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RECORDS:

RECORD No. 1964/140

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FOUNDATION GROUTING AND JOINT PERMEABILITY MEASUREMENTS AT BENDORA DAM, A.C.T.

by

J.K. HILL

The information contained in this report has been obtained by the Department of National Development, as part of the policy of the Commonwealth Government, to assist in the exploration and development of mineral resources. It may not be published in any form or used in a company prospectus without the permission in writing of the Director, Bureau of Mineral Resources, Geology and Geophysics.

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APPENDICES

FIGURES

Fig. 1:- Bendora Dam, Cotter River, A.C.T.
News and Information Bureau photograph

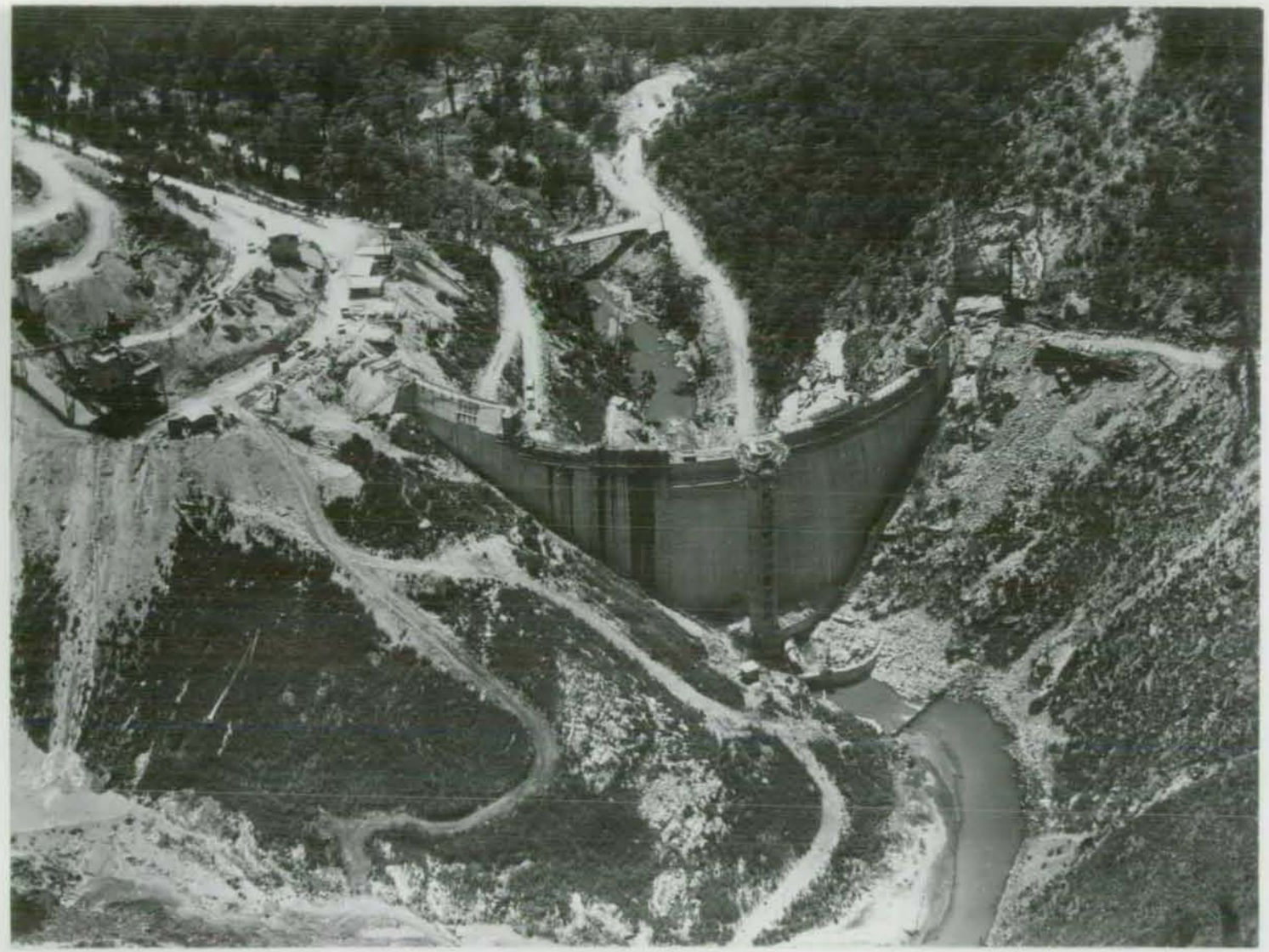

Fig. 2:- Aerial view of Bendora Dam nearing completion in 1961. Beds dipping towards the river are visible in the thrust block excavation on the right hand side. A fairly narrow spur forms the western abutment on the left hand side.

News and Information Bureau photograph.

FOUNDATION GROUTING AND JOINT PERMEABILITY MEASUREMENTS AT BENDORA DAM, A.C.T.

SUMMARY

This report describes methods used in foundation grouting at Bendora Dam, Cotter River, Australian Capital Territory, and places on record an analysis of grout consumption in hard, fractured, metamorphic rocks at the site. Relationships found between joint permeability measurements, made in exploratory drill holes, and grout consumption are discussed. So that comparison with other dam sites in different geological environments may be made, a brief description is given of the site geology. Techniques used to make a joint permeability assessment of the foundations are described and the need to calibrate injection packers when high flow rates occur is emphasised. A summary of the role of engineering geology in the work at Bendora Dam concludes the report.

Joint permeability of the foundations was measured in 1668 feet of drill hole and, above the water-table, ranged from 0 to 2270 feet/year. Rock below the water-table was virtually impermeable, the average being 30 feet/year. The water-table gradient was about 1 in 10.

In consolidation grouting, consumption ranged from 0 to 91 cubic feet of cement per 100 square feet of rock surface, with a mean of 13 cu. ft/100 sq.ft., for 30-foot holes and 6:1 water:cement grout. Curtain grouting consumption ranged from 0 to 29 cu. ft. of cement per lineal foot of hole, with a mean of 0.78 cu. ft. per lineal foot. The mean for consolidation grouting was 0.53 cu. ft/lin. ft.

Analysis of grout consumption shows that rock with a joint permeability of less than 400 feet/year accepted very little grout; not more than 10 cu. ft. of cement per 100 sq. ft. of rock surface. Rock with joint permeability up to 900 feet/year was grouted with takes of up to 50 cu. ft/100 sq. ft. Beyond this point grout consumption increased rapidly with increasing joint permeability, and was at the order of 90 cu. ft/100 sq. ft. for 1200 feet/year.

INTRODUCTION

The Bendora Dam was completed by the Commonwealth Department of Works in 1961 on the Cotter River, about 42 miles from Canberra by road. The storage created by the dam augments that provided by an earlier dam on the Cotter River, 1½ miles from its confluence with the Murrumbidgee River, and is intended to meet the water supply requirements of a population of about 100,000 persons in Canberra. Bendora Dam is a doubly curved thin concrete arch with gravity thrust blocks. A crest spillway discharges onto an apron at the toe of the arch. The maximum height of the wall is 155 feet and the arch has maximum thickness of 30 feet at the base and 9 feet at the crest. The arch rests on a massive gravity plug in the bed of the river, through which the river diversion openings were constructed. The valve tower for water supply and river outlet pipes is situated on the plug at the heel of the dam. The volume of concrete in the arch only is 30,630 cubic yards (cu. yds); the total volume of concrete in the dam and valve tower, but excluding spillway apron and protective paving, is 31,255 cu. yds.

Geological investigations by the Bureau of Mineral Resources commenced at the site in July 1956, culminating in a drilling and joint permeability testing programme of 13 holes totalling 1668 feet, which was completed in February 1958. The use of NM-LC bottom discharge bits with either stationary solid inner tube or stationary split inner tube core barrels gave almost 100% core recovery. Both pneumatic and mechanical packers were used for joint permeability tests; the former proved more satisfactory in closely jointed rock.

