dripping from the overhead valley gravity jacks, or external mechanical damage to the wires or gauges.

2. Bad model making technique, of which the most likely example is failure to bed satisfactorily the monoliths of the left bank.

3. Tensile cracking in the dam or its immediate foundation.

It is clear that while the first suggestion is possible, it is unlikely that such a large group of strain gauges in one particular area would be affected without some visible signs appearing. As no signs appeared, this possibility can therefore be rejected. However, the second possibility cannot be rejected with absolute certainty since concrete, mortar packings, and joint grouting materials are never absolutely homogeneous. It is possible that some fault did occur in the manufacture and placing of the dry pack under the monoliths of the left bank, but great care was taken during the operation due to the obstructions caused by the internal gravity loading rods of the dam. Unfortunately, the dry pack was crushed to powder during the failure of the model, and therefore its condition prior to failure is unknown.

The third possibility, that of tensile cracking and uplift, is more complex. However, if tensile stresses of the kind predicted by
the finite element analyses, Figures 175, 182 and 194, occurred in the model, then neither the dry pack nor the briquette construction would have been able to resist them. The subsequent failure would lead to low stresses on the upstream face, and larger compressive stresses on the downstream face as the hinge formed and rotated, and the line of action of the net membrane force was driven downstream. Another feature of the model lessening its resistance to upstream tensile stress at the interface, was the manner in which Fault 31 undercut the central monoliths.

On the balance of probability, the third possibility seems much the most likely of the three. It may be relevant to note that the ultimate failure of the model was associated with a crack through the interface over most of the left bank area, Figure 158.

If the behaviour of Model II is associated with tensile cracking, Model I would also be expected to show a similar type of behaviour. Similarity exists to the extent that high compressive stresses were found on the left bank downstream in Model I, together with low stresses on the upstream face, Figure 41. However, Figure 35 shows that in Model I, hydrostatic load produced tensile stresses on the upstream face along the interface. It appears, therefore, that the gravity load of the dam reduced the tensile stresses to a greater extent in Model I than Model II. This may possibly be attributed to the larger briquettes in Model I and the procedures used to grout the construction joints. In Model I, the joints were grouted at normal laboratory temperature with half of the gravity load of each monolith applied to it, whereas in Model II, the joints were grouted at 5 - 6°C in the absence of gravity loading. Smaller tensile vertical stresses in Model I were therefore inevitable.

In order to minimise the effect repeated loading might have on the non-linear behaviour of Model II, and for purposes of comparison with Model I and the theoretical results, the stresses obtained from Load Case 2 for a load factor of 0.5, and from Load Case 11 for load factors of 0.5 and 1.0, are presented in Figures 210-213. A comparison
of Figures 211 and 212 shows the development of high compressive stresses on the downstream face at the left bank interface. On the upstream face, opposite the high stresses on the downstream face, only low stresses are found. There is also an increase in the compressive stresses on the upstream face of the crown cantilever. Figure 213 shows the principal stresses for the first subjection of the dam to a load factor of 1.0. They are reasonably consistent with those for a load factor of 0.5, Figure 212, and in good agreement in both magnitude and direction with those of the much later Load Case 34, Figure 113. This suggests that the lift off of the upstream face on the left bank occurred at a very early stage in the testing programme. The tendency which this action has, to increase the compressive stresses on the downstream face, is confirmed by the results of a rigid valley finite element analysis in which some of the upstream nodes at the base of the socket were released; compare Figures 214 and 183. Although the effect is not very marked, it must be remembered that there is better continuity along the base of the finite element model than in the experimental model, and that the latter is certainly more pessimistic than in the prototype.

The experimental and theoretical stresses and displacements for a load factor of 1.0 are compared in Figures 215-217 and, because of the local bending effects and stress concentrations produced by the loading techniques, the mean stresses have been plotted alongside the stresses on the upstream and downstream faces. The mean hoop stresses for various levels are presented in Figure 215. Although theory predicts an almost symmetrical behaviour, both models show higher compressive stresses on the right bank than on the left bank, and generally the right bank stresses are higher than theory, while those on the left bank are less than theory. The hoop stresses on the left bank for Model I are usually less than Model II; this probably reflects a stiffer foundation for Model I, due to the much larger briquettes permitting a more significant cantilever action to develop. The crown cantilever stresses are presented in Figure 216. The mean hoop stresses are less than theory and the mean vertical stresses are greater than theory, and are
therefore consistent, one compensating for the other's deficiency. The results of both models are in good agreement and show a rapid increase in mean vertical stress towards the base, reflecting the reduction in foundation area due to Fault 31 undercutting the central monoliths.

The experimental displacements in Figure 217 were affected by local bending, especially in Model II, and consequently the correlation between the experimental and theoretical results is poor. Further, good agreement was not expected because of difficulties encountered in supporting the deflection frames. The frames were mounted differently in each model, making the reference data different, and both probably underwent unknown rigid body movements during testing due to the bending of the strong floor under the heavy gravity loads. Although the displacements in Model I did indicate the need for sufficient gravity load to be applied to both banks in Model II to prevent unrealistic movement of the abutments, their real value is restricted to indicating failure by sudden increases.

Several reasons can be found for the differences between the experimental and theoretical results. Among them are:

a) In the theoretical idealisation, Fault 31 was modelled upstream of its true position undercutting the central monoliths. Theory would therefore predict greater vertical bending action and a lower mean compressive stress at the base of the monoliths, Figure 216.

b) The theoretical idealisation of the foundation was stronger since the schistosity planes were ignored.

c) Only linear elastic finite element analyses were undertaken. No attempt was made to inhibit the development of tensile stresses in the dam or valley by introducing brittle material properties and solving iteratively for a limited tension solution. The adjustment is particularly necessary in the
vicinity of the abutments where the self-weight of the lower regions of the valley tends to pull down, rotate and lift off the abutment. This is illustrated in Figure 204 by the self-weight of the valley producing tensile stresses on Fault Eloisa.

d) Neither strengthening measures, such as shear keys, mid-height spillway, or size of briquettes were represented in the finite element model.

e) Hydrostatic load in the analyses was applied as a linear pore pressure distribution through the thickness of the dam, whereas in the models it was applied directly to the upstream face. Uplift effects in Model I were represented by reducing the foundation load. No attempt was made in Model II to include them.

f) The boundary conditions of the models and finite element analyses were not identical, since some small outward movement of the model walls was bound to occur due to the close proximity of the abutments.

g) The positions of Fault Eloisa in the theoretical idealisation and the experimental models were different. In the models, Faults Eloisa and X intersect, Plate 1, whereas in the finite element model Fault Eloisa always lay above Fault X, Figure 184. The safety factor is critically dependent on the depth of the fault plane, and since Fault Eloisa is deeper in reality, the safety factors should be higher than those predicted in Chapter 5.

h) In both Model I and Model II, failure was induced with the correct stress distribution on Fault X, with Fault Eloisa overloaded. Initial failure on Fault Eloisa, as predicted by the finite element analyses, could not therefore be expected in the models.
Differences between the two model results are probably due mainly to the size of the briquettes and gravity loading of the left bank, with minor effects from the method of construction. Visual inspection of the construction joints before testing Model II showed them to be of a better quality than in Model I, but unfortunately there is evidence that the construction joint on the right bank, Figure 158, opened before failure occurred, being instigated by the gravity loading applied to the crest of the dam.

6.5 CONCLUSIONS

Model I established that valley failure, rather than dam failure, was the essential criterion. Dam failure was induced by excessive movement of the right bank on Fault X, not by the dam being unable to sustain the increased hydrostatic load. This confirms that arch dams have a high strength, due to their ability to draw on their hyperstatic reserves to solve the stability and strength problems associated with the foundations. This was even more convincingly shown in Model II, when the dam behaved remarkably well under a hydrostatic load factor of 4.0, in spite of the unexpected behaviour at the left bank dam and valley interface. The ensuing stress redistribution from the upstream to the downstream face was quite local, which is not surprising because the dam is very thick at the interface.

Collapse is related to the stability of the right bank, and is related more to the yielding of the right abutment than to the intensity of the load applied. As the gravity load on the valley was reduced to induce failure, the increase in displacements was more significant than the stresses. The abutments eluded large extra loading by undergoing large deformations, not necessarily elastic, until the stiffer concrete structure collapsed upon exceeding its limits of deformability in accordance with the deformations imposed on it by the yielding of the abutments. The amount of deformation depends on the size of the briquettes, and hence the joint spacing. Although the
two models consisted of different sized briquettes, the models were also different in many other respects. The effect of size cannot therefore be accurately assessed, but it is almost certain that it did influence the behaviour at the interface and the well defined failure mechanism of Model II.

In both models failure occurred suddenly, and since the valley movements were relatively small until failure, failure can be reliably associated with the exceeding of the peak strength of the joint material simulating the frictional properties of Fault X. The dam movements were not small, and progressively increased as the gravity load on the valley was decreased. This suggests that if progressive movement of an arch dam is recorded, then there is cause for concern since failure is most likely to occur suddenly. This was certainly characteristic of the Malpasset failure.

The model results show that the existence of faults in the foundations of arch dams cannot be ignored. As the central monoliths were constructed with the upstream edge overhanging Fault 31, the models probably exaggerated its effect. However, the models do illustrate the need to consolidate the foundation and replace joint material with concrete, especially if the foundation rock is weak and high compressive stresses are to be avoided, and differential settlement reduced. If such remedial action is not undertaken, hoop compression in the upper arches may increase, intensifying the load on the abutments.

The models illustrate the importance of foundation stability and strength in arch dam construction. This finding is borne out by the finite element analyses which also indicate other important factors. As Fault Eloisa was idealised as a plane above Fault X, the analytical work can be regarded as a study of the depth of rock required above a fault plane, for its self-weight to provide sufficient stability. Fault X, which is approximately 40 m. deep, was found to be stable. However, Fault Eloisa was found to be unstable when full hydrostatic load was applied to the grout curtain. This accentuates the need for a good drainage
system, in combination with a grout curtain, to reduce pore water pressure in the rock, and prevent seepage into the joints and faults causing a reduction in the coefficients of friction. The stability of Fault Eloisa illustrates the importance of self-weight of the valley and suggests that where faults exist at shallow depths, it is essential to prevent rock slides occurring during excavation and, if necessary, to undertake rock anchoring measures designed not only to prevent sliding, but also to improve the frictional properties of the potential sliding planes.

It can be concluded that, given sufficient skill and care, it is possible to build elaborate models to enable the designer to assess the stability of an arch dam. They can increase our knowledge of the structural interaction between an arch dam and its valley by providing a record of the elastic, inelastic and rupture deformations likely to occur in the structure prior to collapse. Model work can be complemented by analytical work involving the use of known finite element techniques.

Today, as more and more arch dams are built, the quality of arch dam sites is declining and the need for both model and analytical studies increases. With them, and the development of better techniques to strengthen weak valleys, it will be possible to construct arch dams in valleys which may once have been ignored.

6.6 SAFETY OF THE DAM

The experimental and theoretical studies have shown that the right bank is liable to slip, and that valley strength is essential for the safety of the dam. The studies have also shown the likelihood of tensile stresses at the heel of the upstream face of the dam, and bearing in mind the Malpasset failure, this is of major concern. However, adequate drainage may be the solution.

The finite element analyses, especially, have shown the importance of the self-weight of the valley, and therefore, the need to prevent local slides downstream during excavations for the mid-height
spillway and stilling basin. The analyses have also illustrated the precarious nature of the right bank, and the unfavourable action of hydrostatic load on the grout curtain. The designer's decision to reinforce the rock on the right bank by stressed cables to improve the shear resistance along the planes of weakness, and to construct a grout curtain with an efficient drainage system, reflect the need for remedial measures.

The models were subjected to two types of overload tests. The first consisted of increasing proportionally the load factors on gravity and hydrostatic load. If increased to rupture, this can be considered as measuring the safety of the dam. Local stress concentrations below the pedestals on the valley limited the extent to which the loading could be increased, but load factors of 2.4 and 2.1 were reached for Models I and II respectively. In the second type of test, the load factors on gravity and hydrostatic load were varied. With reference to Figure 218, this may be regarded as varying the coefficients of friction of the fault planes. A ratio of hydrostatic load factor to gravity load factor in excess of 4.0 was achieved in both models.

