Abutment Stability Studies for the Gordon Arch Dam

By

D. T. Allen, B.E., M.I.E.Aust.

(Hydro-Electric Commission of Tasmania)

SUMMARY.- This paper deals with the foundation investigations that were carried out for a 460 ft. high arch dam on the Gordon River in Tasmania. A method was developed for computing the safety of an abutment block against sliding on two planes of weakness, each with wave angle shape, based on insitu observations and on an interpretation of the results of laboratory rock shear tests. This is followed by a brief description of the drainage measures adopted to improve the stability of the abutments.

I.- INTRODUCTION

The Hydro-Electric Commission of Tasmania is constructing the first stage of power development on the Gordon River in the South West region of the state. This will have an installed capacity of 750,000 kilowatts and is estimated to cost $95,000,000. The maximum operating head is 638 ft. The storage formed by the Gordon dam will be 9.5 million acre feet.

The Gordon dam will be a 460 ft. high double curvature asymmetrical arch dam. When built it will be the highest concrete dam in Australia.

Before adopting the arch dam an extensive programme of foundation investigation was carried out. The work described in this paper formed a part of these investigations

II.- NOTATION

- θ_1 angle between surface and horizontal
- θ_2 primary wave angle
- θ_3 secondary wave angle
- ϕ basic friction angle
- c cohesion
- α angle between shear direction and horizontal in shear test
- h hydrostatic head
- $\mathbf{n}$ unit vector normal to plane
- $\mathbf{n}'$ unit vector normal to wave angle face
- $\mathbf{t}$ unit vector of intersection of base and side
- $\mathbf{t}_T$ unit vector in direction of sliding
- $\mathbf{W}$ rock weight vector
- $\mathbf{R}$ resultant dam thrust vector
- $\mathbf{V}$ combined load vector
- u percolation pressure
- $\mathbf{U}_C$ vector of percolation force on upstream crack
- $\mathbf{U}_B$ vector of total percolation force on base
- $\mathbf{T}$ vector resultant force in sliding direction
- T scalar resultant force in sliding direction
- N'_B scalar total reaction on wave faces of base
- $\mathbf{U}'_B$ vector percolation force on wave faces of base
- U'_B scalar percolation force on wave faces of base
- A_B area of active wave face of base surface
- F factor of safety against sliding

Note: Suffixes "S" and "B" are used in the text to refer to side and base surfaces of the failure block.

III.- GEOLOGICAL INVESTIGATION

(a) TOPOGRAPHY

Site investigations commenced in 1962. Until the 55 mile long access road reached the damsite in the summer of 1966-67 all transport of men and equipment was by helicopter. Weather conditions restricted air operations to the summer months.

A prominent feature of the area in which the damsite is situated is a series of hard quartzite ridges running north south. The Gordon damsite is located in an extremely narrow gorge where the westward flowing river has cut through one of these ridges. The right abutment is overhanging for a vertical distance of 300 ft. immediately downstream of the dam and the average slope of the left abutment is 2:1 (vertical to horizontal).

(b) METHODS OF GEOLOGICAL INVESTIGATION

The three methods used to obtain geological information were surface mapping, tunnelling or aditing, and diamond drilling.

The Commission's experiences with the foundations of Meadowbank and Cethana dams (Refs. 1 and 2) had shown that the stability was determined largely by the faulting and that faults could only be positively located and inspected by adits. Three adits were driven on the right abutment and

Fig. 1. - Gordon Damsite Gorge.

four were driven on the left abutment, the total length being approximately 5000 ft. A special feature of the investigation was the close control of aditing. Adits were driven in stages following discussion between the engineers and geologists concerning the extra information required. Parts of some adits were located where they could subsequently be used as drainage adits. The adits provided valuable information on the foundation weaknesses in the abutments. A complete exposure of each feature was obtained in the walls and roof of the adit. This revealed the roughness, planarity or irregularity of the contact surfaces and also the infilling material.

In the early stages of investigation the role of surface mapping was the identification of probable fault zones but in the gorge surface mapping was limited by the steepness of the topography and by vegetation. At a late stage valuable information was obtained when a portion of the left abutment was sluiced to expose the rock.

(c) GEOLOGY

The pre-Cambrian rocks at the damsite are intensively folded on north-south axes. The sequence up to crest level consists of fairly massively foliated quartzites containing thin inter-foliated lenses of schistose chloritic and micaceous materials. The quartzite layers are up to about two feet thick but are variable in thickness and lateral extent. Flecks of mica occur throughout the quartzite but are more numerous in the schistose bands where they are oriented parallel with the foliation. During the intense folding process shearing took place preferentially on the chlorite seams. However in most cases these seams are highly contorted and do not form extensive planar weaknesses in the rock.

The purpose of the diamond drilling was to check the general uniformity of the rock. Such information includes the spacing of joints, the location of faults and the watertightness of the rock. Most of the diamond drilling was concentrated in the riverbed. Three holes totalling 900 ft. were drilled in the left abutment to check the watertightness of the rock and to confirm fault locations. The total length of diamond drilling was 9000 ft. This is a relatively small amount of drilling for a dam of this size.

The principal joint directions and the surface trace of the faults are shown in Fig. 2. The joints are near vertical and in general are very tight.