SITE GEOLOGY*

The Cotter River flows in a youthful valley with overlapping spurs and steep sides, and a gradient of about 45 to 50 feet per mile. The dam site is a moderately acute V-shaped gorge eroded out of the Tidbinbilla Quartzite formation whichis considered to be of Silurian age. The rocks consist of very hard strong quartzite and silicified quartz-sandstone and siltstone, in beds ranging from about 6 inches to 10 feet thick, and thin interbeds and lenses of friable sandstone, claystone, and sedimentary breccia, generally less than 6 inches thick. Shaly partings between beds are common. There is a 8 to 10-foot bed of silicified ashstone in the left abutment, and an unusually thick bed (3 to 8 feet) of sedimentary breccia about 50 feet beneath the right abutment excavation line. The sedimentary breccia beds consist of moderately hard, fine to medium grained, finely bedded quartz sandstone and quartzite, containing thin beds and lenses of fine-grained friable quartz sandstone, siltstone and claystone, often in the form of angular fragments and shards. The material is generally weathered and friable in both outcrop and drill core.

In the right abutment (Fig.5) the beds dip towards the river at angles ranging from 12 to 35 degrees; in the left abutment the dip is about 15 to 25 degrees into the hill but tends to flatten to about 10 degrees in the upper section owing to a roll in the bedding. In both abutments there is a small dip component in the upstream direction. Beds in the right abutment are thick and massive and constitute the lowest members of the sequence exposed at the site. Beds in the left abutment are thick near river level, but are mainly thin or moderately thick higher up the sequence.

A granite intrusion occurs 1500 feet east of the site. It extends downstream and crops out in the river some 600 feet from the dam. Twelve hundred feet west of the site is the Cotter Fault, a major lineament extending for more than 40 miles. The course of the Cotter River appears to be related to the fault over much of its length. The fault is the most important structural feature in the area and has controlled the fracture systems at the site, which are shown by joint rosettes in Fig.5.

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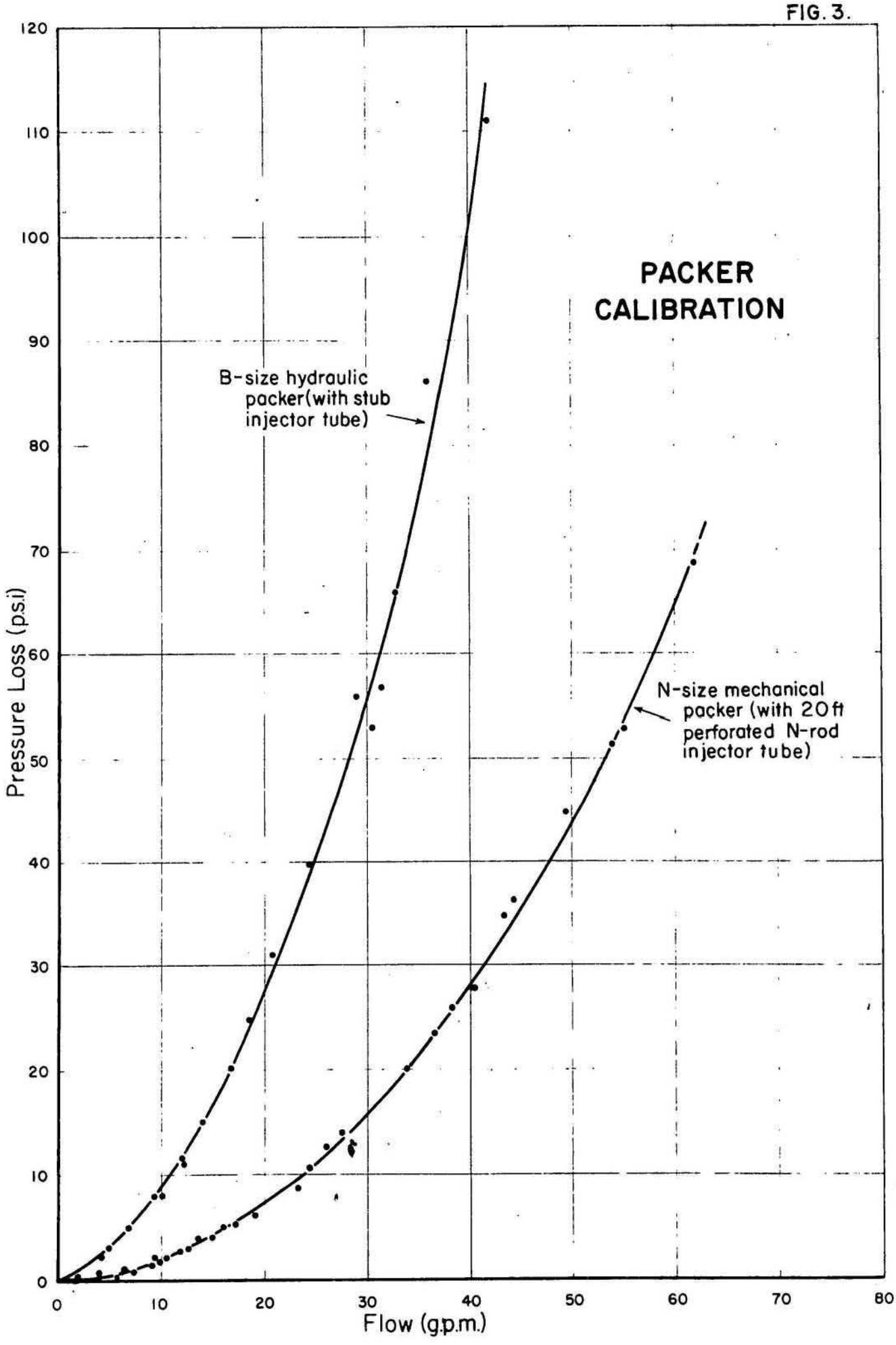
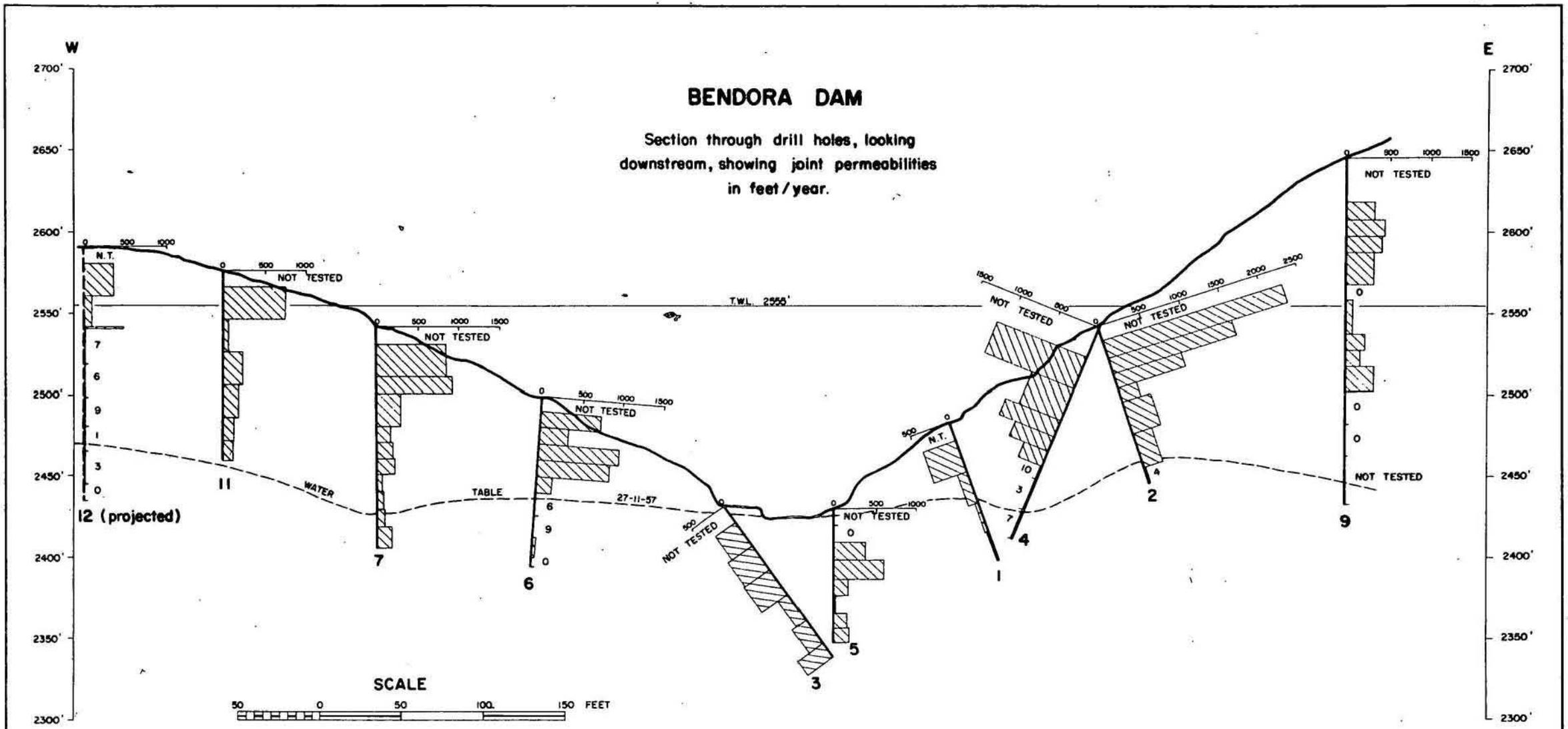
* The geological investigation of the site was done by geologists of the Bureau of Mineral Resources, and reported by Noakes (1956), Noakes, Foweraker, and Burton (1957), and Foweraker (1958). The author worked on the site discontinuously from June 1960, and was assigned to the investigation which is the subject of this report. A full report on the engineering geology of the Bendora Dam site will be published in due course by the Bureau.