The analytical and experimental results might appear at first sight to be inconsistent, but the results presented in Figure 218, based on the limiting equilibrium of a block on an inclined plane, would seem to refute this. The plane considered is Fault Eloisa, and the loads used consisted of half of the hydrostatic and gravity loads acting on the dam, as given by the rigid shell finite element analysis, and an estimation of the weight of rock above Fault Eloisa. The results, which compare favourably with those of the finite element analyses, indicate how sensitive failure is to the upstream dip of Fault Eloisa, in reality approximately $9^\circ$, and that both upstream and downstream movement is possible. The latter is significant because upstream movement of the base in the models would have weakened the briquette construction of the foundation. Although there is no evidence that the
dam in Model I moved upstream, Figure 135 suggests that the base of the dam did move upstream in Model II. The smaller briquettes, and hence the number of joints in the foundation, probably account for this phenomenon.

It is felt unwise to try to assess the strengthening measures included in Model II. Their effect is considered to have been overshadowed by the smaller sized briquettes.

6.7 GENERAL APPRAISAL

The major disasters at Malpasset and Vajont focussed world attention on the question of the stability of rock masses and emphasised the need for thorough studies using both model and analytical techniques. However, site investigation often shows the rock mass to be too complex for rigorous mathematical analyses, and a model study in such circumstances may more closely approximate to the field conditions. This is probably true of the conditions at El Atazar, but nevertheless, the model studies were complemented by finite element calculations.

Any study, whether in the laboratory or on the computer, must necessarily be complex, and rational assumptions must be made if realistic results are to be achieved. The finite element model undertaken, although complex, was relatively simple compared with the two experimental models. However, its comparative simplicity is not to be scorned, for it did support the unexpected tensile cracking on the upstream face of the left bank in Model II, and indicate a mode of failure which was partly in agreement with the experiments. Even so, the experiments and finite element calculations had their limitations.

Unfortunately, there is nearly always conflict between the scale of testing desirable for a realistic appraisal of the rock mass properties, and that which is economically feasible. Thus the properties and geology of the model valley were based on results from relatively few expensive bore-hole and tunnel surveys, and the properties are therefore not known to any great precision for the full extent of the valley. The faults and
rock joints are not known to be planar and their frictional properties were based on in-situ shear tests carried out on samples which must have been drained and disturbed to some extent by excavation. The coefficients of friction supplied by MEDEA were no doubt pessimistic in order to provide a conservative estimate of the probable safety margin.

The properties of the rock on site vary with the level of confinement and therefore vary from place to place. In the studies, the variation was represented by only three mixes. The stress distribution in the valley was also considered due to self-weight only, and no account was taken for stresses due to geological movements, such as stress redistribution due to brittle fracturing of the rock during folding. Although not necessarily affecting the overall stability of the dam, they may well influence the mode of failure.

Much depends on a limited amount of site investigation, from which the problems must be assessed and preliminary calculations made to describe and decide on the most dangerous phenomena in the valley, and their likely effect on the safety of the dam. The model construction was in accordance with the specifications of MEDEA, which no doubt relied on the calculations \(^{14}\) indicating the safety of the right bank to be precarious. The major findings were also included in the finite element idealisation.

In designing the foundations for a dam, or checking the stability of a rock slope, the designer must make allowances for the scale effects which arise in the interpretation of the strength of jointed rock masses. The strength of rock is dependent on the distribution and size of discontinuities, the strength decreasing with increasing specimen size due to the larger number of defects. Thus, as the volume increases, the influence of discontinuities increases, and the behaviour of the rock mass depends significantly on their behaviour. No studies were undertaken to discover the size effects of the briquette construction in either of the two models. If the properties were lower than specified, which
is quite probably in Model H due to the small size of briquettes, the 
foundation deformability would have been greater and the stress 
distribution in the dam would have been incorrect. The size of 
briquettes, and hence the space between joints, should therefore be 
decided after a general check on a large unit consisting of several 
briquettes.

In order to produce a reasonable finite element idealisation of 
the dam, severe but rational assumptions had to be made. It was not 
considered necessary to undertake failure analyses which were incre-
mental and non-linear in property behaviour. They would have been 
extremely expensive and unjustified by the data obtained from the site 
investigations.

For designing the shape of an arch dam, and the unlikely case 
of a dam without valley weaknesses, finite element analysis using 
relatively few thick shell elements is adequate\(^{(19)}\). Only the final 
shape need be checked by constructing a model. If there are compli-
cations in the dam then a model is desirable to complement local fine 
meet analysis using three-dimensional solid elements. The HEX20 
and PEN15 elements are preferable to the HEXI8 and PENI6 elements 
since the latter have recently been shown to satisfy the patch test for 
rectangular meshes only, and only to converge when refinement of a 
rectangular mesh is made\(^{(33)}\). This is obviously a serious limitation 
for any investigation around intakes, outlets and spillways. However, 
despite these limitations, they proved to give satisfactory results for 
the rigid valley analyses, even though the elements were neither 
rectangular nor parallelepipeds. The results of the elastic valley 
analyses must therefore have some reliability, and this may be 
connected with the low value of 0.16 for Poisson's ratio.

At a late stage in the work, it was wondered if, by applying a 
water load as a linear pore pressure distribution through the thickness 
of the dam, the tensile stresses at the heel of the upstream face were 
excessive due to the low crest to base thickness ratio. Consequently,
a rigid shell analysis was undertaken applying the water load as a surface pressure. The results in Figure 219 show that the tensile stresses at the heel are not significantly reduced.

It was also thought that a better form of analysis was required for assessing the stability of the foundations than the statical calculations made available by MEDEA\textsuperscript{(14)}. However, it is now considered that in most cases such calculations are reliable enough, and only in special circumstances when there is extreme doubt about the safety of the foundations, need these calculations be supplemented by three-dimensional finite element analysis.

6.8 FUTURE MODEL TESTING AND RESEARCH

As one cannot really establish parameters for general valley behaviour, since the geometry of the dam specifies the magnitude and direction of the loads transmitted to the valley slopes, and the geology of each valley is different, we must treat each case on its own merits. Since relatively little is known about the failure mechanisms of jointed rock masses, it is essential to construct geomechanical models of designs where an arch dam is placed in a suspect valley. Such models are expensive, and great care must be taken to see that they yield valuable results. The following suggestions may be of some help in future model testing:

a) When modelling in concrete, and gravity load is applied to the valley, the loads may be large enough to deflect the floor. If so, the positioning and supporting of a transducer frame to provide a fixed reference datum becomes difficult. Investigations into the use of photogrammetry and lasers may be beneficial.

b) Careful consideration should be given to the possibility of varying the ratio of hydrostatic load to gravity load. If the ratio is kept constant, it may be better to combine the gravity and hydrostatic load on the dam into one system. This will
eliminate the local stress concentrations due to any internal gravity loading system, and dispose of the tie-rods which interfere with the free deformation of the model under large displacements. Also, external gravity loading of the crest should be avoided if possible.

c) Model loads can be reduced by modelling in plaster. However, a material of low modulus, such as plaster, presents difficulties in the measurement of strain because the strain gauges also have a low modulus. Plaster generally does not simulate the brittle properties of rock as well as concrete, and its use in rupture tests is therefore questionable.

There is also a need for research into:

a) The size effect on the elastic modulus and Poisson's ratio of jointed rock masses.

b) The failure mechanisms of jointed rock masses, for which simple briquette models can be constructed, and possibly verified by non-linear finite element analysis.

c) Progressive and sequential rupture, and changes in strength properties with strain and therefore time. This may best be resolved by studies in the field.
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Fissures and faults at different cross-sections are shown, the perspex sheets being inclined similarly to one of the main families of fissures. Fault X is the major fault on the left of the photograph. The zones of the three different rock properties used in the model are indicated by the letters P, G and D.
Plate 2
Perspex geological model

The major faults, Eloisa and X, are seen from upstream. Fault X forms the base of the model on the right bank.
Plate 3

General view of the laboratory, showing model during construction

The valley foundation formation below the briquettes on the left bank can be seen. The perspex sheet on the top of the frame was used for locating faults and levels in the valley. The reaction blocks in the foreground of the photograph were bolted to the strong floor of the laboratory. On the left of the photograph the briquette surfaces are being painted with emulsion paint before being made into the model valley.
The division plates between the monoliths can be seen. The moulds were made from blockboard panels at 10 metre intervals, the faces being produced by a layer of thin plywood strips. The moulds are in position for casting, the thin crest of the model being at the bottom.
Plate 5

_A general view of the model during construction_

In the left foreground the monoliths are shown with the gravity loading rods protruding from the base.
Plate 6
Valley viewed from upstream

The holes through which the gravity loading rods from the dam pass can be seen and also the larger holes containing discs for applying gravity loading to the foundation. The pattern of the briquettes on the right side of the valley can also be seen. The suspended chains were used to obtain the correct valley shape.
Plate 7

The model during construction

The monoliths are in position before being bedded in the valley.
Plate 8

View of the model during construction, from upstream

The monoliths are bedded in the valley and tapes cover the joints between the monoliths preparatory to grouting. The linkage for gravity loading of the foundation and dam can be seen underneath the model. Faults Eloisa and X are clearly visible. The upstream limit of the foundation has been taken as Fault 31.
Gravity loading on the right side of the valley, downstream of the dam

On the right hand side of the photograph the jacks and extension tubes can be seen bearing on the triangular plates distributing the load equally to four loading pedestals. On the left of the photograph are the pedestals, together with one beam and triangular plate, in position to receive the jack extension tube. The jacks are lapped rams and the troughs for collecting oil are immediately below the jacks.
Plate 10

View of the model from upstream, showing steel hydrostatic loading pads

The load is applied from the loading pads to the upstream face through two orthogonal layers of rubber strip. On the left of the photograph the swivel blocks can be seen ready to receive the jacks. The swivel blocks are mounted on a steel plate fixed to the reinforced concrete reaction blocks which are prestressed to the strong floor of the laboratory.
Plate 11
View from downstream of the model under test

The displacement recording equipment and the Seismotron, which is connected to an oscilloscope, are in the foreground and the valley loading jacks are in the background.
In the foreground are seven loading cabinets and in the left corner is the Solartron Data Logger. The valley gravity loading jacks react against the overhead beams.
Plate 13

A view of the valley downstream, after failure

The loading jacks have been removed. The movement of the valley on Fault X is indicated by the two small parallel lines at the bottom of the photograph. The distance between the lines is the movement of the valley on Fault X. The continuous line between the 2nd and 3rd rows of pedestals shows that the whole of the right bank moved bodily downstream.
Plate 14
The model after failure, from upstream
The main crack formation is shown and also the significant movement on Fault X of 20 - 30 mm on the right hand side of the photograph.
Plate 15

The right bank of the model, after failure

The movement on Fault X is shown from about level 750 and below this level Faults Eloisa and X can be seen. The centre monolith overhangs Fault 31.
Plate 16

View of the right bank of the dam from downstream, after failure
A shear failure occurred in the second joint from the right.
Plate 17

Valley foundation below the briquettes

The photograph shows an upstream view of the concrete foundation of the valley built on a raised steel base plate to which shear connectors were welded. The valley was enclosed by reaction blocks on the sides and perspex sheets at the upstream and downstream ends of the model.
Plate 18

View from upstream during construction

The major fault planes and the zones of the valley mixes, P, G and D were marked on the perspex sheets at the upstream and downstream ends of the model. The valley contours were marked on the perspex sheets supported by the scaffolding. The timber board, which is just visible behind the first few briquettes, was positioned in accordance with the schistosity plane which was used to define the shape of the briquettes.
The major fault planes were reproduced by cutting the briquettes using water as a lubricant. The briquette was clamped to the circular plate which was then rotated and raised so that the cut was in line with the circular saw. The trolley, which ran on rails, was then pushed towards the rotating cutting disc.
The valley at a late stage of construction

The photograph shows an upstream view of the valley with a stilling basin mock-up, piles for the right abutment and, just to the left of the piles, the shear keys in Fault Eloisa.
To ensure that the construction joints were grouted satisfactorily, the dam was cooled to a temperature of approximately 5°C and the joints grouted. At the same time, the gap between the reaction blocks and the valley on the left bank was also grouted, a similar gap on the right bank having previously been grouted under normal laboratory conditions.
The photograph shows the dam, stilling basin and ring beam on the downstream side of the right bank. The eight holes in the ring beam are for wires representing the pre-stressing of the ring beam. The load of each hydraulic jack, applying gravity load to the right bank of the valley, is distributed via a triangular plate and beam to four pedestals, each of which loads four briquettes. On the left bank, however, each pedestal loads one briquette.
The dam and valley

The hydrostatic loading plates on the dam and some of those for the grout curtain are in position, as are the pedestals on both banks.
Plate 24