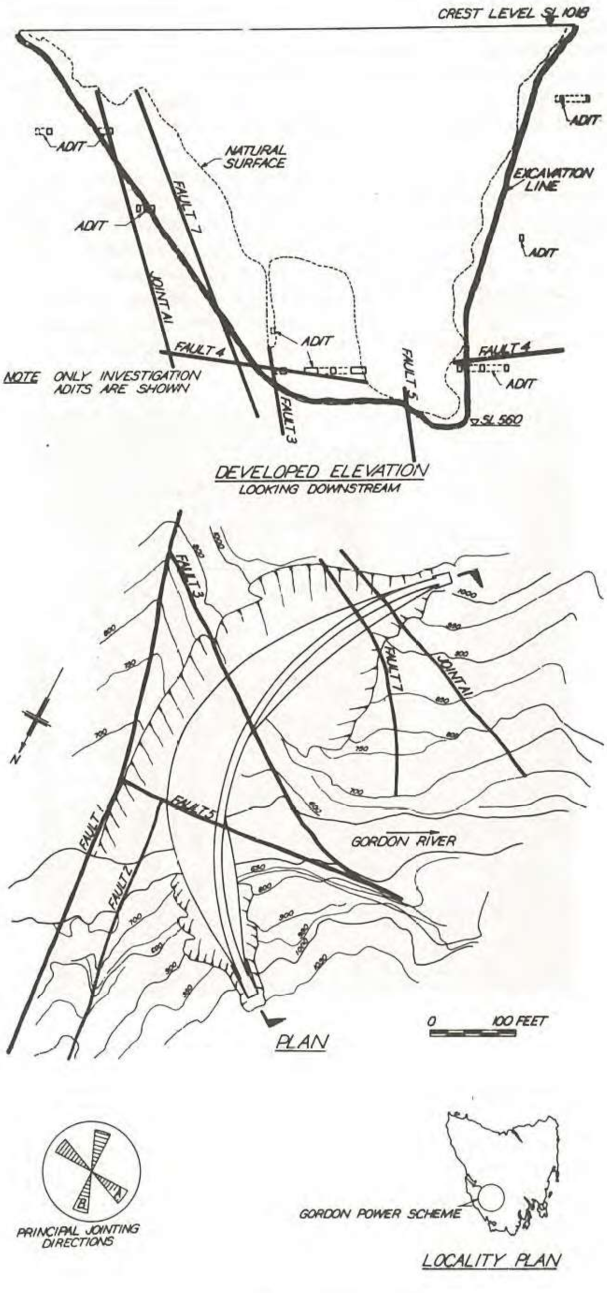

Fig. 2. - Gordon Damsite.

Joints have been designated "A" or "B" joints according to their direction.

Faulting and jointing tend to be in the same directions. The major faulting is along faults 1 and 2. When other faults approach these they become more open and show greater displacement. On the right abutment no near vertical faults could be found in the region downstream of fault 1. On the left abutment three faults occur dipping steeply towards the river. Of these, faults 3 and 5 do not constitute a stability problem as the rock wedges they form are supported at their downstream end.

Fault 4 is the only nearly horizontal fault at the site. It commences close to the riverbed near fault 1 and is continuous in a downstream direction. Fault 4 makes an angle of about 16° to the horizontal, rising in a downstream direction. It consists of a zone several feet thick containing sound quartzite separated by undulating shear planes of chlorite which are typically one half to three inches thick.

Fig. 3 is a photograph of a model which was constructed to aid interpretation of the geological data. It consists of a series of horizontal perspex plates. The plates are generally located at adit levels. All significant observations are recorded on the model. The value of the model as an aid in visualising the relationship between the dam and the underground features cannot be overemphasised.

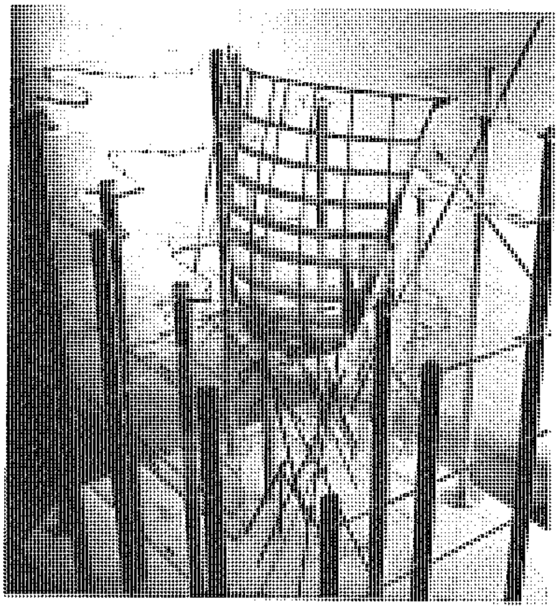

Fig. 3. - Perspex Model of Damsite.

IV.- INITIAL DESIGN

(a) During the course of the investigation the dam was moved progressively further downstream in order to lie completely clear of faults 1 and 2. (Refer to Fig. 2.) Fault 7 on the left abutment then lay immediately beneath the dam contact and emerged on the abutment a short distance downstream. Calculations showed that this block would be unstable under the dam thrust. It was decided to extend the dam profile and the excavation behind fault 7. This