FIG. 3.



May 1965 *To accompany Record 1964/140* I 55/A16/225

FIG 4

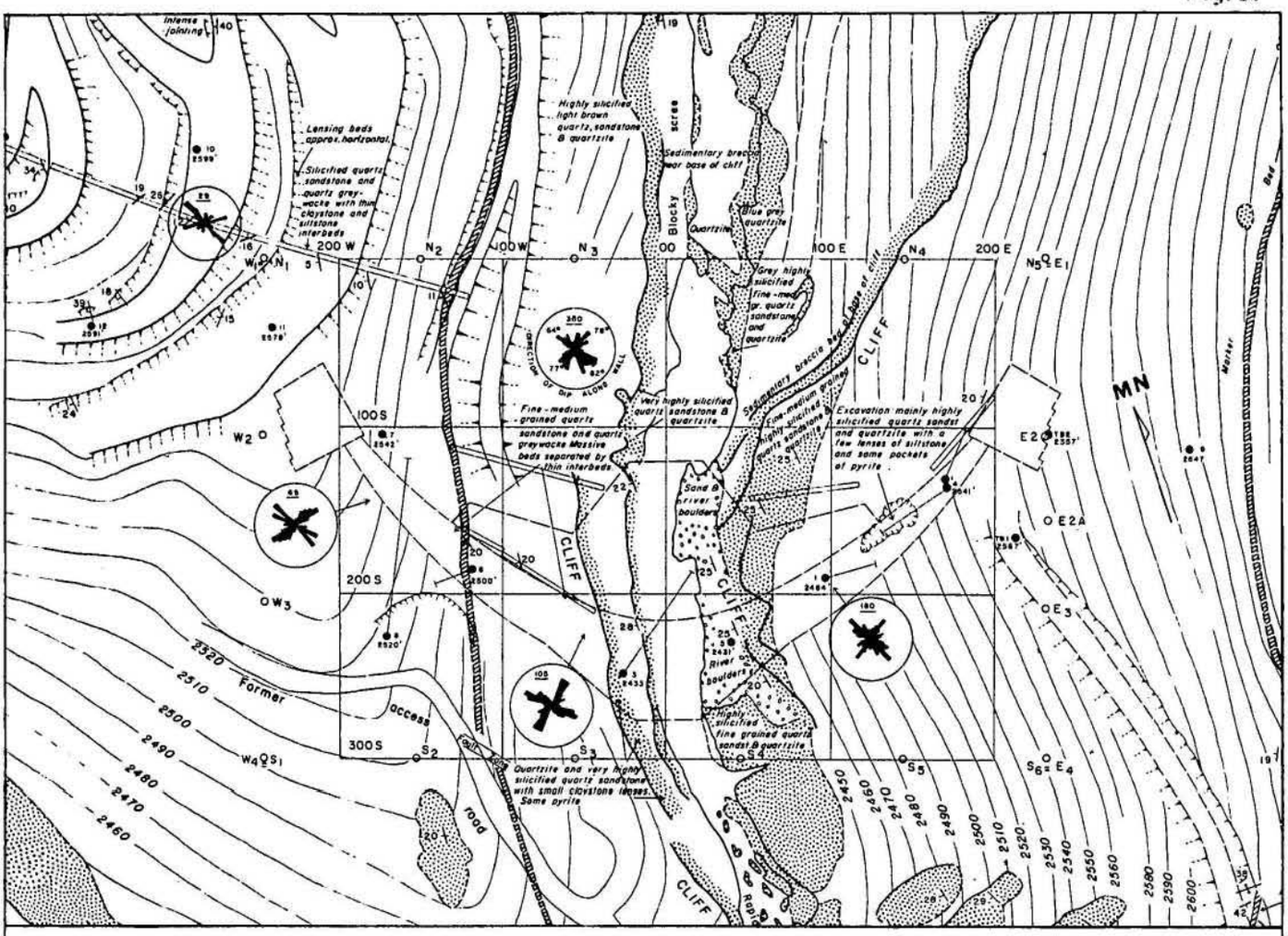


Water pressure testing programme designed by J.C.Fowerakar and L.C.Noakes. Analysis and compilation by J.K. Hill.

Bureau of Mineral Resources, Geology and Geophysics. May 1965.

To accompany Record 1964/140 155/16/221

Fig. 5.



BENDORA DAM

GEOLOGICAL PLAN OF SITE

SCALE

100 0 100 200 FEET

River sand, gravel and boulders.

Tidbinbilla Quartzite. Silicified quartz sandstone, siltstone, and greywacke; quartzite; thin beds and lenses of fine grained friable sandstone, siltstone, claystone, and sedimentary breccia.

Silicified very fine grained ashstone, (marker bed).

Main sedimentary breccia bed.

25 Strike and dip of bedding.

Anticline, position accurate.

Geological boundary, position accurate.

Geological boundary, position approximate.

Joint rosette showing relative frequency of dip directions, average angle of dip, and number of joints measured.

Joint rosette showing relative frequency of strike directions and number of joints measured.

Diamond drill hole — vertical, inclined, with hole number and elevation of collar, (see table for other details).

Cliff.

Embankment, excavation, or road cutting.

Costean.

Access road.

100 E Site grid.

N_5 E_1 Bench mark (metal spike set in concrete).

2310 Ground surface contour, approximate.

Outline of dam, approximate.

Geology by: L.C. Noakes, J.C. Fowerakar, G.M. Burton, J.K. Hill, D.E. Gardner, D. Moore and J. Barrie.

May 1965. *To accompany Record 1964/140.* I55/A16/215

There is evidence of extensive minor faulting at the site, in the form of slickensided joints, thin films of silty gouge on joints, zones of intensely jointed rock, offset bedding planes, and small displacements in the ashstone marker bed. No discrete fault zones were found in the foundations of the arch wall, although a deep zone of open jointing in the river bed, revealed by results of joint permeability tests and grout consumption, is thought to be tectonic in origin. Foweraker (1958) considers the general pattern of rock movement at the site to be consistent with relief of stress by small differential movements along numerous joints and bedding planes. He considers the joint pattern to be in keeping with compression across the Cotter Fault - the two main systems are shear joints oblique to the direction of principal stress. A minor set of tension joints is parallel to it.