Complete model from downstream

The photograph shows the hydraulic jacks and overhead reaction frame for the valley gravity loading, and the raking arms for the displacement transducers.
In the foreground are the teletype, line printer and Compulog data logger which recorded and processed the strain gauge and displacement transducer readings. The four loading cabinets were used to apply hydrostatic load to the dam and grout curtain, and gravity load to the crest of the dam, internally to the dam, and to the left and right banks of the valley. The model is surrounded by the overhead reaction frame for the valley gravity loading.
Plate 26

The dam after failure

The photograph shows the crack which developed between the upstream benching and the dam, and the failure of the left abutment block. In the bottom right corner, the distance between the briquettes and the dark strip indicates the amount which the valley slipped.
The cracks between the abutment block and the valley were the first to be detected. The abutment appears to have been thrust horizontally into the valley surface. Cracks are also visible on the upstream face of the dam.
Plate 28

**Downstream face of the dam after failure**

Extensive cracks are visible on the dam and left bank benching.
Plate 29

Right abutment after failure

Cracking took place only in the outer regions of the abutment block. Failure occurred with a major slide on Fault X; this is illustrated by the white gap between the briquette and the painted dark line.
Plate 30

Foundation after removal of dam

The photograph shows the stilling basin and the cracked left bank dry pack which, in places, appears to have crumbled. The gravity loading rods for the dam passed through the holes in the dry pack.
Plate 31

Piles in the right abutment after failure

The large crack running up the valley from the end of the downstream pile was found after the removal of the loading pedestals and one layer of briquettes. A confused crushing/cracking zone exists between the two piles. The upstream pile cracked at $45^\circ$ to the pile axis at its junction with the abutment. The darker polished surface in the photograph may be the site of some slip.
Plate 32
Right bank after failure

The photograph shows the large crack which was found after removal of the loading pedestals and the top layer of briquettes. The crack runs from the end of the downstream pile to the downstream end of the model.
Plate 33

Left bank after failure

Fault 16 is shown to have opened up while the others have remained closed.
GENERAL PLAN OF THE EL ATAZAR DAM.

FIG. 1
PERFIL GEOLOGICO DE LAS LADERAS

GEOLOGICAL SECTION OF THE VALLEY

FIG. 2
DEFINICION DEL ALZADO AGUAS ABAJO
PROYECCION DE UN PLANO NORMAL AL EJE

DEFINICION DE LAS FORMAS DEL PERFIL CENTRAL
DEFINICION EN PLANTA DE LA BOVEDA

DEFINICION DE LAS SUPERFICIES DE LOS PARAMENTOS

ARCOS CIRCULARES DE 3 CENTROS DEFINIDOS POR LAS ECUACIONES:

$v^2 + x^2 = R^2$ (zona central)

$y = f(x)$ (zona de los estribos)

Las arcos son de espesor verticalmente creciente de la clave para los paramentos.

SEMIRANURA DE LA ZONA CENTRAL

$\alpha = \text{constante} = 26.6^\circ$

COORDENADAS DE LOS CENTROS

COORDENADAS DE AGUAS ARriba

$Cm = G/2 + h/2 - 1/2 D^2/4$ con $h = 0.398980 \times 10^3$

$Dm = 0.849250 \times 10^3$

RADIOS DE LOS ARCOS

RADIOS DE AGUAS ARriba

$Cm + Cm + h = \text{radio de los estribos}$

$Cm + Cm + h = \text{radio de los estribos}$

NOTACIONES

$C = \text{profundidad con relación a la cota estribo}$

$D = \text{orden}$

$E = \text{espesor en el perfil central}$

$F = \text{proyección aguas arriba en el perfil central}$

$G = \text{distancia del orden a los centros de la zona central}$

$H = \text{distancia de los centros de la zona central a los centros de la zona de los estribos}$

$I = \text{ radios de los arcos}$

$J = \text{semiranura de la zona central}$

$K = \text{proyección aguas abajo en el perfil central}$

$L = \text{ corrientes de intersección de los arcos en el zócalo o con los estribos}$

ESCALA ESPACIAL

0 10 20 30 40 50 100 M
REINFORCEMENT OF THE LEFT AND RIGHT BANKS

FIG. 5
TRATAMIENTO DEL TERRENO

PANTALLA DE IMPERMEABILIZACION

Y DRENAJE

PERFIL DE CONSOLIDACION
VIGA ZOCALO

REINFORCED RING BEAM

PLANTA

SECCION A-A

SECCION B-B

DISPOSICION DE LA EMISION DE ANCLAJES.
EN VERTICALES LOS ANCLAJES SON DE 0.17, 0.18 Y 0.19

APARICION DE SOPORTE A X 20 P.M.
CON INTERVALOS DE 500 M.

ARMADURA DE SOPORTE A X 1/50

DESCAGES DE MEDIO FONDO

ESCALA GRAFICA
PROLONGACIONES SUBTERRANEAS EN EL ESTRIBO DERECHO

SECCION A-A

CONSOLIDACION

REINFORCEMENT OF THE RIGHT ABUTMENT

IMPERMEABILIZACION Y DRENAJE
Specified properties for dam and valley materials.

**FIG. 9**
### MIX MATERIALS BY WEIGHT

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<th>Materials</th>
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<td>Sand 100/170</td>
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### FRICION ANGLE SURFACE FINISH AND JOINT MATERIAL

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<td>Emulsion Paint 1 Keene's, 2 Talc</td>
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<td>Emulsion Paint &amp; Sand 7 Keene's, 3 1/2 Talc</td>
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<td>Emulsion Paint &amp; Sand 7 Keene's, 2 Talc</td>
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### CEMENTITIOUS MATERIALS JOINT MATERIALS

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<th>Dip</th>
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<th>Yg (m)</th>
<th>Zg (m)</th>
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#### DEFINITION OF FAULT PLANES

Materials and major fault planes.

**FIG. 10**
Elasticity of materials used in model valley.

FIG. 11
Ultimate strength of materials used in model valley.

**FIG. 12**
Mohr-Coulomb circles for materials used in model valley.

FIG. 13
Properties of dam, bedding and grouting materials.

**FIG. 14**
Coefficients of friction of materials used in joints of valley.

Fig. 15
Load-displacement characteristics of materials used in joints of valley.

**FIG. 16**
NOTES:-

a) The briquette is a parallelepipedon defined by the intersection of Family D1, Family D2 and Schistosity.

b) If $l_1$ is the intersection of D2 and the Schistosity $l_2$ is the intersection of D1 and the Schistosity $l_3$ is the intersection of D1 and D2

The parallelogram defined by $l_1, l_2$ and $l_3$ has sides of length 210 mm. and 240 mm.

c) The length of the side $l_3$ is governed by the distance of 100 mm. between the surfaces corresponding to Family D2.

d) The angles between the sides $l_1, l_2, l_3$ are $\angle l_1, l_2 = 99^\circ 5'$
$\angle l_1, l_3 = 77^\circ 55'$
$\angle l_2, l_3 = 134^\circ 18'$

*Definition of briquette.*

*FIG. 17*
Location of electrical resistance strain gauges.

UPSTREAM FACE
Viewed from upstream

ARCLENGTHS IN METRES
LINEAR GAUGES, PL10
GAUGE FACTOR = 2.07
ROSETTE GAUGES, PR 10
1
GAUGE FACTOR = 2.06
2
GAUGE FACTOR = 2.07
3
GAUGE FACTOR = 2.08

DOWNSTREAM FACE
Viewed from upstream
DIAL GAUGES ARE RADIAL TO DOWNSTREAM FACE

DOWNSTREAM FACE Viewed from upstream

DIAL GAUGES ON VALLEY
USE OF LOADING CABINETS DURING TESTS FOR SLIDING ON FAULT X

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<th>Loading Cabinet</th>
<th>Purpose</th>
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<td>Hydrostatic Load on Dam and Grout Curtain</td>
</tr>
<tr>
<td>2</td>
<td>External Gravity Loading on Dam</td>
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<tr>
<td>3</td>
<td>Internal Gravity Loading on Dam</td>
</tr>
<tr>
<td>4</td>
<td>Gravity Loading on Foundation</td>
</tr>
<tr>
<td>5</td>
<td>Left Bank Gravity Loading</td>
</tr>
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<td>6</td>
<td>Right Bank Gravity Loading</td>
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USE OF LOADING CABINETS DURING TESTS FOR SLIDING ON FAULT ELOISA

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<td>Gravity Loading on Foundation</td>
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<td>Gravity Loading of Fault X and Left Bank</td>
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<td>Top row of Gravity Loading on Fault Eloisa</td>
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<td>7</td>
<td>Bottom row of Gravity Loading on Fault Eloisa</td>
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Use of the loading cabinets.

FIG. 20
## Preliminary Tests

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<th>Load Series</th>
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<th>Gravity Load</th>
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*Summary of load cases 1-52.*

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**FIG. 22**
Crown cantilever radial displacements for hydrostatic load with faults X and Elisa prevented from moving.

**FIG. 23**

- LOAD CASE 44
- LOAD CASE 46
- RESIDUAL SHAPE

**Crown cantilever radial displacements (mm)**

**Hydrostatic load on dam**

**Level (m)**

**Hydrostatic load on dam and grout curtain**
Radial displacements at levels 817 and 867 for hydrostatic load with faults X and Eloisa prevented from moving.

FIG. 24
Hoop stresses at levels 817 and 867 for hydrostatic load with faults X and Eloisa prevented from moving.

**FIG. 25**
Crown cantilever hoop stresses for hydrostatic load with faults X and Eloisa prevented from moving.

**Fig. 26**
Crown cantilever vertical stresses for hydrostatic load with faults X and Eloisa prevented from moving.

Fig. 27
Principal stresses for hydrostatic load on dam with faults X and Elosa prevented from moving given by load case 40.

UPSTREAM FACE
Viewed from upstream

DOWNSTREAM FACE
Viewed from upstream

Tension (Kg/cm²)

Compression (Kg/cm²)
Principal stresses for hydrostatic load on dam and grout curtain with faults X and Etola prevented from moving given by load case 42.

**Downstream Face**
Viewed from upstream

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Viewed from upstream

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Testing fault Eloisa with fault X prevented from moving.

Testing fault X with fault Eloisa overloaded.
Radial displacements at levels 817 and 867 for hydrostatic load for sliding on fault Eloisa, and for sliding on fault X.
Hoop stresses at levels 817 and 867 for hydrostatic load for sliding on fault Eloisa, and for sliding on fault X.

FIG. 32
Crown cantilever hoop stresses for hydrostatic load for sliding on fault Eloisa, and for sliding on fault X.

Fig. 33
Crown cantilever vertical stresses for hydrostatic load for sliding on fault Eloisa, and for sliding on fault X.

FIG. 34
Fault X prevented from moving given by load case 50.

Fig. 35
Principal stresses for hydrostatic load testing fault X with fault Elia, overloaded given by load case 55.

**Fig. 36**

- **Tension (Kg/cm²)**
- **Compression (Kg/cm²)**
Total crown cantilever radial displacements for uplift conditions on fault Eloisa with fault X prevented from moving given by load case 60.
Total radial displacements at levels 817 and 867 for uplift conditions on fault Eloisa with fault X prevented from moving given by load case 60.

**FIG. 38**
Total hoop stresses at levels 817 and 867 for uplift conditions on fault Eloisa given by load case 60.

**FIG. 39**
Total crown cantilever hoop and vertical stresses for uplift conditions on fault Eloisa given by load case 60.

**FIG. 40**
Principal stresses for full load without uplift on fault.

Elsia given by load case 60.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

- Tension (Kg/cm²)
- Compression (Kg/cm²)
Principal stresses for full load with 100% uplift on foundation and 100% uplift on foundation.

**UPSTREAM FACE**

Viewed from upstream

**DOWNSTREAM FACE**

Viewed from upstream

Tension (Kg/cm²)

Compression (Kg/cm²)
Total crown cantilever radial displacements for uplift conditions on fault X given by load case 59.

**FIG. 43**
Total radial displacements at levels 317 and 867 for uplift conditions on fault X given by load case 59.

**FIG. 44**
Total hoop stresses at levels 817 and 867 for uplift conditions on fault X given by load case 59.

**FIG. 45**
Total crown cantilever hoop and vertical stresses for uplift conditions on fault X given by load case 59.

**FIG. 46**
UPSTREAM FACE
Viewed from upstream

DOWNSTREAM FACE
Viewed from upstream

Principal stresses for full load without uplift on fault X
given by load case 59

Tension (Kg/cm²)
Compression (Kg/cm²)
Principal stresses for full load with 100% uplift on fault X given by load case 59.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

- **Tension (Kg/cm²)**
- **Compression (Kg/cm²)**
Total radial displacements for load factors of 1.0-1.3 with uplift on fault X given by load case 61.