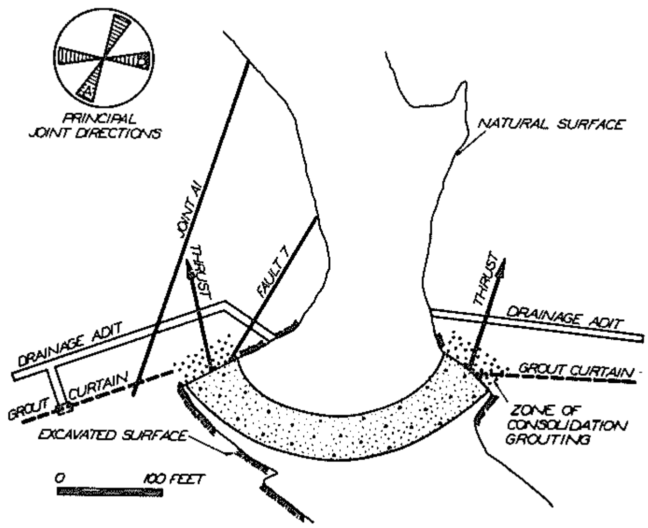

Fig. 4. - Horizontal Section.

accentuated the asymmetry of the dam and gave greater chord lengths across the arch. The volume of rock excavation on the left abutment was thereby increased. Total left abutment excavation volume was 170,000 cub.yds.

The dam is oriented in the gorge with its thrust directions approximately equally positioned with respect to surface contours. This is illustrated by Fig. 4 which is a horizontal section through the dam and the abutments.

(b) LEFT ABUTMENT STABILITY

Beyond the foundation contact lay a prominent tight joint designated joint A1. Sluicing of the abutment surface indicated that it emerged at the surface downstream of the dam (see Figs. 2, 4). Of the joints exposed by the aditing and sluicing this was the only joint that was continuous in the foundation immediately downstream of the dam. Joint A1 was oriented such that it made a small angle with the thrust and therefore its stability warranted scrutiny. As the vertical height between the adits was approximately 120 ft. there was some difficulty in identifying joint A1 in the different adits. In order to confirm the joint identification adits at three levels were driven along it thereby obtaining an exposure of the joint in the roof. The joint was limonite stained but very tight for most of its length.

The depth of joint A1 was such that it would have been very costly to extend the dam beyond it. Preliminary calculations were made using a polar plot to aid in the vector manipulations, and these confirmed that detailed study was received. The devastating failure of the Malpasset arch dam in France (Ref. 3) has also emphasized the necessity for detailed studies of rock defects below or behind arch dam foundations.

In order to avoid delays in the design programme

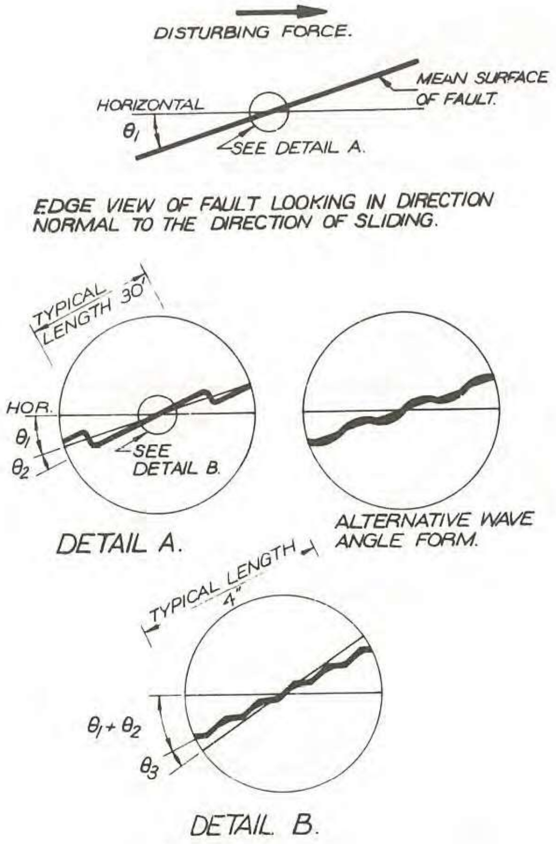

Fig. 5. - Wave Angle Concept.

for the dam itself it was decided to adhere to the dam profile and to ensure the stability of the abutment by other means if necessary. One such method would have been to drive a tunnel or shaft across the weakness transverse to the load and to provide the shear strength with concrete backfill (Ref. 4). Other methods considered were: flattening of the arches; rotating the dam at the expense of the stress state in the right abutment. Fortunately none of these measures were required.

V.- WAVE ANGLE CONCEPT

A close inspection of joint A1 and fault 4 indicated that although their overall position was planar their surfaces exhibited substantial irregularities. A realistic stability analysis would need to take account of these. The concept used to describe these irregularities is referred to here as the wave angle concept. A similar approach has been used in Ref. 5.

The concept is illustrated in Fig. 5 by reference to fault 4. The profile of fault 4 in the direction of sliding consisted typically of straights about 30 ft. long making an angle of about 10 degrees to the mean plane surface. The rock forming these irregularities was sound and extremely hard. With a rock compressive strength of the order of 25,000 p.s.i. failure could not occur by crushing these irregularities. Thus for sliding to occur, the upper block must move at the angle θ_2 to the mean surface. This is the primary wave angle effect.

The faces on the upstream portion of each wave shall subsequently be referred to in this paper as the active faces because the shear strength properties of these faces determine the resistance to sliding in the sliding direction described above.

Surface irregularities on a much smaller scale appeared again on each of the 30 ft. lengths. (See detail "B" in Fig. 5.) However, because of their small size, the likelihood of shearing through infilling material precluded the use of the secondary wave angle effect (θ_3).

Observations on the joint A1 in the roofs of adits indicated that its primary wave angle effect (θ_{2S}) could be taken as 20 degrees.