JOINT PERMEABILITY MEASUREMENTS

The joint permeability values quoted in this paper are calculated from water losses in gallons per minute from a 10 or 20-foot section of NX or BX drill hole subjected to a series of pressures generally in the range 10 to 120 pounds per square inch (p.s.i.). The test methods used and the results obtained are described by Foweraker (1958). Each section of hole was tested immediately after it was drilled, using a single injection packer, either pneumatic or mechanical in operation. After steady flow conditions were established, the water loss at each of five gauge pressures was metered in a series of 5-minute tests. Pressure water was usually supplied by the drill circulation pump. In the Bureau's current work, either gravity feed from a tank higher up the hillside or a centrifugal pump is preferred because of the absence of pressure fluctuations and the ability to test satisfactorily any very high leakage sections encountered. The gauge pressures were corrected to give effective pressure in the test section by allowing for the water column in the supply line in the drill hole, back pressure due to ground water, pressure loss in the supply line due to friction, and pressure loss at packer and injector tube. A mathematical formula was used to reduce water losses for each section of hole to a "permeability" value in feet per year. The latter, because of anisotropy and inhomogeneity of fissured rock, and assumptions made in deriving the formula, is not an absolute quantity equivalent to the permeability of porous sedimentary rocks, but rather is a relative joint or fracture permeability. It provides a convenient means of comparing leakages from drill holes in fractured rocks between different parts of a site or between different sites. The formula was developed by Mr. M.A. Chapple, of the Snowy Mountains Hydro-Electric Authority, and was made available to the Bureau of Mineral Resources in 1961. The formula is given in Appendix II.

Presentation of data from joint permeability tests offers scope for new methods to be introduced. A method currently used by the Engineering Geology group of the Bureau is to plot the drill holes or their projections on a geological section showing lithology or structure, and then plot the joint permeability values for each test section with the hole as one axis and a joint permeability scale at right angles to it as the other axis. This has been done in Fig.4. Then, providing a sufficient number of holes has been drilled and tested, the joint permeability pattern can be established by assessing the relationships between the measured joint permeability values and: rock types and competency, joint systems and other structures, attitude of bedding, weathering processes and products, position of water table, topography, depth below surface, height above valley floor (with reference to

the unloading effect), steepness and stability of slopes, and other factors - all of which affect the joint permeability at a given point in the bedrock.

In the histograms shown in Fig. 4, all pressure corrections, except the packer correction, have been made before the joint permeabilities were calculated. Unfortunately packers used in the tests were not calibrated for pressure loss at various flow rates as it was not realised at the time that significant pressure losses might occur. As an illustration of the effect a packer can have on pressure lost between gauge and test section, two packer calibration curves from recent work at the Corin Dam site, on the Cotter River, are given in Fig. 3. Highest flows at Bendora were about 15 gallons per minute (g.p.m.) so the packer correction would be fairly small, and the fully corrected joint permeability values a little higher than those shown in Fig. 4, whereas flows up to 50 g.p.m. have occurred in several holes at Corin Dam site with pressure losses of the order of 40 p.s.i., which may almost double the joint permeability value when finally calculated.

With the limitation in mind that no packer corrections have been applied, the joint permeabilities and grout-consumption figures given in this paper may form a useful basis of comparison for other dam sites in similar geological environments.

Results from joint permeability testing helped to determine the optimum depth for curtain grouting and located the more permeable sections where extra treatment was required. The joint permeability varied from an average of 30 feet/year near or below the water table in the abutments to a maximum of 2270 feet/year in the openly jointed zone near top water level on the eastern bank. Joint permeability in the river bed rock was somewhat higher than for rock below the water table elsewhere, being about 250 feet/year.

Concern was felt at one stage that leakage might take place through the spur forming the left abutment because of very low water-table readings in some of the drill holes. The water-table gradient was found to be about 1 in 10. It was inferred that rock above the water table might have sufficient open joints to pass large volumes of water under a head of up to 100 feet. However, water losses from drill holes 7, 10, 11, 12 and 13 showed that, although foundation rock in the immediate vicinity of the left thrust block was fairly permeable (500 to 1000 feet/year), rock composing the spur west of the thrust block was much less so (0 to 650 feet/year), below R.L. 2540 feet, and that no appreciable leakage through the spur was likely. In addition. the water-table readings were taken during a dry period and probably give the lower limit of the water table.

Generally it was found that the zone of fluctuation of the water table marked the lower limit of open jointing and weathering. Joint permeability decreased near this zone, except where open joints of structural origin occurred unrelated to depth, and the rock became virtually impermeable below it. The general picture which emerged of the water tightness of the site was that leakage would be low or moderate over most of the foundations and easily controllable by standard grouting techniques. More critical sections would be the openly jointed zone high on the right abutment in the thrust block area, and the corresponding section at the left thrust block. It was thought that intensive consolidation grouting would provide strong foundations and that wing extensions of the grout curtain would be sufficient to control leakage in these areas.

Fig. 6a:- Western abutment foundations showing thinly to thickly bedded quartzite, quartz sandstone, quartz siltstone and ashstone, dipping into the hillside. The different intensity of jointing in thin and thick beds is evident. This rock, in the vicinity of Blocks 3 and 4, had a joint permeability of 550 to 700 feet/year and a cement consumption of 16 to 20 cu.ft/100 sq.ft of rock surface during consolidation grouting. Department of Works photograph.

Fig. 6b:- Massive thickly bedded rock in the eastern abutment dipping towards the river . The joint permeability of rock in the lower half of this view was about 100 to 250 feet/year. The cement consumption during consolidation grouting ranged from ½ to 4 cu.ft/100 sq.ft of rock surface. Department of Works photograph.

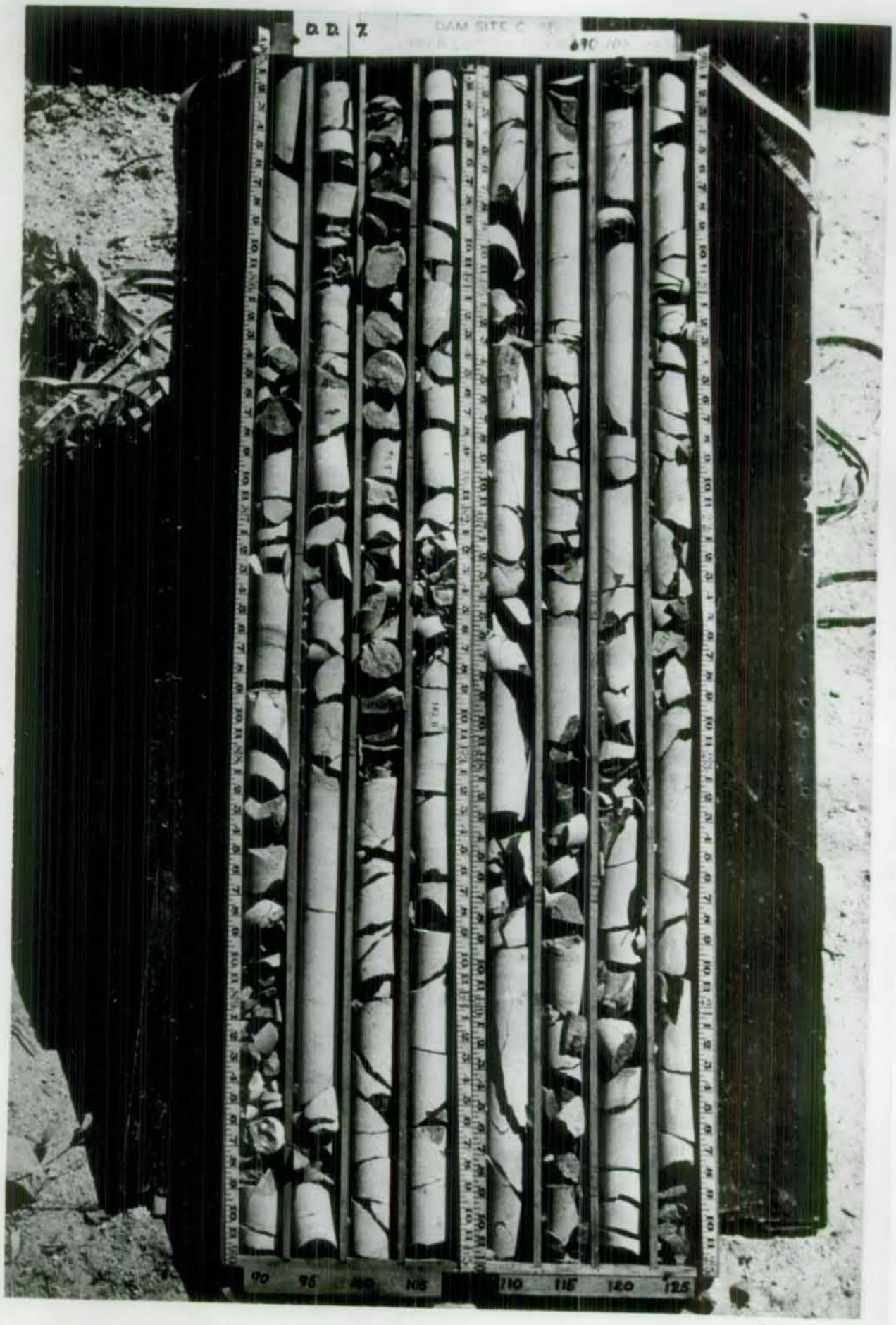


Fig. 7:- Drill core from hole No. 7, 85 to 125 feet, showing intensity of jointing. The joint permeability of test sections from 90 to 123 feet ranged from 60 to 85 feet/year. The depth to water table in this hole was 115 feet.