**FIG. 49**
Total load — total displacement history of selected gauge points for load case 61.

FIG. 50
Total radial displacements for load factors of 1.0-1.7 with uplift on fault X given by load case 62.

**FIG. 51**
Total load - total displacement history of selected gauge points for load case C2.

FIG. 52
**Total radial displacements for load factors of 1.6-2.0 with uplift on fault X given by load case 63.**

**FIG. 53**
Total load - total displacement history of selected gauge points for load case 63.

**FIG. 54**
Total radial displacements for hydrostatic load factors of 2.0–2.4 with a constant gravity load factor of 2.0 given by load case 65.

FIG. 55
Total load - total displacement history of selected gauge points for load case 65.

FIG. 56
Total hoop stresses at levels 817 and 867 given by load case 65.

**Fig. 57**
Total crown cantilever hoop and vertical stresses given by load case 65.

FIG. 58
Total load-total strain history of selected strain gauges for load case 65.

Fig. 59
Principal stresses for full load with 100% uplift on foundation and fault X given by load case 65.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

+ [TENSION (Kg/cm²)]
- [COMPRESSION (Kg/cm²)]
Principal stresses for 2.0 x full load with 100% uplift, given by load case 0.5.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

TENSION (Kg/cm²) →
COMPRESSION (Kg/cm²) ←
Principal stresses for 20x full load with 100% uplift, except factor of 2.4 given by load case 65.

UPSTREAM FACE
Viewed from upstream

DOWNSTREAM FACE
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
Total radial displacements for first failure given by load case 66.

FIG. 63
Total load - total displacement history of selected gauge points on dam for first failure given by load case 66.

**FIG. 64**
Load factor history for first failure with fault X loading system

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</table>

Total load - total displacement history of selected gauge points on valley for first failure given by load case 66.

**FIG. 65**
Refer to Fig. 65 for loading conditions

- **Loading Condition A**
- **Loading Condition B**
- **Loading Condition E**
- **Loading Condition J**

**Upstream Face**

**Downstream Face**

**Tension**

**Compression**

**Refer to Fig. 65 for loading conditions**

- **Loading Condition K**
- **Loading Condition M**
- **Loading Condition N**
- **Loading Condition P**
- **Residual Stresses**

**Upstream Face**

**Downstream Face**

**Tension**

**Compression**

**Total hoop stresses at level 867 for first failure given by load case 56.**

*Fig. 66*
Total hoop stresses at level 817 for first failure given by load case 66.

**FIG. 67**
Total crown cantilever hoop stresses for first failure given by load case 66.
Total crown cantilever vertical stresses for first failure given by load case 66.

Fig. 69
Total load-total strain history of selected strain gauges for first failure given by load case 66.

**FIG. 70**
Principal stresses for loading condition A (refer to FIG. 65).

UPSTREAM FACE
Viewed from upstream

DOWNSTREAM FACE
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
Principal stresses for loading condition B (refer to Fig. 66).

UPSTREAM FACE
Viewed from upstream

DOWNSTREAM FACE
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
Principal stresses for loading condition J (refer to Fig. 66)
given by load case 66.

UPSTREAM FACE
Viewed from upstream

- TENSION (Kg/cm²)
- COMPRESSION (Kg/cm²)

DOWNSTREAM FACE
Viewed from upstream
Principal stresses for loading condition K (refer to Fig. 66) given by load case 66.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

- **TENSION** (Kg/cm²)
- **COMPRESSION** (Kg/cm²)

[Diagram showing stress measurements on upstream and downstream faces]
Principal stresses for loading condition W (refer to Fig. 66) given by load case 66.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

+ ↔ TENSION (Kg/cm²)
- ➩ COMPRESSION (Kg/cm²)
Principal stresses for loading condition P (refer to Fig. 65) given by load case 6.

**UPSTREAM FACE**

Viewed from upstream

**DOWNSTREAM FACE**

Viewed from upstream

TENSION (Kg/cm²) →

COMPRESSION (Kg/cm²) →
Residual principal stresses after first failure given by load case 66.

UPSTREAM FACE
Viewed from upstream

DOWNSTREAM FACE
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
TOTAL RADIAL DISPLACEMENTS FOR LEVEL 867 (CREST)

NO RESIDUAL DISPLACEMENTS OBTAINED

265

0 50 100 150 200 250 300 350 400 430

Dam radial displacements (mm)

0 50 100 150 200 250 300 350 400 430

Total radial displacements at level 867 and total load – total displacement history of selected gauge points on dam for second failure given by load case 67.

FIG. 78
Total hoop stresses at levels 817 and 867 for second failure given by load case 67.

FIG. 79
Total crown cantilever hoop and vertical stresses for second failure given by load case 67.

FIG. 80
Total load-total strain history of selected strain gauges for second failure given by load case 67.

**FIG. 81**
Principal stresses for full load with 100% uplift on foundation and fault X given by load case 87.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
Principal stresses for 2.0 x full load with 100% uplift on foundation, given by load case 2.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
Principal stresses for 20 x full load with 100% uplift on foundation, and hydrostatic load factor of 2.9 given by load case 67.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

**TENSION (Kg/cm²)**

**COMPRESSION (Kg/cm²)**
Principal stresses for full load with 100% uplift on foundation and fault X with hydrostatic load factor of 2.5, given by load case 67.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
Residual principal stresses after second failure given by load case 87.

**UPSTREAM FACE**
Viewed from upstream

**DOWNSTREAM FACE**
Viewed from upstream

- TENSION (Kg/cm²)
- COMPRESSION (Kg/cm²)
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<th>UNITS</th>
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<th>MODEL 2</th>
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<tr>
<td>Ultimate Strength</td>
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<td>Poisson's Ratio</td>
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PROPERTIES OF DAM, SPILLWAY, SHEAR KEYS, RING BEAM AND STILLING BASIN

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<th>MATERIAL</th>
<th>LOCATION</th>
<th>CRUSHING STRENGTH Kg/cm²</th>
<th>YOUNG'S MODULUS x10⁻³ Kg/cm²</th>
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<td>MODEL 2</td>
<td>SPECIFIED</td>
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<tr>
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<td>Above Level 830</td>
<td>70-85</td>
<td>63</td>
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<td>G</td>
<td>Between Levels 790 and 830</td>
<td>70-100</td>
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<td>P</td>
<td>Below Level 790</td>
<td>100-140</td>
<td>170</td>
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PROPERTIES OF VALLEY

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<tr>
<th>FRICTION ANGLE</th>
<th>SPECIFIED COEFFICIENT OF FRICTION</th>
<th>MODEL 2 COEFFICIENT OF FRICTION</th>
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<td>35°</td>
<td>0.700</td>
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PROPERTIES OF JOINTS

MATERIAL PROPERTIES
LOCATION OF DISPLACEMENT TRANSDUCERS ON DOWNSTREAM FACE OF DAM

The displacement transducers are radial to the downstream face.
The position of each transducer is only approximate as the strain gauge rosettes were located at the given positions and the transducers were then placed as close as possible to the centres of the rosette.
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<th>Y</th>
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<table>
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<td>345</td>
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<td>3692</td>
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</table>

CO-ORDINATES OF THE POINTS AT WHICH TRANSDUCERS WERE LOCATED ON THE VALLEY

FIG. 91
* Transducers which were found to be faulty after the dam had been tested to failure.
LOCATION OF HYDRAULIC JACKS FOR APPLYING GRAVITY LOAD

KEY
- Jack positions for gravity loading on the valley, abutments and crest of dam.
- Transducers
### SUMMARY OF LOAD CASES 1-24

**DATE OF TEST**

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<th>22-3-73</th>
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<th>12-3-73</th>
<th>19-3-73</th>
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**LOAD FACTOR ON GRAVITY AND HYDROSTATIC LOAD**

| Load | 1   | 0.2  | 0.1  | 0.1  | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   | 0.1   |

**SUMMARY OF LOAD CASES**

- **Case 1:**
  - 19-2-73: X
  - 9-3-73: X
  - 22-3-73: X
  - 9-3-73: X
  - 12-3-73: X
  - 19-3-73: X
  - 20-3-73: X
  - 22-3-73: X
  - 23-3-73: X
  - 24-3-73: X

**FIG. 93**
<table>
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<td>X X X</td>
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FIG.24
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LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE DOWNSTREAM FACE DURING THE PRELIMINARY TESTS, LOAD CASES 1-33.

FIG. 96
LOAD FACTOR–STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE DOWNSTREAM FACE DURING THE PRELIMINARY TESTS, LOAD CASES 1–33

FIG. 97
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE DOWNSTREAM FACE DURING THE PRELIMINARY TESTS, LOAD CASES 1-33

FIG. 96
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE UPSTREAM FACE DURING THE PRELIMINARY TESTS, LOAD CASES 1–33

FIG. 99
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE UPSTREAM FACE DURING THE PRELIMINARY TESTS, LOAD CASES 1-33

FIG. 100
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS GAUGES ON THE RIGHT BANK SIDE OF THE DOWNSTREAM FACE DURING THE PRELIMINARY TESTS, LOAD CASES 1-33

FIG. 102
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS GAUGES ON THE RIGHT BANK SIDE OF THE UPSTREAM FACE DURING THE PRELIMINARY TESTS LOAD CASES 1-33

FIG.103
FIG. 104

HOOP AND VERTICAL STRESSES FOR GRAVITY LOAD
Obtained from Load Case 28
FIG. 105

PRINCIPAL STRESSES FOR GRAVITY LOAD
obtained from Load Case 26.

UPSTREAM FACE
Viewed from Upstream

DOWNSTREAM FACE
Viewed from Upstream

Tension (Kg/cm²)
Compression (Kg/cm²)
HOOP STRESSES
AT LEVEL 867

HOOP STRESSES
AT LEVEL 842

HOOP STRESSES
AT LEVEL 817

HOOP STRESSES
AT LEVEL 792

867
842
817
792
782
758

LEVEL (m)

LEFT BANK ARCLENGTH (m) RIGHT BANK

LEFT BANK ARCLENGTH (m) RIGHT BANK

VERTICAL STRESSES FOR LEFT BANK CANTILEVER

VERTICAL STRESSES FOR LEFT BANK CANTILEVER

VERTICAL STRESSES FOR CROWN CANTILEVER

VERTICAL STRESSES FOR CROWN CANTILEVER

VERTICAL STRESSES FOR RIGHT BANK CANTILEVER

VERTICAL STRESSES FOR RIGHT BANK CANTILEVER

VERTICAL STRESS (Kg/cm²)

VERTICAL STRESS (Kg/cm²)

VERTICAL STRESS (Kg/cm²)

VERTICAL STRESS (Kg/cm²)

VERTICAL STRESS (Kg/cm²)

VERTICAL STRESS (Kg/cm²)

HOOP STRESS (Kg/cm²)

HOOP STRESS (Kg/cm²)

HOOP STRESS (Kg/cm²)

HOOP STRESS (Kg/cm²)

HOOP STRESS (Kg/cm²)

HOOP STRESS (Kg/cm²)

HOOP AND VERTICAL STRESSES FOR HYDROSTATIC LOAD

Obtained from Load Case 28

FIG. 106
LOAD FACTOR - STRAIN GAUGE CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE DOWNSTREAM FACE DURING TESTS FOR FAILURE BY CRUSHING LOAD CASES 33 - 47

FIG. 109
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT RANK SIDE OF THE UPSTREAM FACE DURING TESTS FOR FAILURE BY CRUSHING LOAD CASES 33-47

FIG. 110
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS ROSETTES ON THE LEFT BANK SIDE OF THE UPSTREAM FACE DURING TESTS FOR FAILURE BY CRUSHING LOAD CASES 33-47.