VI.- SHEAR TESTS

(a) SAMPLING AND TESTING

In order to determine the strength parameters of the potential failure surfaces laboratory shear tests were carried out. Insitu shear tests were abandoned after several unsuccessful attempts to isolate a section of fault 4 in an adit.

The value of a laboratory shear test is dependent on the degree of disturbance obtained during sampling. Isolation of samples of rock straddling the joint (or fault) was carried out by line drilling in a Vee pattern, firing the holes with detonating fuse only, then removing the rock by hand chiselling. In every case some disturbance of the sample was caused. Taking care to retain any infilling material, the sample halves were then carefully matched together, marked with the shear directions corresponding with the sense of loads applied by the dam, and encased in wax for transport. In the laboratory, after embedding each half of the sample in plaster, the specimen was ready for testing.

The initial vertical stress was generally set at the computed insitu stress. The horizontal load was incremented to a maximum and then continued at reducing values until displacement took place at constant horizontal load thereby indicating the residual strength phase. Then the vertical stress was incremented and the shearing process repeated. Some specimens were tested at initial vertical stresses below the computed insitu stress to check whether this affected the shear resistance. Contact areas were typically 8 sq. in.

(b) RESULTS

Vertical and horizontal loads in the test were resolved to obtain shearing and normal loads for the actual plane of shearing in the test. In Fig. 6 the slope of the line BC in the displacement plot is a direct measure of the slope of this plane. The range of values obtained for the angle between the plane of shearing and the horizontal (from +25 to -20 degrees) indicates the difficulties in predicting the probable plane of shearing.

Tests were carried out for shear displacements greater than could be tolerated in the abutment.

For stability calculations it was decided to use shear strength values for shear displacements up to 0.1 inches because greater displacements would have seriously increased the stresses in the dam and the uplift pressures. In some tests it was apparent that up to 0.05 inches shear displacement took place during the "bedding in" phase caused through sample disturbance. When the peak horizontal load (P1 in Fig. 6) occurred at less than 0.1 inches shear displacement a point was obtained on the initial shear strength envelope. However, for some samples, the first peak occurred beyond the displacement limit. In such cases the shear strength at the limiting displacement was plotted against the normal stress. By plotting the results for all the specimens it was possible to determine a mean strength envelope and a minimum strength envelope and hence the coefficient of friction and the cohesion. For tests on rock joints the term "cohesion" is a measure of the force required to shear through rock points or through infilling material when there is no normal load.

The small area constituting the plane of shearing in the test represents one of the small planes forming a secondary wave angle face. This face is superimposed on the active primary wave angle face of the joint. As it had previously been decided to ignore the secondary wave angle effect in order to be conservative, the shear strength parameters obtained for the plane of

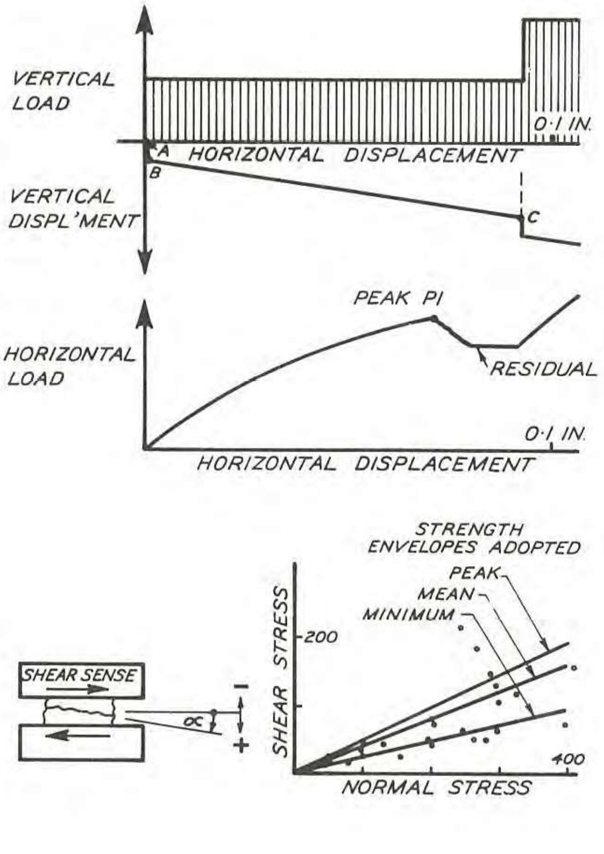

Fig. 6. - Typical Shear Test Graphs.

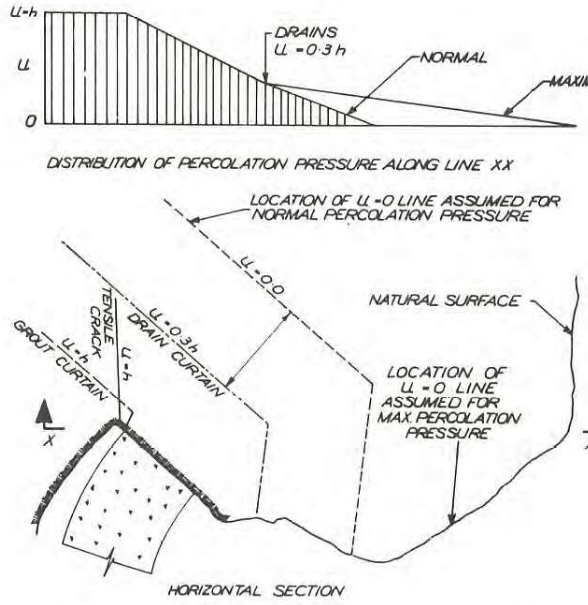

Fig. 7. - Percolation Pressure Distribution.

shearing in the test become the shear strength parameters of an active face of the primary wave angle surface.