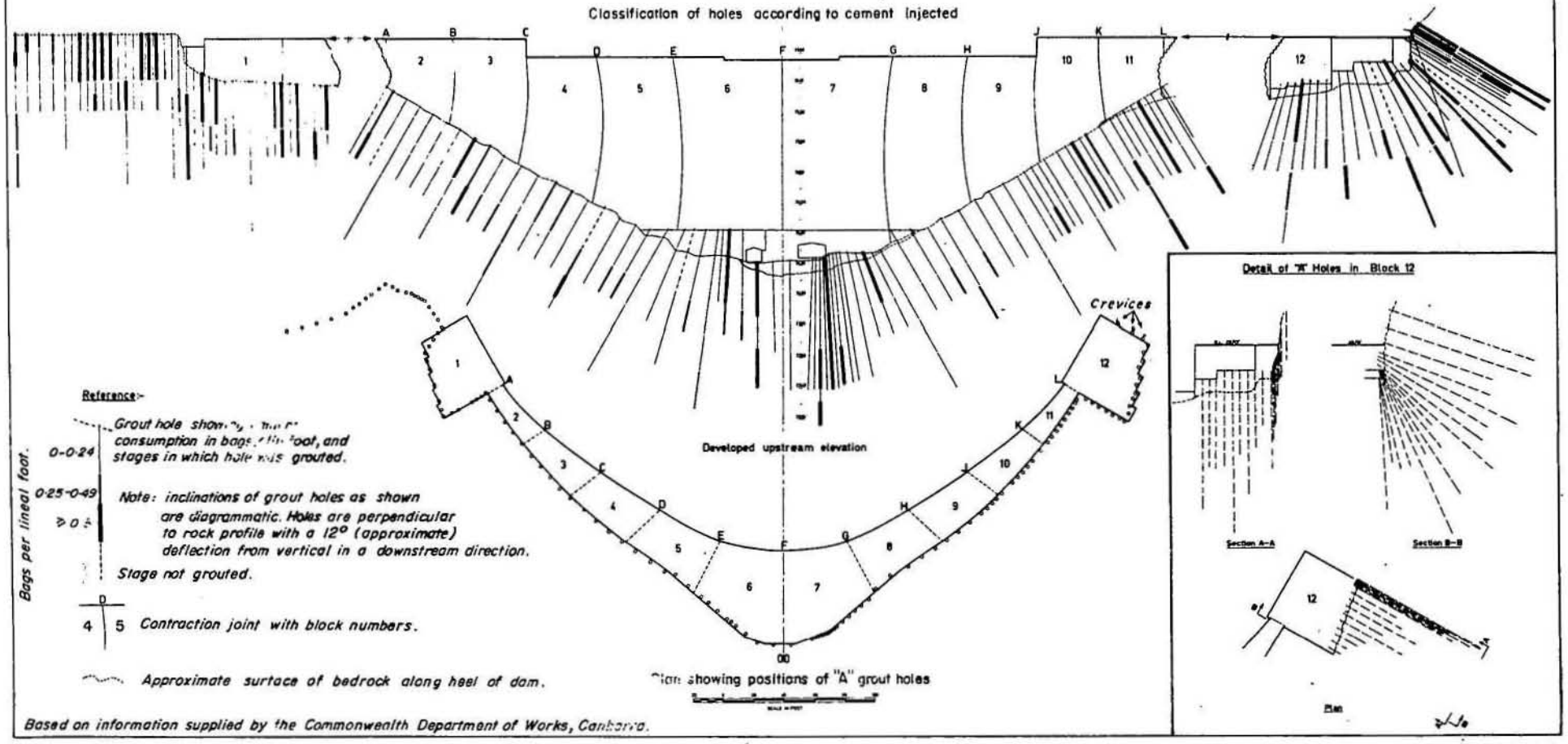
Bureau of Mineral Resources photograph.

Fig. 8.

BENDORA DAM

"A" Hole Curtain Grouting

Classification of holes according to cement injected



Based on information supplied by the Commonwealth Department of Works, Canberra.

May 1965. To accompany Record 1964/140.

155/A16/201

LOCATION AND EXTENT OF GROUTING

General

Two types of grouting were carried out at the site:-

(i) "A" hole high pressure curtain grouting to reduce seepage under the dam and to reduce uplift pressures.

(ii) "B" hole low pressure consolidation grouting, to strengthen the foundation and to reduce uplift pressures.

The consolidation and curtain-grouting programmes were planned by the Commonwealth Department of Works, Canberra. Recommendations by consultants from the Snowy Mountains Hydro-Electric Authority and geological advice from the Bureau of Mineral Resources played an important role in this work. All grouting operations, including washing and water testing of holes prior to grouting, and drilling of cored holes to check results, were supervised by Department of Works site engineers. Modifications were made to the spacing and orientation of some holes after consultations between geological and engineering staff.

"A" Holes

The "A" hole grout curtain extends from a point 90 feet west of the western corner of Block 1 to a point 90 feet east of the eastern corner of Block 12 (Fig.8). The curtain is composed of 137 holes of various depths down to 100 feet, but most (89 holes) are 50 feet deep or less. (Provision was made in the tender specifications for holes to a maximum depth of 150 feet, but these were not required). The holes are normal to the profile of the foundations with a 12-degree deflection from the vertical in a downstream direction, except those holes forming the western extension of the curtain from Block 1, which are vertical, and the fan holes on the eastern side of Block 12, which range from 0 to 60 degrees from the vertical. Holes in the centre foundations are fanned at an even rate from vertical at the centre to the same inclination as those in Blocks 5 and 8 in the abutments. The azimuths were obtained by compass and dips by templates fitted to the dam wall or by spirit level and protractor. The approximate "area" of the curtain is 102,000 sq. ft.

Checks of the inclinations and directions of "A" hole standpipes in Block 7 indicate that some holes may have been drilled with slightly incorrect orientations, so it is possible that some holes converge or intersect at depth. It has not been possible to determine whether this has had an effect on grout takes.

The nominal spacing of the collars of the 50-foot holes (which make up the bulk of the curtain) is 10 feet, but extra holes in very permeable sections have reduced the spacing to an average of 7.3 feet. The minimum spacing is 2.5 feet. The 75 and 100-foot holes have nominal spacings of 40 and 80 feet respectively. Further information on grout-hole spacing is given in Table IV, Appendix I.

The collars of the "A" grout holes are situated a few inches above the junction of the concrete footing and the foundation rock on the upstream side of the wall, except at Blocks 1 and 12, and in the vicinity of the valve tower at

Block 6, where the holes were drilled from benches through a few feet of concrete. "A" holes were drilled and grouted through 2-inch diameter standpipes placed directly into the rock through the heel of the dam. The holes were diamond-drilled, EX size, by contract drillers using a Mindrill E500 machine, and are 1½ inches in diameter. To prevent sealing of fissures during drilling, the use of grease or other lubricants on the drill rods was not permitted.