FIG. 111
LOAD FACTOR - STRAIN GAUGE CHANNEL READING CHARACTERISTICS FOR VARIOUS GAUGES ON THE RIGHT BANK SIDE OF THE UPSTREAM FACE DURING TESTS FOR FAILURE BY CRUSING LOAD CASES 33-47

FIG. 115
HOOP STRESSES AT LEVELS 867 AND 842 FOR VARIOUS LOAD FACTORS
Obtained during Tests for Failure by Crushing, Load Cases 33-47

FIG. 116
HOOP STRESSES AT LEVELS 817 AND 792 FOR VARIOUS LOAD FACTORS
Obtained during Tests for Failure by Crushing, Load Cases 33-47
FIG. 117
LEFT BANK CANTILEVER HOOP STRESSES (Kg/cm²)

LEFT BANK CANTILEVER HOOP STRESSES (Kg/cm²)

LOAD FACTOR = 1.0 : LOAD CASE 34

LOAD FACTOR = 1.25 : LOAD CASE 36

LOAD FACTOR = 1.5 : LOAD CASE 39

LOAD FACTOR = 1.75 : LOAD CASE 45

LOAD FACTOR = 2.0 : LOAD CASE 47

LEFT BANK CANTILEVER HOOP AND VERTICAL STRESSES FOR VARIOUS LOAD FACTORS

Obtained during Tests for Failure by Crushing, Load Cases 33–47

FIG. 118
CROWN CANTILEVER HOOP STRESSES (Kg/cm²)  
CROWN CANTILEVER VERTICAL STRESSES (Kg/cm²)  

Load Factor = 1.0: Load Case 34  
Load Factor = 1.25: Load Case 38  
Load Factor = 1.5: Load Case 39  
Load Factor = 1.75: Load Case 45  
Load Factor = 2.0: Load Case 47  

CROWN CANTILEVER HOOP AND VERTICAL STRESSES  
FOR VARIOUS LOAD FACTORS  
Obtained during Tests for Failure by Crushing, Load Cases 33–47  
FIG. 119
RIGHT BANK CANTILEVER HOOP AND VERTICAL STRESSES FOR VARIOUS LOAD FACTORS
Obtained during Tests for Failure by Crushing, Load Cases 33-47

FIG. 120
PRINCIPAL STRESSES FOR LOAD FACTOR = 2.0

Obtained from Load Code 47 during Tests 47.

Figures 123 & 124 refer to a different scale.

UPSTREAM FACE
Viewed from Upstream

DOWNSTREAM FACE
Viewed from Upstream

Tension (Kg/cm²)

Compression (Kg/cm²)
HOOP STRESSES AT LEVELS 867 AND 842 FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2.0
Obtained during Tests for Failure by Sliding, Load Cases 48-56
FIG. 124
HOOP STRESSES AT LEVELS 817 AND 792 FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2.0

Obtained during Tests for Failure by Sliding, Load Cases 48-56

FIG. 125
LEFT BANK CANTILEVER HOOP AND VERTICAL STRESSES FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2.0

Obtained during Tests for Failure by Sliding, Load Cases 48-56

FIG. 126
CROWN CANTILEVER HOOP AND VERTICAL STRESSES FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2-0

Obtained during Tests for Failure by Sliding, Load Cases 48-56

FIG. 127
RIGHT BANK CANTILEVER HOOP STRESSES (Kg/cm²)  

RIGHT BANK CANTILEVER VERTICAL STRESSES (Kg/cm²)  

2·0 x GRAVITY + 2·0 x HYDROSTATIC : LOAD CASE 48  

2·0 x GRAVITY + 2·5 x HYDROSTATIC : LOAD CASE 49  

2·0 x GRAVITY + 3·0 x HYDROSTATIC : LOAD CASE 51  

2·0 x GRAVITY + 3·5 x HYDROSTATIC : LOAD CASE 55  

2·0 x GRAVITY + 4·0 x HYDROSTATIC : LOAD CASE 55  

RIGHT BANK CANTILEVER HOOP AND VERTICAL STRESSES FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2·0  

Obtained during Tests for Failure by Sliding, Load Cases 48-56  

FIG. 128
PRINCIPAL STRESSES FOR 2.0 x GRAVITY + 2.0 x HYDROSTATIC
Obtained from Load Case 48 during Tests for Failure by Sliding, Load Cases 48-56

UPSTREAM FACE
Viewed from Upstream

DOWNSTREAM FACE
Viewed from Upstream

Tension (Kg/cm²)
Compression (Kg/cm²)
PRINCIPAL STRESSES FOR 2.0 x GRAVITY + 3.0 x HYDROSTATIC

Obtained from Load Case 51 during Trials for Failure by Slaming, Load Cases 46-50.

FIG. 130

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
PRINCIPAL STRESSES FOR 2.0 × GRAVITY + 4.0 × HYDROSTATIC
Obtained from Load Case 55 during Tests for Failure by Sliding, Load Cases 45–56.

FIG. 131

Tension (Kg/cm²)
Compression (Kg/cm²)
RADIAL DISPLACEMENT OF HORIZONTAL ARCHES FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2·0

Obtained during Tests for Failure by Sliding, Load Cases 48-56

FIG. 132
RADIAL DISPLACEMENTS OF LEFT BANK CANTILEVER (mm)

RADIAL DISPLACEMENTS OF CROWN CANTILEVER (mm)

RADIAL DISPLACEMENTS OF RIGHT BANK CANTILEVER (mm)

<table>
<thead>
<tr>
<th>Load</th>
<th>Load Case</th>
<th>Gravity Load Factor</th>
<th>Hydrostatic Load Factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>48</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>49</td>
<td>2.0</td>
<td>2.5</td>
</tr>
<tr>
<td>C</td>
<td>51</td>
<td>2.0</td>
<td>3.0</td>
</tr>
<tr>
<td>D</td>
<td>55</td>
<td>2.0</td>
<td>3.5</td>
</tr>
<tr>
<td>E</td>
<td>55</td>
<td>2.0</td>
<td>4.0</td>
</tr>
</tbody>
</table>

RADIAL DISPLACEMENTS OF LEFT BANK, RIGHT BANK AND CROWN CANTILEVERS FOR VARIOUS HYDROSTATIC LOAD FACTORS WITH A GRAVITY LOAD FACTOR OF 2.0

Obtained during Tests for Failure by Sliding, Load Cases 4B-56

FIG. 133
HOOP STRESSES AT LEVELS 867 AND 842
Resulting from an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5
FIG. 134
HOOP STRESSES AT LEVELS 817 AND 792

Resulting from an attempt to induce a sliding failure by reducing the gravity load factor from 1·75 to 1·0 with a hydrostatic load factor of 3·5

FIG. 135
LEFT BANK CANTILEVER HOOP AND VERTICAL STRESSES

Resulting from an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5.
CROWN CANTILEVER HOOP STRESSES (Kg/cm²)

CROWN CANTILEVER VERTICAL STRESSES (Kg/cm²)

1.75 • GRAVITY • 1.75 • HYDROSTATIC : LOAD CASE 52

1.75 • GRAVITY • 2.5 • HYDROSTATIC : LOAD CASE 52

1.75 • GRAVITY • 3.0 • HYDROSTATIC : LOAD CASE 52

1.75 • GRAVITY • 3.5 • HYDROSTATIC : LOAD CASE 52

1.0 • GRAVITY • 3.5 • HYDROSTATIC : LOAD CASE 54

CROWN CANTILEVER HOOP AND VERTICAL STRESSES

Resulting from an attempt to induce a sliding failure
by reducing the gravity load factor from 1.75 to 1.0
with a hydrostatic load factor of 3.5

FIG. 137
RIGHT BANK CANTILEVER HOOP STRESSES (Kg/cm²)

1-75 • GRAVITY + 1-75 • HYDROSTATIC : LOAD CASE 52

1-75 • GRAVITY + 2-5 • HYDROSTATIC : LOAD CASE 52

1-75 • GRAVITY + 3-0 • HYDROSTATIC : LOAD CASE 52

1-75 • GRAVITY + 3-5 • HYDROSTATIC : LOAD CASE 54

1-0 • GRAVITY + 3-5 • HYDROSTATIC : LOAD CASE 54

RIGHT BANK CANTILEVER HOOP AND VERTICAL STRESSES
Resulting from an attempt to induce a sliding failure by reducing the gravity load factor from 1-75 to 1-0 with a hydrostatic load factor of 3-5

FIG.138
PRINCIPAL STRESSES FOR 1.75 × GRAVITY + 1.75 × HYDROSTATIC

Obtained from Load Case 52 during an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5.

FIG. 139
PRINCIPAL STRESSES FOR 1.75 x GRAVITY + 3.5 x HYDROSTATIC

Obtained from Load Case 54 during an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5.

FIG. 140
PRINCIPAL STRESSES FOR 1.0 × GRAVITY + 3.5 × HYDROSTATIC

Obtained from Load Case 54 during an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5.

FIG. 141
RADIAL DISPLACEMENTS OF HORIZONTAL ARCHES

Resulting from an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5

FIG. 142
RADIAL DISPLACEMENTS OF LEFT BANK, RIGHT BANK AND CROWN CANTILEVERS

Resulting from an attempt to induce a sliding failure by reducing the gravity load factor from 1.75 to 1.0 with a hydrostatic load factor of 3.5

FIG. 143
HOOP STRESSES AT LEVELS 867 AND 842
Obtained from Load Case 56 in which a sliding failure was induced by reducing
the gravity load factor below 1.0 with a hydrostatic load factor of 4.0

FIG. 144
Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0
LEFT BANK CANTILEVER HOOP STRESSES (Kg/cm²)

LEFT BANK CANTILEVER VERTICAL STRESSES (Kg/cm²)

2-0 GRAVITY + 2-0 HYDROSTATIC : LOAD CASE 56

2-0 GRAVITY + 4-0 HYDROSTATIC : LOAD CASE 56

1-5 GRAVITY + 4-0 HYDROSTATIC : LOAD CASE 56

1-0 GRAVITY + 4-0 HYDROSTATIC : LOAD CASE 56

1-0 GRAVITY ON DAM + 0-8 GRAVITY ON VALLEY + 3-6 HYDROSTATIC : LOAD CASE 56

LEFT BANK CANTILEVER HOOP AND VERTICAL STRESSES

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1-0 with a hydrostatic load factor of 4-0

FIG. 146
CROWN CANTILEVER HOOP AND VERTICAL STRESSES

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0

FIG. 147
RIGHT BANK CANTILEVER HOOP STRESSES (Kg/cm²)

RIGHT BANK CANTILEVER VERTICAL STRESSES (Kg/cm²)

RIGHT BANK CANTILEVER HOOP AND VERTICAL STRESSES

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0.
PRINCIPAL STRESSES FOR 2.0xGRAVITY + 2.0xHYDROSTATIC

Obtained from Load Case 95 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0.

FIG. 149

Tension (Kg/cm²)
Compression (Kg/cm²)
PRINCIPAL STRESSES FOR 2.0 x GRAVITY + 4.0 x HYDROSTATIC

Obtained from Load Case 56 in which a sliding failure
was induced by reducing the gravity load factor
below 1.0 with a hydrostatic load factor of 4.0.

FIG. 150

Tension (Kg/cm²)
Compression (Kg/cm²)
PRINCIPAL STRESSES FOR 1.5 x GRAVITY + 4.0 x HYDROSTATIC
Obtained from Load Case 56 in which a sliding failure
was induced by reducing the gravity load factor
below 1.0 with a hydrostatic load factor of 4.0.

FIG. 151
Obtained from Load Case 56 in which a sliding failure below 1/0 with a hydrostatic load factor of 4.0.

Principal Stresses for 1.0 x Gravity + 4.0 x Hydrostatic was induced by reducing the gravity load factor to 1.0.

Tension (Kg/cm²) compression (Kg/cm²)
UPSTREAM FACE
Viewed from Upstream

Tension (Kg/cm²)
Compression (Kg/cm²)

PRINCIPAL STRESSES FOR [1.8 x GRAVITY ON DAM] + 3.6 x HYDROSTATIC

Obtained from load case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydraulic load factor of 4.0.