Nine shear tests were carried out on samples from fault 4 and twelve tests on samples from joint A1. As the results exhibited a considerable range a mean value and a minimum value were extracted:-

Fault 4 : tan ϕ_B : mean 0.4, min. 0.25
C_B : mean 0 p.s.i., min. 0 p.s.i.
Joint A1: tan ϕ_S : mean 0.8, min. 0.6
C_S : mean 50 p.s.i., min. 0 p.s.i.

VII.- PERCOLATION PRESSURES

The watertightness of the rock was checked by means of water pressure tests on the diamond drill holes. Three holes totalling approximately 900 feet were drilled in the left abutment above fault 4. In general the rock was very watertight. One of the characteristics of the tectonicly jointed quartzite rock is that although most of the joints were limonite stained this was not necessarily indicative of leakage potential. Water loss from a 10 ft. stage rarely exceeded 2 g.p.m. when tested at stage pressures up to 350 p.s.i. This corresponds to a loss rate of .0006 g.p.m./ft./p.s.i. - a rate at which grouting is normally considered to be unnecessary. However in parts of the abutment where open joints were detected typical water losses were 10 g.p.m. at stage pressures of 45 p.s.i. All adits in the abutment were dry. No natural aquifers were detected in the left abutment.

The term "percolation pressure" is used to describe the pressure of water in joints and fissures. It is considered that for abutment studies this term is more meaningful than "uplift".

The prediction of percolation pressures within the abutment is overshadowed by the existence of the joints and faults which would invalidate an analytical solution based on the assumption of homogeneity. One of the conclusions from the water pressure testing at this site was that most of the flow would take place along a few relatively open joints. One consequence of the limited coverage of the abutment by diamond drilling was that, at the design stage, open joints may have been undetected for distances of 50 feet or more. It was considered that a more extensive drilling program was not economically justified. Unless all such potential aquifers were identified any mathematical model would very likely be a poor one. The mathematical problem for homogeneous conditions has been solved (Ref. 6).

The permeability of jointed rock also varies with applied stress (Ref. 7). The tensile zone in the abutment upstream of the dam will have greater permeability than the compressive zone downstream of the dam. It can be shown that this effect increases the percolation pressures in the abutment.

The view was taken that it would be reasonable to use empirical data as a guide to the percolation pressures and to review the drainage system if measured pressures exceed those assumed. Much data has been accumulated on the pressures that exist under gravity dams (e.g. Refs. 8 and 9) but very little exists on pressures within steep dam abutments. The differences between the two situations are significant. The drain holes beneath a gravity dam usually take the form of vertical drill holes within which a hydrostatic pressure distribution exists. Mean percolation pressure at the plane of the drains lies in the range 0.2 to 0.4 of the hydrostatic head of the reservoir.

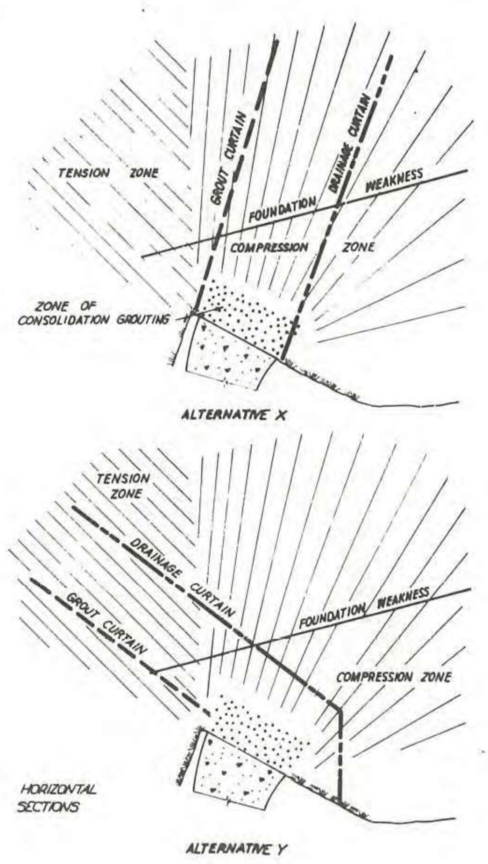

Fig. 8 - Alternative Locations for Drainage Curtains.

In steep abutments it is possible to provide drains that are free draining, thus lowering the mean pressure at the drain curtain. Drainage is further improved by orienting the drain holes to intersect the principal joints. The mean percolation pressure at the drainage curtain was assumed to be 0.3 of the hydrostatic head. The system was designed such that a blocked drain condition could not occur. (See Section XI.) Two alternative assumptions were made for the distribution of percolation pressure downstream of the drainage curtain. See Fig. 7.

The advantages of locating the drains well upstream have been pointed out in Ref. 7 by Minassian, Sabarly and Londe. This feature was adopted. In Fig. 8 the drain location "Y" intersects the foundation weakness further upstream than the conventional location shown in "X". This reduces the total percolation force on the weakness but is achieved at the expense of increased leakage because the drainage curtain extends into the tension (and hence more open) zone where the drains may function more effectively. The tight rock conditions at this damsite enable the upstream alternative to be used.