Interesting features of the "A" hole curtain are the wing extensions from each thrust block (Blocks 1 and 12. A line of vertical holes extends from the left thrust block directly away from the reservoir for about 50 feet, then follows the roadway leading to the former site of the concrete batching plant for about 70 feet (Fig.8). The collars of the holes are about 15 feet above top water level. Grouting of the holes was aimed at preventing seepage through the abutment in the immediate vicinity of the thrust block, where open jointing below top water level gave rise to rather high joint permeability figures, up to 950 feet/year in some sections during exploratory drilling. These higher permeabilities were subsequently reflected by relatively high grout takes, the average consumption for Block 1 being 0.69 cubic feet per lineal foot of hole, as opposed to the average for Blocks 2 to 5 lower down the abutment, of 0.18 cu. ft/lin. ft. An efficient grout curtain will have increased the length of leakage path around the thrust block foundations and may have reduced interstitial water pressure in joint systems in the abutment, with a resultant increase in rock stability.

The extension of the "A" hole curtain from Block 12 (the right-hand thrust block) is shown in the detail on Fig. 8. Excavation of foundations for Block 12 confirmed the persistence with depth of open joints previously found in outcrops and costeans, and also indicated by permeability testing and core from diamond drill holes. Values of joint permeability ranged from about 500 to 2300 feet/year. Open joints extend down to a silty sedimentary breccia bed which dips towards the river at about the level of the designed foundation line. Immediately below the breccia bed joints are tighter and the the rock is less permeable (200 to 600 feet/year); the excavation was therefore deepened to 5 to 10 feet below the bed to provide better foundations for the thrust block. However, movement of the overlying mass of interlocking joint blocks down the sedimentary breccia bed towards the river was considered possible should the coefficient of surface friction and/or shear strength of the silty material be reduced by accumulation of ground water. An anchored retaining wall to take part of the loading of the mass has been installed and the grout curtain is fanned out from the corner of the thrust block in an up-dip direction so that ground-water drainage from the hillside will either take place freely into the reservoir or be diverted downstream away from the thrust block. Any other direction of the grout-curtain extension might result in an undesirable accumulation of ground water in fissures leading to the sedimentary breccia bed.

"B" Holes

Low-pressure "B" hole consolidation grouting was carried out over the entire foundations of the dam and spillway, a developed area of approximately 32,300 sq. ft. Two-inch diameter holes, spaced from 5 to 20 feet apart (depending on joint patterns, the results of water tests, and previous grouting) were percussion-drilled to a depth of 30 feet normal to the plane of the foundations. Holes in the river bed are vertical, while those in the abutments

average about 36 degrees to the vertical. Some holes in the abutments and particularly in the river bed, are inclined in order to penetrate certain joints or faults at favourable angles. A total of 281 "B" holes was grouted; a density of 1 hole per 115 sq. ft. of rock surface.

PATTERN OF GROUTING

Information contained in this and the next four sections was supplied by the Commonwealth Department of Works.

"A" Holes

The intermediate stage or closure method of grouting was adopted at Bendora Dam for the high-pressure grout curtain. The method consists of drilling and grouting primary holes at 20-foot spacing to a depth of 30 feet. Rock with high grout consumption is then drilled and grouted with a series of intermediate or secondary holes at 20-foot spacing, bisecting the primary pattern. Tertiary and quaternary holes may be drilled later if high takes persist, so that the final hole spacing may be as small as 5 or 2½ feet. The second stage of each primary hole is then drilled and grouted from 30 to 50 feet, and the bisection process repeated wherever necessary. For the Bendora curtain, every second primary hole and the secondary holes on either side of it were terminated at this stage and the remainder (at a nominal spacing of 40 feet) were drilled and grouted from 50 to 75 feet. Of these holes every second one was continued to a depth of 100 feet, which is the deepest extent of the curtain. The nominal spacing is 80 feet. The last holes grouted at each depth (closure holes) indicate the effectiveness of the preceding grouting, providing the primary spacing has not been too great.

In practice, the planned layout was adhered to consistently except in the right thrust-block area, where the bisection method was discontinued and closure holes were irregularly placed. In this and other sections, a number of intermediate holes extending from the surface to 50 or 75 feet were drilled and grouted in one stage, when unusually high takes had occurred in adjacent holes (e.g. Block 7). In some cases extra single stage holes were specially drilled and grouted as a check.

"B" Holes

The "B" holes were either drilled and grouted in two stages, 0 to 10 feet, and 10 to 30 feet, or in one stage, 0 to 30 feet. No particular sequence of grouting was adhered to, other than that each area of foundation rock was grouted before concrete was placed. Test holes were drilled and grouted to check earlier grouting in some areas.

The original grid spacing of "B" holes was 10 feet, but this distance was varied to suit geological conditions in different parts of the foundations.

WASHING OF HOLES AND WATER TESTS BEFORE GROUTING

Washing

After drilling, each stage was washed, before water testing and grouting, by a compressed air and water jet which was passed up and down the hole. The use of compressed air was discontinued in holes that tended to cave. Up to 3 or 4 holes were washed simultaneously by filling each hole with water, circulating with clean water for a short period, and then blowing

the water out with compressed air. Holes were not washed out by capping and injecting water and air under pressure with the object of flushing clay and other joint filling materials out of adjacent holes, because it was decided to avoid any possibility of opening up joints or bedding planes which could subsequently cause subsidence if not completely filled by grout. For the same reason, no attempt was made to direct the flow of water to different holes or to reverse the direction of flow between two holes. When water escaped from nearby holes during washing, the holes were not sealed but left open to allow any loose material to be flushed out.

The washing and cleaning programme was considered to be sufficient preparation of the rock for grouting. Cores and joint-permeability testing had shown that most of the joints were either tight or, where open, clean. It was felt that an intensive washing programme was not desirable.

Water Tests

All grout holes were water-tested in stages immediately before grouting for a period of 10 minutes, except some early holes which were tested for only 1 minute. Gauge pressures for the tests were based on a rate of 1 p.s.i. per foot of depth and were calculated for the bottom of each test section or stage. The immediate purpose of water-testing each stage was to obtain a rough guide, by comparison with other holes, to the amount of grout likely to be used and to the pressures and grout mix best suited to the conditions. Water tests also revealed surface leaks which needed to be caulked to prevent subsequent leakage of grout. In practice, the contractor left much of the caulking till leakage occurred during grouting.

Water for testing was obtained from a tank situated higher up the abutment and the pressure was regulated by a valve in the supply line. This method, although very simple had the disadvantage that sometimes the desired pressure could not be maintained when a very permeable section was tested. The water was injected through a manifold, on which a pressure gauge and flow meter were mounted, attached to a standpipe at the collar of the hole. Correlation of water-test results with geology, joint permeability, and grout takes would probably reveal interesting relationships, but has not been attempted.

METHOD OF GROUT INJECTION

Grout was injected by the continuous circulation method, using a Gardner-Denver air-powered double-acting single bank reciprocating pump capable of maintaining pressures up to 400 p.s.i. The grout mixer was mechanically operated and provided with a meter for measuring the amount of water mixed with the cement. Mechanically agitated hold-over tanks were also provided with the unit. Pressures were measured by gauges at the pump and the grout-hole manifold, and volumes were measured by dip-stick in the grout tanks.

Most stages were grouted by injection at the standpipe, but in some "B" holes, notably in the right abutment, packers were used, each stage of the hole being grouted at a different pressure, from bottom to top. Packer grouting was adopted when the surface layers of rock were openly jointed, preventing the

use of higher pressure at depth with the stage grouting method because of excessive loss of grout at the surface. With packer grouting, normal pressure can be used in the lowest stage of each hole, ensuring efficient penetration of grout. The highest stage is grouted at the usual low pressure, reducing cement wastage to a minimum. Pneumatic packers were used for this work. When grout was found to be returning from an adjacent hole during packer grouting, the pressure of the grout was measured by seating a packer in the hole. It was specified that the pressure should not exceed the allowable pressure for that stage of the hole.

If, during normal stage grouting of a hole, large quantities of grout were found to flow from adjacent ungrouted holes, the latter were temporarily capped. However, for small losses, ungrouted holes were left open to facilitate the escape of displaced air and water. Connections between grout holes were always recorded. Generally areas with most connections correspond to those with highest grout consumption shown in Fig.9. Connected holes were grouted to completion at the specified pressures before grout from the original holes had set to ensure that no joints were left only partly filled or unable to accept more grout because of blockages. Surface leaks were caulked mainly with oakum, but wooden wedges, leadwool, and mortar were also used. It was necessary to reduce pressure on occasions before some cracks could be successfully caulked.