Fig. 153
RADIAL DISPLACEMENTS OF HORIZONTAL ARCHES

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0

FIG. 154
RADIAL DISPLACEMENTS OF LEFT BANK CANTILEVER (mm)

RADIAL DISPLACEMENTS OF CROWN CANTILEVER (mm)

RADIAL DISPLACEMENTS OF RIGHT BANK CANTILEVER (mm)

<table>
<thead>
<tr>
<th>Load Case</th>
<th>Load Factor (Gravity)</th>
<th>Load Factor (Hydrostatic)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>56</td>
<td>2.0</td>
</tr>
<tr>
<td>B</td>
<td>56</td>
<td>2.0</td>
</tr>
<tr>
<td>C</td>
<td>56</td>
<td>1.5</td>
</tr>
<tr>
<td>D</td>
<td>56</td>
<td>1.0</td>
</tr>
<tr>
<td>E</td>
<td>56</td>
<td>1.0 Dam</td>
</tr>
</tbody>
</table>

RADIAL DISPLACEMENTS OF LEFT BANK, RIGHT BANK AND CROWN CANTILEVERS

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0

FIG.155
**Movement of Abutments and of Valley Immediately Downstream of Dam**

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0.
Gravity - Hydrostatic Load Factor

A 2.0 2.0
B 2.0 4.0
C 1.5 4.0
C1 1.25 4.0
C2 1.0 4.0
D 1.0 4.0
E 1.0 4.0

Remarks:
- Increase hydrostatic load factor
- Reduce gravity
- After 13 minutes
- Failure occurring

Transducer No. 332 - T. 332

**Fig. 1:**

**Left Bank:**

**Right Bank:**

**Upstream - Downstream Movement of Valley**

Obtained from Load Case 56 in which a sliding failure was induced by reducing the gravity load factor below 1.0 with a hydrostatic load factor of 4.0

**Fig. 157**
LOCATION OF CRACKS IN DAM AFTER FAILURE

UPSTREAM FACE
Viewed from Upstream

DOWNSTREAM FACE
Viewed from Upstream
Cracked both sides of socket

Wall 152.5 cm. from £

Fault 16

Socket

Wall 172.5 cm. from £

Back of Valley

LOCATION OF CRACKS IN VALLEY AFTER FAILURE

Stilling Basin

Left Abutment

Right Abutment

Crack

Loose Blocks

Rear of Crest

FIG. 159
Declaration of Blank Common - COMMON ELPA(900)
Declaration of NLPZ - NLPZ = 900

Common Subroutines

CALL OVERLAY(1,0)
CALL OVERLAY(2,0)
CALL OVERLAY(3,0)
CALL OVERLAY(4,0)
CALL OVERLAY(5,0)
CALL OVERLAY(6,0)

Is Dummy Overlay Required? yes
no

Is there another problem?

Start

Stop

End

OVERLAY (0,0) - MAIN PROGRAM

OVERLAY (1,0) - DATA PROCESSING 1
Initialisation
Structural Type
Options
Nodal Point Coordinates and Generation
Elements and Element Generation

OVERLAY (2,0) - DATA PROCESSING 2
Elastic Property Types
Elastic Properties of Elements
Thermal Properties of Elements
Support Conditions

OVERLAY (3,0) - DATA PROCESSING 3
Load Types - surface pressures, concentrated loads, constant body forces, body force potentials, uniformly distributed loads
Loading
Combined Load Cases
Data Summary
Selection of Method of Solution

OVERLAY (4,0) - PRE-SOLUTION PROCESS
Element Stiffness Matrices - Tape 1
Individual Element Information - Tape 2
Stress Matrix for Each Node of an Element - Tape 2
Combined Load Case Data - Tape 3

OVERLAY (5,0) - SOLUTION
OVERLAY (5,1) - CHOLESKI SOLUTION
OVERLAY (5,2) - FRONTAL SOLUTION
Displacements - Tape 3

OVERLAY (6,0) - POST-SOLUTION PROCESS
Calculation of displacements for combined load cases, nodal point stresses, reactions to earth
Output of displacements
reactions to earth
stresses for each element
average nodal point stresses
element results - Tape 7

OVERLAY (7,0) - DUMMY OVERLAY
Element results on Tape 7 are available for processing according to user's own requirements

OVERLAYING OF COMPUTER PROGRAM FOR CDC 6400/6600/7600 COMPUTERS

FIG. 160
**ELPA/NLPA During Data Processing**

- **NIZZ**
- **NLPFO**
- **NLPLO**
- **NLPBO**
- **NLPTO**
- **NLPPO**
- **NLPEO**
- **NLPCO**
- **NLPZ**

**String of Element Nodal Point Numbers**
**Combined Load Cases**
**Load Cases**
**Support Conditions**
**Element Thermal Properties**
**Element Elastic Properties**
**Element Numbers**
**Nodal Point Coordinates**

---

**ELPA/NLPA During Pre-Solution**

- **NIZZ**
- **NLPFO**
- **NLPLO**
- **NLPBO**
- **NLPTO**
- **NLPPO**
- **NLPEO**
- **NLPCO**
- **NLPZ**

**String of Element Nodal Point Numbers**
**Combined Load Cases**
**Load Cases**
**Support Conditions**
**Element Thermal Properties**
**Element Elastic Properties**
**Element Numbers**
**Nodal Point Coordinates**

---

**Individual Element Information**

- **LSGMAX**
- **MNODZ**

**Set 1 of Equivalenced Variables**
**Element Coordinates**
**Element Elastic Properties**
**Element Thermal Properties**
**Element Support Conditions**
**Element Loading**
**Element Stiffness Matrix**

**Set 2 of Equivalenced Variables**
**Element Nodal Points and Destinations**
**Element Support Condition Coding**
**Element R.H.S.**

---

**Set 1 of Equivalenced Variables**

- **NELT**
- **NPR**
- **NPA**
- **NPRZ**
- **NTHERM**
- **J7**
- **J8**
- **J9**
- **J10**
- **J11**
- **J12**
- **J13**
- **J14**
- **J15**
- **J16**

- **NELT** = Element type
- **NPR** = Element property type
- **NPA** = Number of locations required by element coordinates
- **NPRZ** = Number of elastic properties
- **NTHERM** = Number of thermal properties
- **J7-J16** = Labels in individual element information vector

**Set 2 of Equivalenced Variables**

- **NEL**
- **LNODZ**
- **NDF**
- **LPREQ**
- **LSEGZ**
- **J1**
- **J2**
- **J3**
- **J4**
- **J5**
- **J6**

- **NEL** = Element number
- **LNODZ** = Number of nodes for element
- **NDF** = Number of degrees of freedom for element
- **LPREQ** = Number of stored equations when previous element was assembled
- **LSEGZ** = Number of locations required by element stiffness matrix
- **J1-J6** = Labels in element record vector

---

**Legend:**

- **LSGMAX**
- **MNODZ**
- **MAXVAR**
- **(Element Record)**
ELPA/NLPA DURING CHOLESKI SOLUTION

<table>
<thead>
<tr>
<th>NODES</th>
<th>RREC/IREC</th>
<th>ELPA</th>
<th>STIFF</th>
<th>RHS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nodal Point Numbers in Numerical Order</td>
<td>Element Record</td>
<td>Element Stiffness Matrix</td>
<td>Overall Stiffness Matrix</td>
<td>Overall R.H. Sides (current load cases)</td>
</tr>
</tbody>
</table>

MAXNIC  MAXVAR  MAXELT  NSTIFF  MAXNDF x NRESOL

ELPA/NLPA DURING FRONTAL SOLUTION

a) ELIMINATION PHASE FOR FIRST SOLUTION

<table>
<thead>
<tr>
<th>NODES</th>
<th>RREC/IREC</th>
<th>ELPA</th>
<th>STIFF</th>
<th>RHS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nodal Point Numbers in Numerical Order</td>
<td>Element Record</td>
<td>Element Stiffness Matrix</td>
<td>Overall Stiffness Matrix</td>
<td>Overall R.H. Sides (current load cases)</td>
</tr>
</tbody>
</table>

MAXNIC  MAXVAR  MAXELT  NSTIFF  MAXNDF x NRESOL

b) ELIMINATION PHASE FOR RE-SOLUTION

<table>
<thead>
<tr>
<th>NODES</th>
<th>RREC/IREC</th>
<th>ELPA</th>
<th>STIFF</th>
<th>RHS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nodal Point Numbers in Numerical Order</td>
<td>Element Record</td>
<td>Element Stiffness Matrix</td>
<td>Overall Stiffness Matrix</td>
<td>Overall R.H. Sides (current load cases)</td>
</tr>
</tbody>
</table>

MAXNIC  MAXVAR  MAXELT  NSTIFF  MAXNDF x NRESOL

RREC/IREC  RREC/IREC  EQU  EQU

Element Record + Equation Coefficients  Element Record + Equation Coefficients

4  MAXVAR  MAXELT

L1

NWRITE  MNODS  NGRAND  MAXPA x (NSOLVE + 1)

NWRITE  MNODS  NRESOL x MAXPA

NWRITE  NRESOL x MAXNDF  NRESOL x MAXPA

3 4 5 6

c) BACK-SUBSTITUTION PHASE

<table>
<thead>
<tr>
<th>NODES</th>
<th>RREC/IREC</th>
<th>ELPA</th>
<th>STIFF</th>
<th>RHS</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nodal Point Numbers in Numerical Order</td>
<td>Element Record</td>
<td>Element Stiffness Matrix</td>
<td>Overall Stiffness Matrix</td>
<td>Overall R.H. Sides (current load cases)</td>
</tr>
</tbody>
</table>

MAXNIC  MAXVAR  MAXELT  NSTIFF  MAXNDF x NRESOL

RREC/IREC  RREC/IREC  EQU  EQU

Element Record + Equation Coefficients  Element Record + Equation Coefficients

4  MAXVAR  MAXELT

L1

NWRITE  MNODS  NGRAND  MAXPA x (NSOLVE + 1)

NWRITE  MNODS  NRESOL x MAXPA

NWRITE  NRESOL x MAXNDF  NRESOL x MAXPA

3 4 5 6

FIG. 162
ELPA/NLPA DURING POST-SOLUTION PHASE

a) DURING CALCULATION OF COMBINED DISPLACEMENT VECTOR

<table>
<thead>
<tr>
<th>RCOMB / ICOMB</th>
<th>DISPV</th>
<th>DISPC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Combined Load Cases</td>
<td>Solution/Displacement Vector</td>
<td>Solution/Combined Displacement Vector</td>
</tr>
</tbody>
</table>

- N1: MAXNDF
- N2: N3: MAXNDF
- NLPZ:

b) DURING CALCULATION OF STRESSES, REACTIONS TO EARTH AND AVERAGE NODAL POINT STRESSES

<table>
<thead>
<tr>
<th>MNODS</th>
<th>ELPA/NLPA</th>
<th>RHS</th>
<th>DISP</th>
<th>DB</th>
<th>STRESS</th>
<th>NREACT</th>
<th>REACT</th>
<th>AVSTR</th>
<th>NLPZ</th>
</tr>
</thead>
<tbody>
<tr>
<td>Current Active Nodes</td>
<td>Individual Element Information</td>
<td>Solution Displacement Vector</td>
<td>Displacements for Element</td>
<td>Element Stress Matrix</td>
<td>Stresses for Element</td>
<td>Nodal Pt. Nos. with Supports</td>
<td>Reactions to Earth for Support Nodes</td>
<td>Average Nodal Point Stresses</td>
<td></td>
</tr>
</tbody>
</table>

- MNODZ = Maximum size of an element record
- LSGMAX = Maximum size of Individual element information
- MAXELT = Length available for storing element stiffness matrix during solution (matrix may be in segmented form)
- MAXNDF = Maximum nodal point number * Number of degrees of freedom per node = MAXNIC * NODVZ
- MAXNIC = Maximum nodal point number
- MAXPA = Maximum number of degrees of freedom in wavefront = MNODZ * NODVZ
- MAXVAR = Maximum size of a stress matrix
- M6 = Maximum size required for storing the stresses at all the nodal points defining an element
- M12 = Maximum number of stresses per node when average nodal point stresses are required, or M12=1 and M11=M10 when they are not required
- M13 = Maximum nodal point number (MAXNIC) when average nodal point stresses are required, or M13=1 and M11=M10 when they are not required
- NBCNDZ = Number of nodal points with support conditions
- NDFFTOT = Total number of degrees of freedom
- NGRAND = Maximum size of the current overall stiffness matrix (Grandpa’s L.H.S.) for frontal solution = MAXPA(MAXPA+1)/2
- NBWTOT = Bandwidth of overall stiffness matrix for Choleski solution
- NODVZ = Number of degrees of freedom per node
- NRESOL = Maximum size of the current overall stiffness matrix for frontal solution = MAXPA(MAXPA+1)/2
- NSOLVE = Number of load cases which can be processed during back-substitution (frontal and Choleski) and elimination for re-solution (frontal)
- NSTIFF = Locations required for in-core storage of the overall stiffness matrix for solution by Choleski method = NBWTOT(1+2 NDFFTOT - NBWTOT)/2
- NWRITE = Locations required for random access disc records