VIII.- STRESS ANALYSES

Stress analyses of horizontal sections of the abutments were made using the finite element method and assuming that the rock has tensile strength. The studies showed tensile stresses upstream of the dam contact and oriented roughly normal to the "B" joints. As the tensile strength of the joints would be negligible it was evident that they would open and cause a redistribution of stress. Movement could tend to concentrate on one joint emanating from near the upstream corner of the dam. In order to determine whether it continued in the "B" joint direction a short crack was inserted and the section re-analysed. This process was repeated. These studies defined the probable crack direction at various levels in the abutment, but generally the crack made an angle of about 55° with the thrust direction. In the subsequent stability calculations it was assumed that full hydrostatic head existed in the crack.

IX.- EARTHQUAKE LOADING

In recent years the possibility that earthquake activity may be associated with the filling of reservoirs of great depth or of great volume has been described in the literature, eg. Ref. 13. Studies were made of this effect but expert overseas opinion indicated that the problem was not sufficiently understood to make any firm statements about a particular locality.

The evidence of arch dams in Japan and elsewhere indicates that even high magnitude earthquakes close to a dam have not caused damage. The extraordinary acceleration - (0.62g) - sustained by Koyna dam in India without irreparable damage is but one example of this. Tasmania is in an area of very low seismicity, the largest earthquake on record being magnitude 5.3. Earthquakes of magnitude 5-6 require rupturing to occur simultaneously over a length of perhaps 10 miles of fault and they tend to occur on existing active faults (Ref. 14). The Lake Edgar fault scarp passes through the storage 15 miles from the damsite and has a length of approximately 20 miles. The decision was made not to include earthquake loading on the foundations after taking into account the complete absence of population downstream of the dam.

In order to obtain detailed information on the seismicity before and after filling, the State seismic net has been extended by five stations, microseismic recorders installed at Strathgordon, precise surveys carried out, and in the dam a strong motion instrument and four seismoscopes will be installed. A full time seismologist has been appointed.

X.- STABILITY ANALYSES

(a) POTENTIAL FAILURE BLOCK

The potential failure block analysed is illustrated in Fig. 9. The boundaries of the block are:

(i) A plane defining the base of the block (Fault 4)
(ii) A steeply dipping plane which intersects the base plane and forms the side of the block. Prominent joint A1 is the side plane.
(iii) The surface of the abutment after excavation.
(iv) An upstream boundary formed by the tension crack.

After considering the deformation characteristics of the foundation rock it was decided to assume that the potential failure block behaved as a rigid body. The weight of the block was computed assuming a rock density of 160 lb./cub.ft.

(b) DAM THRUST

Stresses in the dam shell due to its own weight and due to water at maximum flood level were determined mainly by means of the Commission's computer programme for the analysis of arch dams. Stresses were confirmed by model studies. The results gave radial, tangential and vertical components of thrust at various levels on the foundation contact. As a check the thrust components acting downstream were integrated around the perimeter of the dam and compared with the water load. Finally the thrust components were used to determine the single resultant thrust vector applied by the dam to the potential failure block.

(c) METHOD OF ANALYSIS

The literature on arch dam foundations contains many approaches to the problem (e.g. Refs. 10, 11, 12). For the Gordon damsite a lengthy computer programme was written to perform all stages of the analysis including the calculation of rock weight, dam thrust and percolation forces. The main value of the programme was that it enabled a large number of cases to be studied covering the many combinations of possible values of the parameters. It would also permit a rapid reassessment of the stability if other weaknesses were found during construction.

Unit vectors ($\mathbf{n}_S$ and $\mathbf{n}_B$) normal to the side and base planes are used to compute the unit vector ($\mathbf{t}$) of the plane intersection. This gives the direction of movement for zero wave angles. The true direction of incipient sliding ($\mathbf{t}_T$) makes an angle equal to the wave angle with each surface. (See Fig. 9). Using the vector cross product,

$$\mathbf{t} = \mathbf{n}_S \times \mathbf{n}_B$$

$$\mathbf{t}_T = \mathbf{t} - (\mathbf{n}_B)(\sin \theta_B) - (\mathbf{n}_S)(\sin \theta_S)$$

Unit vectors normal to the wave angle faces ($\mathbf{n}'_S$ and $\mathbf{n}'_B$) are then computed after making $\mathbf{t}_T$ a unit vector.

$$\mathbf{n}'_S = \mathbf{t}_T \times (\mathbf{n}_S \times \mathbf{t}_T)$$

$$\mathbf{n}'_B = \mathbf{t}_T \times (\mathbf{n}_B \times \mathbf{t}_T)$$

The external forces acting on the block are combined into a single load vector for convenience.

$$\mathbf{V} = \mathbf{W} + \mathbf{R} + \mathbf{U}_C$$

By neglecting the tensile strength of the upstream crack and using the vector dot product the total force in the direction $\mathbf{t}_T$ is given by the vector $\mathbf{T}$ from which its scalar magnitude (T) can be computed.

$$\mathbf{T} = \mathbf{t}_T(\mathbf{V}\cdot\mathbf{t}_T + \mathbf{U}_S\cdot\mathbf{t}_T + \mathbf{U}_B\cdot\mathbf{t}_T)$$

It is to be noted that the wave angle concept required that there be a component of $\mathbf{U}_S$ and $\mathbf{U}_B$ in the direction of sliding. This effect is illustrated in Fig. 10. However the small increase thus obtained in T is offset by the improved thrust directions relative to the wave angle faces. The reactions (N'_S and N'_B) on the wave angle faces are obtained by apportioning $\mathbf{V}$ between them to satisfy the condition:

$$\mathbf{V} - (\mathbf{V}\cdot\mathbf{t}_T)\,\mathbf{t}_T = -N'_S(\mathbf{n}'_S) - N'_B(\mathbf{n}'_B)$$