It was specified that grouting of each stage was to be continued until the hole accepted grout at a rate of less than 1 cu. ft. in 20 minutes at pressures* of 50 p.s.i. or less, in 15 minutes at pressures between 50 and 100 p.s.i., in 10 minutes at pressures between 100 and 200 p.s.i. and in 5 minutes at pressures in excess of 200 p.s.i. Whenever possible the full pressure was maintained continuously during grouting operations. On completion of grouting at each hole, a valve at the top of the standpipe was closed to retain the grout in the hole and connected joint systems until pressure dropped or until the grout attained its initial set. Grout in the hole was then washed out to permit drilling and grouting of lower stages.

INJECTION PRESSURES

"A" Holes

Pressures used in grouting the 0 to 30 foot stages of the grout curtain were limited according to the concrete load on the rock surface. It was assumed that although "B" hole consolidation grouting had been carried out before concrete was placed, it would be possible for grout under pressure to gain access to surface layers of rock or to the base of the dam itself, and, by applying pressure equal or nearly equal to that at which it had been injected. to deflect the wall. It was further assumed that the maximum area of concrete base over which the pressure grout could act would not be more than half the total area. This arbitrary rule therefore limited the first stage injection pressure to less than twice the concrete load on the rock at the time grouting began, and it was adhered to whenever practicable, exceptions being gravity blocks 1 and 12 (see Table V). The factor of safety was increased considerably by lateral constraint exerted by adjacent rock through shear

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* Gauge pressure at the hole

strength and friction forces developed along joints. These forces were regarded as intangible and no attempt was made to evaluate them. No "A" hole grouting was undertaken until the placing of concrete was completed or nearly completed in each block. The concrete load figures given in Table V were calculated by dividing the weight of concrete (at 150 lbs./cu. ft.) in each block at the time grouting commenced by the surface area over which its weight is distributed. Contraction joints between blocks had not been grouted at the time "A" holes were started, so each block was independent of its neighbours. Grout injection pressures for all stages are given in Table VI.

Second stage (30 to 50 foot) pressures were generally limited to not more than 25 p.s.i. above the corresponding first stage pressure for each block. This increase was slightly less than that corresponding to a gradient of 1 p.s.i. per foot of rock depth and ensured that there was no likelihood of the concrete block and the first 30 feet of rock being displaced together by hydraulic jacking.

The third and fourth stages of the curtain were grouted at higher pressures than the theoretical ones determined from the rule given above. It was felt that the 50 foot thickness of rock consolidated by "A" and "B" hole grouting and the greater depths of the third and fourth stages were adequate insurance against damage by uplift. The higher pressures ensured maximum penetration of grout into all open joints and fissures. No attempt was made to determine whether or not these pressures were likely to increase joint permeability by hydraulic fracturing or elastic deformation, since it was believed they were within the safe range of pressures as defined by current practice.

It is difficult to estimate the actual pressure differential operating in the section being grouted. It is mainly dependent on the following factors: gauge pressure at the collar of the hole; pressure due to the column of grout in the hole above the section being injected; pressure drop due to friction from the unknown wall roughness; back pressure at the section injected due to groundwater or previously injected grout or both; and changes in viscosity of grout with temperature or settling out of cement particles. Pressure losses due to friction effects are probably small because of low grout velocities, so the applied pressure in the injected section is probably somewhat higher than the gauge pressure, due to the column of grout in the hole.

"B" Holes

Consolidation grouting was carried out in each area before concrete was placed, at gauge pressures of 10 p.s.i. for the 0 to 10 foot stage and 30 p.s.i. for the 10 to 30 foot stage. Early in the "B" hole programme three uplift gauges were installed on the left, or western abutment and one on the right abutment. The gauges were checked every 10 minutes or so during "B" hole grouting in the vicinity by a depth gauge calibrated in sixty-fourths or an inch. No measurable movement was detected at any stage and checking of gauges was later discontinued. Most of the consolidation grouting in the centre spillway area, on the right abutment, and in the right thrust-block foundations, and all "A" hole grouting was subsequently carried out without uplift gauges. No displacement of rock was noticed at any time, although conditions for movement

were rather favourable on the right abutment, where the bedding dips at almost the same inclination as the ground surface.

COMPOSITION OF GROUT

Grout with a water:cement ratio of 6:1 by volume was used whenever possible for all grouting. It was considered that the thin mix achieved the best penetration of narrow fissures and at the same time filled the wider fissures in a satisfactory manner. Thin grout continued to flow for longer periods in fissures than did thick grout (say 1:1); therefore larger areas in each joint were grouted and the effective radius of grouting was greater. It was found that cement particles readily settled out from thin grout, forming laminae, which built up as flow continued until the crevice was filled, whereas thick grout tended to set or gel after a certain time, preventing further circulation.

In rare circumstances the grout mixture was changed to 3:1 and in emergencies to 1:1. This was when a hole was taking large quantities of grout at such a rate that pressures could not be maintained and there appeared little chance of the rate decreasing.

Cement used in grouting was required to meet the following size limits:

98% to pass 200-mesh British Standard Sieve

100% to pass 100-mesh British Standard Sieve

No information on composition of cement or grout setting times is available.

ANALYSIS OF GROUT CONSUMPTION

"A" Holes

Results of "A" hole curtain grouting are given in Fig.8 and Tables I, II, and III. The curtain holes are classified by line thickness into three consumption groups:

0 to 0.24 bags* per lineal foot;

0.25 to 0.49 bags per lineal foot;

More than 0.5 bags per lineal foot;

It is evident that the zones of highest grout consumption occurred in the vicinity of the left thrust block (Block 1), and the right thrust block (Blocks 11 and 12). Zones of moderate grout consumption occurred in Blocks 3, 6, 7, 9, and 10. Zones of low consumption occurred in the left abutment at Blocks 2, 4, and 5, and in the right abutment at Block 8. The average cement consumptions (Table I) in Blocks 1, 11, and 12 were 0.69, 0.50 and 1.70 cu. ft. of cement per lineal foot of grout hole. These figures may be compared with those for Blocks 2, 4, and 5 - 0.15, 0.17, and 0.13 cu. ft/lin. ft. respectively, and Block 8 - 0.14 cu. ft/lin. ft. The average cement consumptions for the moderately permeable river-channel section in Blocks 6 and 7 were 0.23 and 0.21 cu. ft/lin. ft., which is fairly low because of the number of holes which were

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* One bag of cement is approximately one cu. ft.

drilled to test the effectiveness of the early grout holes and which took little grout themselves. Grout consumption of primary and some secondary holes in this section was high.

The total footage grouted in the curtain was 8280 feet in 299 stages or their equivalents. The average cement consumption for all holes was 0.78 cu. ft/lin. ft. and the maximum consumption for any one stage was 29.3 cu. ft/lin. ft. (Table I). Table III gives the cement consumption of closure holes. In some cases the closure hole was part of the secondary pattern, in others part of the tertiary or quaternary patterns. The ratio between average lineal cement consumption of closure holes and non-closure holes is considered to be one measure of the completeness with which fissures in the rock have been filled, i.e. the most effective grouting produces the lowest "closure ratio". At Bendora Dam the closure ratios for different parts of the foundations ranged from 0.09 to 1.27 (Table III), indicating a wide variation in the effectiveness of treatment.

On an areal basis, the average cement consumption for Blocks 1 and 12 were 7.1 and 16.9 cu. ft/100 sq. ft. of curtain compared with average for Blocks 2, 4 and 5 of 0.85 cu. ft/100 sq. ft. The average for the entire site was 5.5 cu. ft/100 sq. ft. These figures are derived from Table II. Table II also shows that the overall cement consumption of 69.4 cu. ft/100 lin. ft. for the 0 to 50 foot section of the curtain only decreased to 65.5 cu. ft/100 lin. ft. for the 50 to 100 foot section. In many sections the unit cement consumption was considerably higher in the deeper stages, but the hole density was not sufficient to indicate whether the curtain should have been deeper or whether more 50 to 100 foot stages should have been drilled and grouted.