VARIABLE NAMES

- LSGMAX = Maximum size of Individual element information
- MAXELT = Length available for storing element stiffness matrix during solution (matrix may be in segmented form)
- MAXNDF = Maximum nodal point number * Number of degrees of freedom per node = MAXNIC * NODVZ
- MAXNIC = Maximum nodal point number
- MAXPA = Maximum number of degrees of freedom in wavefront = MNODZ * NODVZ
- MAXVAR = Maximum size of an element record
- MNODZ = Maximum number of active nodes in wavefront
- M6 = Maximum size of a stress matrix
- M12 = Maximum number of stresses per node when average nodal point stresses are required, or M12=1 and M11=M10 when they are not required
- M13 = Maximum nodal point number (MAXNIC) when average nodal point stresses are required, or M13=1 and M11=M10 when they are not required
- NBCNDZ = Number of nodal points with support conditions
- NBWTOT = Bandwidth of overall stiffness matrix for Choleski solution
- NDFFTOT = Total number of degrees of freedom
- NGRAND = Maximum size of the current overall stiffness matrix (Grandpa’s L.H.S.) for frontal solution = MAXPA(MAXPA+1)/2
- NODVZ = Number of degrees of freedom per node
- NRESOL = Maximum size of the current overall stiffness matrix for frontal solution = MAXPA(MAXPA+1)/2
- NSOLVE = Number of load cases which can be processed during back-substitution (frontal and Choleski) and elimination for re-solution (frontal)
- NSTIFF = Locations required for in-core storage of the overall stiffness matrix for solution by Choleski method = NBWTOT(1+2 NDFFTOT - NBWTOT)/2
- NWRITE = Locations required for random access disc records
HEXAHEDRONAL ELEMENT WITH STRAIGHT EDGES
Number of nodes 8
Degrees of freedom u,v,w at each node
Displacement field linear + selected higher order terms
References 27, 28
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

HEXAHEDRONAL ELEMENT WITH CURVED EDGES
Number of nodes 20
Degrees of freedom u,v,w at each node
Displacement field quadratic + selected higher order terms
References 27, 28
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

HEXAHEDRONAL ELEMENT WITH CURVED EDGES
Number of nodes 32
Degrees of freedom u,v,w at each node
Displacement field cubic + selected higher order terms
References 27, 28
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

PENTAHEDRONAL ELEMENT WITH STRAIGHT EDGES
Number of nodes 6
Degrees of freedom u,v,w at each node
Displacement field linear + selected higher order terms
References 27, 28
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

PENTAHEDRONAL ELEMENT WITH CURVED EDGES
Number of nodes 15
Degrees of freedom u,v,w at each node
Displacement field quadratic + selected higher order terms
References 27, 28
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

HEXAHEDRONAL ELEMENT WITH STRAIGHT EDGES
Number of nodes 8
Degrees of freedom u,v,w at each node; 24 compatible and 9 incompatible. The latter are eliminated.
Displacement field linear + selected higher order terms
References 29
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

PENTAHEDRONAL ELEMENT WITH STRAIGHT EDGES
Number of nodes 6
Degrees of freedom u,v,w at each node; 18 compatible and 6 incompatible; The latter are eliminated.
Displacement field linear + selected higher order terms
References -
Stress output $\sigma_i$, $\sigma_j$, $\tau_{ij}$ at each node
Coordinates $x, y, z$ at each node
Elastic properties isotropic

ELEMENTS AVAILABLE FOR ANALYSING
THREE-DIMENSIONAL SOLID PROBLEMS

FIG.164
<table>
<thead>
<tr>
<th>QUADRILATERAL JOINT ELEMENT</th>
<th>Number of nodes</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>u,v,w at each node</td>
<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>linear + selected higher order terms; same field used for both surfaces</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>16</td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>relative in-plane and normal displacements, shear and normal stresses, for each nodal pair</td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>x,y,z for each node (nodal pairs have the same coordinates)</td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>$k_x$, $k_z$ - strike and dip shear stiffness $k_y$ - normal stiffness</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>QUADRILATERAL JOINT ELEMENT</th>
<th>Number of nodes</th>
<th>16</th>
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</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>u,v,w at each node</td>
<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>quadratic + selected higher order terms; same field used for both surfaces</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>relative in-plane and normal displacements, shear and normal stresses, for each nodal pair</td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>x,y,z for each node (nodal pairs have the same coordinates)</td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>$k_x$, $k_z$ - strike and dip shear stiffness $k_y$ - normal stiffness</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>QUADRILATERAL JOINT ELEMENT</th>
<th>Number of nodes</th>
<th>24</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>u,v,w at each node</td>
<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>cubic + selected higher order terms; same field used for both surfaces</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>relative in-plane and normal displacements, shear and normal stresses, for each nodal pair</td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>x,y,z for each node (nodal pairs have the same coordinates)</td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>$k_x$, $k_z$ - strike and dip shear stiffness $k_y$ - normal stiffness</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>TRIANGULAR THIN SHELL ELEMENT</th>
<th>Number of nodes</th>
<th>3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>u,v,w,θ_1,θ_2 at each node</td>
<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>linear for membrane action, incomplete third order for bending action</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>27</td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>$N_N,N_B,M,M,M$, for each node</td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>x,y,z for each node</td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>QUADRILATERAL GENERAL SHELL ELEMENT</th>
<th>Number of nodes</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>u,v,w,α, β at each node</td>
<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>quadratic + selected higher order terms</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>19,20,27</td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma \tau_\alpha \tau_\beta \tau_\alpha \tau_\beta \tau_\alpha \tau_\beta$ local stresses for both surfaces, at each node</td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>x,y,z for top surface and x,y,z for bottom surface, for each node</td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>QUADRILATERAL GENERAL SHELL ELEMENT</th>
<th>Number of nodes</th>
<th>12</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>u,v,w,α, β at each node</td>
<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>cubic + selected higher order terms</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>19,20,27</td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma \tau_\alpha \tau_\beta \tau_\alpha \tau_\beta \tau_\alpha \tau_\beta$ local stresses for both surfaces, at each node</td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>x,y,z for top surface and x,y,z for bottom surface, for each node</td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
<td></td>
</tr>
</tbody>
</table>

Elements available for representing fault planes in rock masses, and elements available for analysing thick and thin shell problems.

Fig. 165
<table>
<thead>
<tr>
<th>ELEMENTS AVAILABLE FOR ANALYSING PLANE STRESS AND PLANE STRAIN PROBLEMS.</th>
</tr>
</thead>
</table>

**QUADRILATERAL PLANE MEMBRANE ELEMENT**

<table>
<thead>
<tr>
<th>Number of nodes</th>
<th>4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u,v$ at each node</td>
</tr>
<tr>
<td>Displacement field</td>
<td>linear + selected higher order terms</td>
</tr>
<tr>
<td>References</td>
<td>27, 28</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \sigma_{xx}, \sigma_{yy}, \tau_{yx}, \tau_{xy}$ at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ at each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
</tr>
</tbody>
</table>

**QUADRILATERAL PLANE MEMBRANE ELEMENT**

<table>
<thead>
<tr>
<th>Number of nodes</th>
<th>8</th>
</tr>
</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u,v$ at each node</td>
</tr>
<tr>
<td>Displacement field</td>
<td>quadratic + selected higher order terms</td>
</tr>
<tr>
<td>References</td>
<td>27, 28</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \sigma_{xx}, \sigma_{yy}, \tau_{yx}, \tau_{xy}$ at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ at each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
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</tbody>
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**QUADRILATERAL PLANE MEMBRANE ELEMENT**

<table>
<thead>
<tr>
<th>Number of nodes</th>
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<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u,v$ at each node</td>
</tr>
<tr>
<td>Displacement field</td>
<td>cubic + selected higher order terms</td>
</tr>
<tr>
<td>References</td>
<td>27, 28</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \sigma_{xx}, \sigma_{yy}, \tau_{yx}, \tau_{xy}$ at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ at each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
</tr>
</tbody>
</table>

**TRIANGULAR PLANE MEMBRANE ELEMENT**

<table>
<thead>
<tr>
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<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u,v$ at each node</td>
</tr>
<tr>
<td>Displacement field</td>
<td>quadratic</td>
</tr>
<tr>
<td>References</td>
<td>27, 28</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \sigma_{xx}, \sigma_{yy}, \tau_{yx}, \tau_{xy}$ at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ at each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
</tr>
</tbody>
</table>

**TRIANGULAR PLANE MEMBRANE ELEMENT**

<table>
<thead>
<tr>
<th>Number of nodes</th>
<th>10</th>
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</thead>
<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u,v$ at each node</td>
</tr>
<tr>
<td>Displacement field</td>
<td>cubic</td>
</tr>
<tr>
<td>References</td>
<td>27, 28</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \sigma_{xx}, \sigma_{yy}, \tau_{yx}, \tau_{xy}$ at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ at each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
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**QUADRILATERAL PLANE MEMBRANE ELEMENT**

<table>
<thead>
<tr>
<th>Number of nodes</th>
<th>4</th>
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<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u,v$ at each node</td>
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<tr>
<td>Displacement field</td>
<td>linear + selected higher order terms</td>
</tr>
<tr>
<td>References</td>
<td>29</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \sigma_{xx}, \sigma_{yy}, \tau_{yx}, \tau_{xy}$ at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ at each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic / anisotropic</td>
</tr>
<tr>
<td>QUADRILATERAL GENERAL PLATE ELEMENT</td>
<td>QDB8</td>
</tr>
<tr>
<td>-----------------------------------</td>
<td>------</td>
</tr>
<tr>
<td>Degrees of freedom</td>
<td>$u, v, w, \alpha, \beta$ at each node</td>
</tr>
<tr>
<td>Displacement field</td>
<td>quadratic + selected higher order terms</td>
</tr>
<tr>
<td>References</td>
<td>19, 20, 27</td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \tau_{xx}$ for both surfaces at each node</td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y, z$ for each node</td>
</tr>
<tr>
<td>Thickness</td>
<td>constant or thickness specified at each node</td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic/anisotropic</td>
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</tbody>
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<table>
<thead>
<tr>
<th>QUADRILATERAL GENERAL PLATE ELEMENT</th>
<th>QDB12</th>
<th>Number of nodes</th>
<th>12</th>
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<tbody>
<tr>
<td>Degrees of freedom</td>
<td>$u, v, w, \alpha, \beta$ at each node</td>
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</tr>
<tr>
<td>Displacement field</td>
<td>cubic + selected higher order terms</td>
<td></td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>19, 20, 27</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Stress output</td>
<td>$\sigma_x, \sigma_y, \tau_{xy}, \tau_{xx}$ for both surfaces at each node</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y, z$ for each node</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Thickness</td>
<td>constant or thickness specified at each node</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Elastic properties</td>
<td>isotropic/anisotropic</td>
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<table>
<thead>
<tr>
<th>TWO-DIMENSIONAL FIELD ELEMENT</th>
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<tr>
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<td>$0$ at each node</td>
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<td></td>
</tr>
<tr>
<td>Displacement field</td>
<td>linear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>27</td>
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<td></td>
</tr>
<tr>
<td>Output</td>
<td>$\delta x, \delta y$ which are constant throughout element</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Coordinates</td>
<td>$x, y$ for each node</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Properties</td>
<td>isotropic/anisotropic</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

ELEMENTS AVAILABLE FOR ANALYSING THIN AND THICK PLATE PROBLEMS, AND TWO-DIMENSIONAL FIELD PROBLEMS.

FIG. 167
FLOW CHART FOR THE ASKA ELEMENT GENERATION SCHEME ADOPTED FOR THE PROGRAM

FIG. 168
CENTRAL PROCESSING TIME FOR PRE-SOLUTION PROCESS

CENTRAL PROCESSING TIME FOR FIRST SOLUTION ELIMINATION OF FRONTAL SOLUTION

CENTRAL PROCESSING TIME FOR BACK-SUBSTITUTION PHASE OF FRONTAL SOLUTION

CDC 6400 CENTRAL PROCESSING TIMES FOR THE PRE-SOLUTION PROCESS AND FRONTAL SOLUTION
SIMPLE BENDING ACTION

EXACT DISPLACEMENTS

QDM4 DISPLACEMENTS

a) FAILURE OF THE QDM4 ELEMENT TO REPRESENT PURE BENDING.

\[ u = \beta_x (1 - \xi') \]

\[ v = \beta_z (1 - \eta') \]

b) INCOMPATIBLE DISPLACEMENT MODES FOR QDM16 ELEMENT.

ADDITION OF INCOMPATIBLE DISPLACEMENT MODES TO THE QDM4 ELEMENT TO FORM THE QDM14 ELEMENT

FIG. 170
TWO-DIMENSIONAL IDEALISATION OF EL ATAZAR DAM USING QDM14 FINITE ELEMENTS.

FIG.171
STRESSES IN THE LOWER REGIONS OF THE DAM OBTAINED FROM THE TWO-DIMENSIONAL F.E. ANALYSIS FOR GRAVITY + HYDROSTATIC LOAD ON THE DAM.