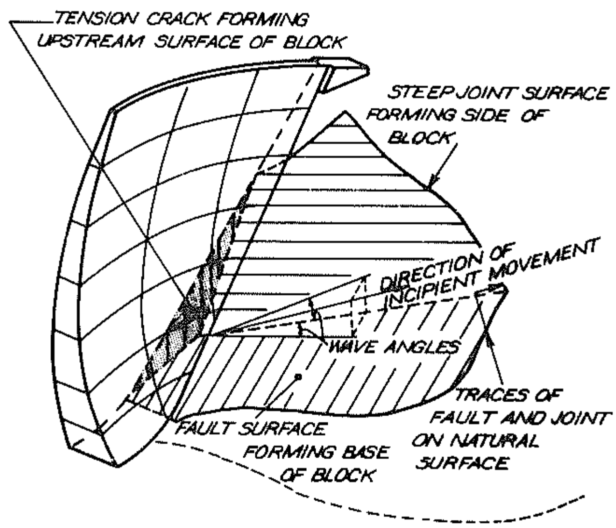

Fig. 9. - Potential Failure Block.

The solution, which is obtained by resection, is possible because there are two unknowns but three equations implied in this expression. The minus signs on the right correct for the convention adopted in Fig. 10.

The original percolation forces acting on the mean failure surfaces are then corrected to obtain the percolation forces applicable to the wave angle faces.

$$U'_S = U_S - (U_S \cdot t_T)t_T$$

$$U'_B = U_B - (U_B \cdot t_T)t_T$$

These are then converted to obtain the corresponding scalars U'_S and U'_B.

The factor of safety is defined as the ratio of the forces resisting sliding to the forces tending to cause sliding. The only forces actively resisting the sliding are the shear forces mobilised on the active faces of the wave angle surfaces. The net force tending to produce sliding is T

$$F = \frac{(N'_S - U'_S)\tan\phi_S + (N'_B - U'_B)\tan\phi_B + A_S c_S + A_B c_B}{T}$$

Attention is drawn to the fact that, by this definition, the component of the rock weight acting upstream has been included in the denominator, thereby reducing the net sliding force. It is also worth noting that if $(N'_S - U'_S)$ is negative the problem must be re-analysed as a block sliding on a base plane, making suitable changes to the wave angle if necessary. In this condition, which did not occur, the side plane would not be providing any resistance to sliding.

(d) RESULTS

By means of the computer programme analyses were made not only for the most probable values of shear parameters but also for possible combinations of the range of parameters. It was considered unlikely that every parameter could have its most adverse value simultaneously. However the stability was checked when one parameter at a time had its most adverse value. For the most realistic case normal percolation pressure was assumed and mean values of shear parameters were used. The measured wave angles ($\theta_{2S} = 20°$ and $\theta_{2B} = 10°$) were used. This gave a value $F = 3.0$ for the factor of safety. For the assumption of maximum percolation pressure $F = 2.6$. For the most unfavourable case the assumptions were: maximum percolation pressure, minimum shear strength parameters, and measured wave angles. This case gave a factor of safety $F = 1.2$. Other studies showed that the stability was sensitive to changes in the predicted crack direction. Rotating the assumed crack direction 10° in a downstream direction gave a factor of safety $F = 1.6$ for all other conditions assumed normal.

It was required that the factor of safety be greater than 2.0 for normal conditions and that it should be greater than 1.0 for the worst case. On this basis it was concluded that the stability of the block was satisfactory.

XI.- DESIGN FOR STABILITY

(a) DRAINAGE DESIGN

It is essential that drainage be adequate to hold percolation pressures to the design limits. The drainage curtain as designed consists of 3" dia. holes drilled at 15 ft. centres. To prevent the possibility of the drains becoming blocked, all drain holes connect between the adits. The required drilling accuracy can be achieved with percussion equipment by the use of 2½" tuberod extensions and by restricting the adit spacing to 120 feet. This was verified by drilling trials. Within the inclined plane of the curtain all holes are inclined relative to the steepest line on the plane to give frequent intersections with the near vertical joints.

Supplementary drainage holes will be drilled from the adits to intersect any features that appear to be important aquifers. All significant weaknesses should be evident in the drainage adits. Other holes may be drilled to intersect fault 4 and joint A1 at wide spacings downstream of the curtain. As a check on the percolation pressures 33 Kyowa piezometers will be installed.

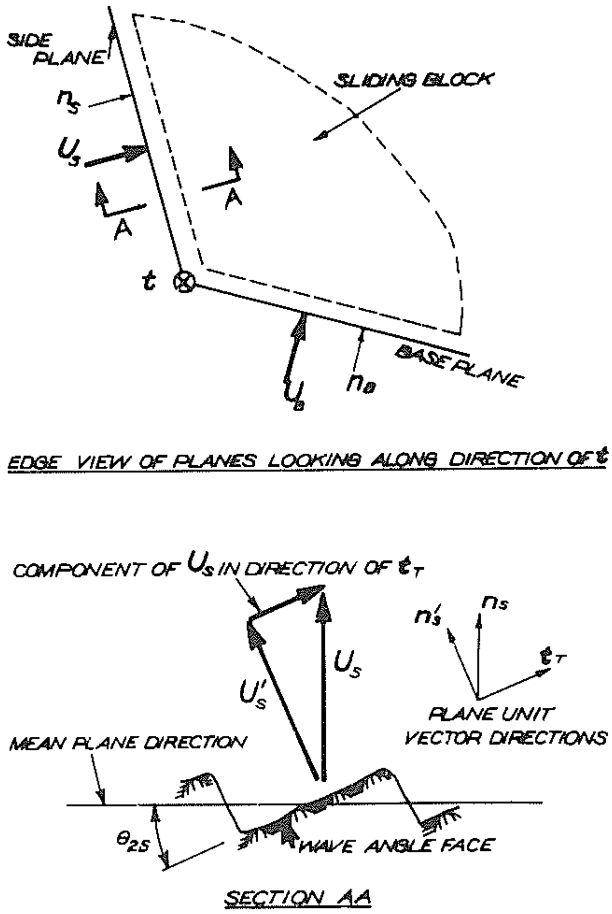

Fig. 10. - Percolation Force Directions.

(b) GROUTING

The grout curtain will be installed from chambers located as shown in Fig. 4. The quantity of grout injected will be limited to reduce the likelihood of grout travelling to the drainage curtain area and blocking the natural drainage paths.

(c) CONTROLLED FILLING

When filling the storage the rate of filling may be controlled after the water has risen to 210 feet below full supply level. Below this level the total load on the potential failure block is less than a quarter of its value with water at F.S.L. For the upper half of the storage the rate of increase in water level will be of the order of two feet per month if all water is stored. The control outlet at mid height will enable the lake level to be held if abutment measurements are unsatisfactory. For the extremely large volume of water stored in the upper levels of the lake it was considered not economically justified to install an outlet of sufficient size to produce a rapid fall in lake level if an emergency arose.

XII. CONCLUSIONS

The possible failure mechanism revealed by the exploration required extensions to existing rock mechanics approaches to make a realistic assessment of the stability of the abutment. When storage filling is completed the piezometer readings will provide useful information on percolation pressures in abutments.

XIII. ACKNOWLEDGEMENTS

The author is grateful to the Commissioner for the Hydro-Electric Commission, Sir Allan Knight, Kt., C.M.G., M.E., B.Sc., B.Comm., F.I.E.Aust., for permission to publish this paper. Mr. G. T. Colebatch, O.B.E., B.E., F.I.E.Aust. is the Chief Civil Engineer of the Commission. The author is also indebted to many other members of the Commission's staff. The work described was carried out under the direction of Mr. M. D. Fitzpatrick, M.I.E.Aust. Mr. N. O. Boughton, M.E., M.I.E.Aust. was the senior engineer responsible for the execution of the design work on the dam and the foundations and all work was carried out under his direct supervision. The computer programme for analysing the abutment stability was written in collaboration with Mr. P. J. Bullock, B.E., Grad.I.E.Aust.

REFERENCES

1. Maddox, J.M., Kinstler, F.L. and Mather, R.P. - Meadowbank dam foundations. Civ. Engg. Trans. I.E. Aust., Vol. CE9, No. 2, Oct. 1967, pp. 321-333.

2. Boughton, N.O. and Hale, G.E. - Foundation studies for Cethana arch dam. Proc. Ninth I.C.O.L.D. Conf. Istanbul 1967, Q 32, R 10, pp. 143-164.

3. Jaeger, C. - Malpasset Report. Water Power Vol. 15, No. 2, Feb. 1963.

4. Artola, P.M. - Les travaux souterrains du barrage D'Aldeadavila. Proc. Seventh I.C.O.L.D. Conf., Rome 1961, pp 851-872.

5. Bray, J.W. - A study of jointed and fractured rock. Rock Mechanics and Engg. Geology, Vol. V, Nos. 2, 3, 4, 1967.

6. Zienkiewicz, O., Mayer, P. and Cheung, Y.K. - Solution of anisotropic seepage by the finite element method. Journal A.S.C.E., EMI, Vol. 92, Feb. 1966.

7. Minassian, W. Ter., Sabarly, F.L. and Londe, P. - How to protect arch dams from water pressure in the abutments. Proc. Ninth I.C.O.L.D. Conf., Istanbul 1967, Q. 32, R. 12, pp. 185-199.

8. Ginsbourg, M.B. - Study of the effect of grout curtains and drainage on uplift intensity in concrete dams. Proc. Sixth I.C.O.L.D. Conf., New York 1958, Q. 21, R. 130.

9. Casagrande, A. - Control of seepage through foundations and abutments of dams. Geotechnique, Vol. XI, No. 3, Sept. 1961.

10. Jaeger, C. - Rock mechanics and hydro power engineering. Water Power, Vol. 13, Nos. 9 and 10, Sept., Oct. 1961.

11. Muller, L. - Safety of rock abutments of concrete dams. Proc. Seventh I.C.O.L.D. Conf., Rome 1961, pp. 663-678.

12. Drnovsek, J. - Static and kinematic conditions of the stability of jointed and fissured rocky foundations of arch dams. Rock Mechanics and Engg. Geology, Vol. 6, No. 4, 1968.

13. Rothé, J.P. - Seismes artificiels. Tectonophys., 9, 1970, pp. 215-238.

14. Albee, A.L. and Smith, J.L. - Earthquake characteristics and fault activity in Southern California. Engg. Geology in Southern California. Assn. of Engg. Geologists, Oct. 1966.