FIG. 172
DISPLACEMENT OF THE BASE AND DOWNSTREAM FACE OBTAINED FROM THE TWO-DIMENSIONAL F.E. ANALYSIS FOR GRAVITY + HYDROSTATIC LOAD ON THE DAM. FIG. 173
RADIAL DEVELOPMENT OF DAM
VIEWED FROM UPSTREAM

PROJECTION OF DAM AND SOCKET IN XZ-PLANE
VIEWED FROM UPSTREAM

LEVEL (m)

647
642
617
747
745
742
753
750
757
817
842
847
850
853
857
917
942
947
950
953
957

SCALE
0 20 40 60 m.
PRINCIPAL STRESSES FOR GRAVITY + HYDROSTATIC LOAD
FOR THE SHELL OF THE DAM RIGIDLY SUPPORTED AT ITS PERIPHERY.

FIG. 175
HOOP AND VERTICAL STRESSES FOR GRAVITY LOAD OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS.

FIG. 176
RADIAL DISPLACEMENTS FOR GRAVITY LOAD OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS. FIG. 177
HOOP STRESSES AT LEVEL 847

HOOP STRESSES AT LEVEL 792

HOOP STRESSES FOR CROWN CANTILEVER

VERTICAL STRESSES FOR CROWN CANTILEVER

KEY

TYPES OF ELEMENT

--- HEXO AND PEN5

---- HEX8 AND PEN5

--- HEX8 AND PEN6

HOOP AND VERTICAL STRESSES FOR HYDROSTATIC LOAD OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS.

FIG. 178
RADIAL DISPLACEMENTS FOR HYDROSTATIC LOAD
OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS.

FIG. 179
HOOP AND VERTICAL STRESSES FOR GRAVITY + HYDROSTATIC LOAD
OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS

FIG. 180
RADIAL DISPLACEMENTS FOR GRAVITY + HYDROSTATIC LOAD
OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS. FJG. 181
PRINCIPAL STRESSES FOR GRAVITY+HYDROSTATIC LOAD OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS USING HEX20 AND PEN15 ELEMENTS.

FIG. 182
Principal stresses for gravity + hydrostatic load obtained from a rigid valley FE analysis using HEX8 and PEN16 elements.

**UPSTREAM FACE**

Viewed from upstream

**DOWNSTREAM FACE**

Viewed from upstream

- TENSION (Kg/cm²)
- COMPRESSION (Kg/cm²)
CROSS-SECTION OF THREE-DIMENSIONAL IDEALISATION ADOPTED FOR ELASTIC VALLEY F.E. ANALYSIS USING HEX18, PEN16 AND PLJ8 ELEMENTS.

FIG. 184
Plan of three-dimensional idealisation adopted for elastic valley F.E. analysis using HEX8, PEN6, and PLTB elements.
HOOP AND VERTICAL STRESSES FOR GRAVITY LOAD OBTAINED FROM AN ELASTIC VALLEY F.E. ANALYSIS.
DISPLACEMENTS AT LEVEL S67

DISPLACEMENTS AT LEVEL S42

DISPLACEMENTS AT LEVEL S17

DISPLACEMENTS AT LEVEL S72

DISPLACEMENTS OF CROWN CANTILEVER

RADIAL DISPLACEMENTS FOR GRAVITY LOAD OBTAINED FROM AN ELASTIC VALLEY F.E. ANALYSIS. FIG. 187
HOOP AND VERTICAL STRESSES FOR HYDROSTATIC LOAD
OBTAINED FROM AN ELASTIC VALLEY F.E. ANALYSIS. FIG. 189
DISPLACEMENTS AT LEVEL 607

DISPLACEMENTS AT LEVEL 642

DISPLACEMENTS AT LEVEL 817

DISPLACEMENTS AT LEVEL 732

DISPLACEMENTS OF CROWN CANTILEVER

RADIAL DISPLACEMENTS FOR HYDROSTATIC LOAD OBTAINED FROM AN ELASTIC VALLEY F.E. ANALYSIS. FIG. 190
Figure 191

Principal Stresses for Hydrostatic Load Obtained from an Elastic Valley F.E. Analysis.
HOOP AND VERTICAL STRESSES FOR GRAVITY + HYDROSTATIC LOAD OBTAINED FROM AN ELASTIC VALLEY F.E. ANALYSIS.

FIG. 192
RADIAL DISPLACEMENTS FOR GRAVITY + HYDROSTATIC LOAD
OBTAINED FROM AN ELASTIC VALLEY F.E. ANALYSIS.  FIG. 193
PRINCIPAL STRESSES FOR GRAVITY + HYDROSTATIC LOAD
OBTAINED FROM AN ELASTIC VALLEY FE. ANALYSIS

FIG. 194

UPSTREAM FACE
VIEWED FROM UPSTREAM

DOWNSTREAM FACE
VIEWED FROM UPSTREAM

TENSION (Kg/cm²)

COMPRESSION (Kg/cm²)
<table>
<thead>
<tr>
<th>Plane</th>
<th>Element Numbers</th>
</tr>
</thead>
<tbody>
<tr>
<td>124</td>
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<td>122</td>
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<td>121</td>
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<td>120</td>
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<tr>
<td>117</td>
<td>109 101 93 85</td>
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</table>

FIG. 195

ELEMENT NUMBERS FOR FAULT X
STABILITY OF FAULT X FOR R = 1000 AND HOG \((G_{\text{dam}} + G_{\text{v}})^{2} + H_{\text{dam}} + \gamma_{\text{gc}}\)

Shear Stress
Normal Stress (kg/cm²)

Safety Factor (T = Tension)

Overall Safety Factor and Direction of Resultant Shear Force

FIG. 197
Shear Stress
Normal Stress (kg/cm²)

Safety Factor
(T = Tension)

Overall Safety Factor and Direction of Resultant Shear Force

0.50
DIP

2:1

Overall Safety Factor and Direction of Resultant Shear Force

0.47
DIP

STABILITY OF FAULT ELOISA FOR
\(10(G_{am} + G_v) + 10(H_{am} + H_g)\)

FIG. 198

a) ELEMENT NUMBERS
b) RESULTS FOR R=1
c) RESULTS FOR R=1000
15.2
0.S.E
4

O.S.F. - Overall Safety Factor
D.R.S.F. - Direction of Resultant Shear Force

SCALE

Shear Stress (kg/cm²)
Normal Stress (kg/cm²)
Safety Factor

b) RESULTS FOR R-1

c) RESULTS FOR R-1000
1) \(G_{\text{dam}} + G_{\text{v}} + 10(G_{\text{dam}} + H_{\text{ec}})\)

**STABILITY OF PLANE 3 FOR FIG. 200**

**a) ELEMENT NUMBERS**

**b) RESULTS FOR R=1**

**c) RESULTS FOR R=1000**
STABILITY OF PLANE 9 FOR I-0 \((C_m + G_v) + 10 (H_{dam} + H_{gc})\)

**a) ELEMENT NUMBERS**

**b) RESULTS FOR R=1**

**c) RESULTS FOR R=1000**

**Shear Stress**

**Normal Stress**

**Direction of Resultant Shear Force**

**Overall Safety Factor**

**Safety Factor**

**Valley Surface**

**Fault X**

**Downstream**

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OVERALL STABILITY OF THE JOINT PLANES FOR
\[ I \cdot (G_{\text{DAM}} + G_{V}) + I \cdot (H_{\text{DAM}} + H_{\text{GC}}) \]

FIG. 203
The self-weight of the valley as a stabilising force, safety factors and directions of maximum shear forces.

Fig. 204
<table>
<thead>
<tr>
<th>Name</th>
<th>R</th>
<th>Shear Force in Strike Direction (tonnes)</th>
<th>Shear Force in Dip Direction (tonnes)</th>
<th>resultant Shear Force (tonnes)</th>
<th>Direct Normal Force (tonnes)</th>
<th>Direction of resultant Shear Force (degrees)</th>
<th>Safety Factor</th>
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**OVERALL STABILITY OF THE JOINT PLANES FOR**

\[ I \cdot O \left ( G_{\text{Dam}} \right ) + I \cdot O \left ( H_{\text{Dam}} + H_{\text{GC}} \right ) \]

**FIG. 205**
HYDROSTATIC LOAD ON THE GROUT CURTAIN AS A DESTABILISING FORCE. SAFETY FACTORS AND DIRECTIONS OF MAXIMUM SHEAR FORCES

FIG. 206
over all Stability of the Joint Planes for
\[ I \cdot O \left( G_{\text{DAM}} + G_v \right) + I \cdot O \left( H_{\text{DAM}} \right) \]
AN OVERLOAD CONDITION.
SAFETY FACTORS AND DIRECTIONS OF MAXIMUM SHEAR FORCES

FIG. 208
<table>
<thead>
<tr>
<th>Name</th>
<th>R</th>
<th>Shear Force in Strike Direction (tonnes)</th>
<th>Shear Force in Dip Direction (tonnes)</th>
<th>Resultant Shear Force (tonnes)</th>
<th>Direct Normal Force (tonnes)</th>
<th>Direction of Resultant Shear Force (degrees)</th>
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**OVERALL STABILITY OF THE JOINT PLANES FOR**

\[1.0 \left( G_{\text{dam}} + G_{v} \right) + 4.0 \left( H_{\text{dam}} + H_{\text{gc}} \right)\]

**FIG. 209.**
HOOP AND VERTICAL STRESSES
OBTAINED FROM LOAD CASES 2 & 11 DURING THE PRELIMINARY TESTS, LOAD CASES 1-33.

FIG. 210
Principal Stresses for Load Factor = 0.5

Obtained from Load Case 2 during the preliminary tests, Load Cases 1-33.

Tension (kN/cm²)

Compression (kN/cm²)
Principal Stresses for Load Factor = 0.5

Obtained from load case II during the preliminary tests. Load cases 1-33.

**Fig. 212**

- **UPSTREAM FACE**
- **DOWNSTREAM FACE**

- Tension (Kg/cm²)
- Compression (Kg/cm²)
PRINCIPAL STRESSES FOR LOAD FACTOR = 1.0
OBTAINED FROM LOAD CASE II DURING THE PRELIMINARY TESTS, LOAD CASES 1-33.

FIG. 213
PRINCIPAL STRESSES FOR LOAD FACTOR = 1.0

OBTAINED FROM A RIGID VALLEY F.E. ANALYSIS, USING HEX18 AND PEN16 ELEMENTS, AND WITH THE UPSTREAM NODES AT THE BASE OF THE SOCKET ON THE LEFT BANK AT LEVELS 767, 792 AND 817 RELEASED.

FIG. 214
Comparision of Experimental and Theoretical Hoop Stresses For Load Factor - 1.0

**Key Results**
- **Load Case**
  - **Model I**: Results for Loading Condition A
  - **Model II**: Model's first loading to L.F. = 1.0

**Comments**
- Elastic Valley F.E. Analysis
Key Results Load Comments Case
---
Theory  •  Elastic Valley F.E. Analysis
Model I  59  Results for Loading Condition A
Model II 11  Model's first loading to L.F. = 1.0

COMPARISON OF EXPERIMENTAL AND THEORETICAL
CROWN CANTILEVER STRESSES FOR LOAD FACTOR = 1.0

FIG. 216
### Key Results

<table>
<thead>
<tr>
<th>Model</th>
<th>Load Case</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Model I</td>
<td>59</td>
<td>Results for Loading Condition A</td>
</tr>
<tr>
<td>Model II</td>
<td>48</td>
<td>Scaled from Results for LF = 2.0</td>
</tr>
<tr>
<td>Model II</td>
<td>52</td>
<td>Scaled from Results for LF = 1.75</td>
</tr>
</tbody>
</table>

All results correspond to $E = 174,000$ Kg/cm².

### COMPARISON OF EXPERIMENTAL AND THEORETICAL DISPLACEMENTS FOR LOAD FACTOR = 1.0

FIG. 217
THE STABILITY OF FAULT ELOISA
AND ITS DEPENDENCE ON THE ANGLE OF INCLINATION

FIG. 218
STRESSES AND DISPLACEMENTS FOR HYDROSTATIC LOAD
OBTAINED FROM A RIGID SHELL F.E. ANALYSIS USING QDS12 ELEMENTS.

<table>
<thead>
<tr>
<th>Key</th>
<th>Method of Applying Hydrostatic Load</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Linear pore pressure distribution through thickness of dam.</td>
</tr>
<tr>
<td></td>
<td>Surface pressure on upstream face of dam.</td>
</tr>
</tbody>
</table>

FIG. 219