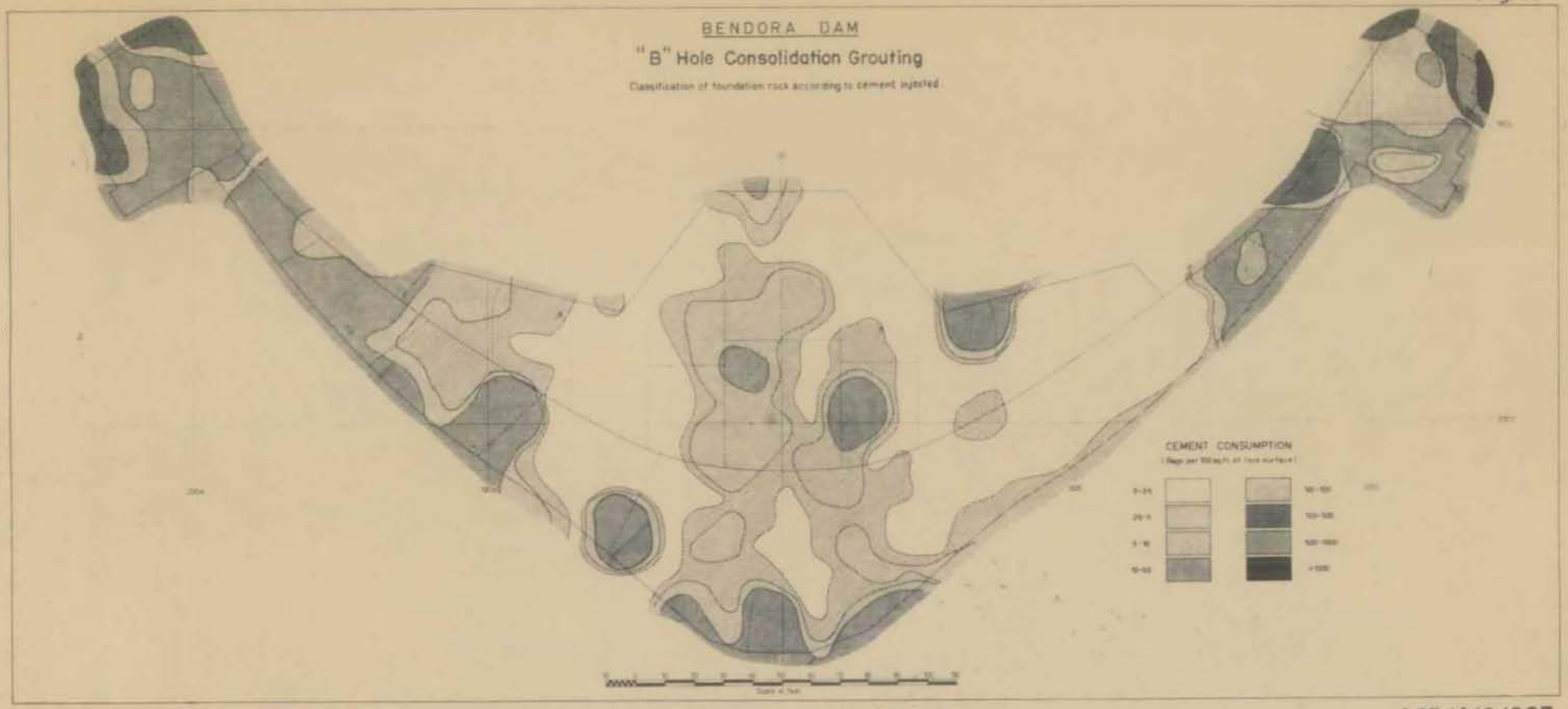
Information on hole density, in terms of lineal feet per 100 sq. ft. of curtain for each section of the foundation, and average hole spacing, is given in Table IV.

"B" Holes

Results of consolidation grouting at Bendora Dam are given in Table VII. The pattern of grout consumption has been analysed in Fig.9, in which the foundation rock has been classified statistically into 8 classes, varying from 0 to 2½ bags of cement per 100 sq. ft. of rock surface (in plan) to more than 1000 bags per 100 sq. ft. The map was prepared by plotting cement consumption in bags at each hole* and then moving a transparent overlay with a square sample area of 400 sq. ft. marked on it across the map on a 10 foot grid. The great majority of "B" holes are 30 feet deep, so no attempt was made to allow for volume of rock grouted, it being considered that a uniform "slice" was treated. At each point the cement consumptions of all holes falling within the sample area were added up and divided by 4 to reduce it to consumption for 100 sq. ft. This figure was then plotted to the grid point under the centre of the overlay. Allowance was made for areas adjacent to the edge of the foundations by reducing the divisor by an appropriate amount whenever the sample area on the overlay overlapped the edge. When the whole foundation area had been covered in this manner the figures at the 10 foot grid centres

* 0 to 30 foot stages of "A" holes were treated at "B" holes in preparing this map. This is why no consumptions greater than 91 cu. ft/100 sq. ft. appear in Table VII, in which only "B" holes are taken into account.

Fig. 9.



May 1965. To accompany Record 1964/140. 155/A16/203.

were "contoured" by equal-value lines according to the eight arbitrary classes. The latter were chosen to avoid over-simplifying the pattern on the one hand and to avoid producing a pattern too complicated for "contouring" on the other. Different sample areas produced patterns which varied in the amount of detail shown.

The cement consumption pattern produced by this method (Fig.9) shows three main areas of high or moderately high consumption, near the top of the left and right abutments and in the former river channel. Two areas of very low consumption occur immediately above the river channel on each side. This pattern corresponds to that in Fig. 8 for the "A" hole grouting. The zone of moderately high consumption in the river channel is irregular but approximately the same width as that revealed by curtain holes. The distribution of high consumption zones on "B" hole grouting corresponds well with results obtained during joint permeability testing of exploratory drill holes, although there are anomalies at greater depths between joint permeability and "A" hole consumptions.

High grout consumption in the upper abutments and low consumption in the lower abutments near the river channel are due to a combination of geological effects. Rock higher up the sides of the valley has been exposed to weathering longer than rock near the valley floor. The nature of the rock and the mainly physical weathering processes acting on it have produced a high proportion of open joints instead of joints filled with clay or other products of chemical weathering. Unloading due to removal of mass by valley erosion has resulted in outward expansion of rock near the surface and opening up of joints in proportion to the period for which the rock has been unconfined. Consequently rock near the river channel, which has only recently been exposed in terms of geological time, has joints which are relatively impermeable and which accept only a little grout whereas rock near the thrust blocks has been exposed much longer and has an abundance of open joints. Slope creep has probably played a part in opening up joints still farther once the initial loosening has taken place. This effect is more marked on the right abutment where the beds dip at the same angle as the hillside.

The original grouting programme envisaged "B" holes at 10 feet grid centres over the entire foundation area, but the geological examination** of beds exposed in excavations in the left abutment showed that thickly-bedded competent quartzite and silicified quartz-sandstone have a lower joint density than the thinly-bedded less competent quartz sandstones, siltstones and ashstone. Moreover, joints in the massive competent beds are more continuous than those in the thin beds, and permitted the former to be grouted effectively by fewer holes. It was also observed that many joints did not continue from one bed to another, particularly in the thinly bedded rocks. The density of grout holes was therefore decreased in massive beds of low joint density and left the same or increased in thin beds with numerous discontinuous joints. For example, the "B" hole density in Blocks 3 and 4, where the foundations consist of thin-bedded quartzite, silicified quartz sandstone, and ashstone, with close irregular jointing and numerous lenses of impermeable sedimentary breccia, is 68 and 79 sq. ft. per hole respectively, whereas the hole density in Blocks 8 and 9, where the rock is

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** L.C. Noakes, personal communication.

massive thickly-bedded quartzite and silicified quartz sandstone with only small zones of close jointing, is 253 and 237 sq. ft. per hole respectively.

In the early stages of "B" hole grouting geological sections were drawn through lines of holes to check whether or not particular holes were long enough to penetrate certain main joints and beds, or whether holes would require packer grouting to ensure that beds isolated between impermeable lenses of sedimentary breccia were thoroughly treated. Such close control was not required during the later stages of the programme in the massive right abutment rock. A record kept of all connections between grout holes and surface leaks that occurred during consolidation grouting shows that connections and surface leaks were common in the upper parts of each abutment and in the river channel, but were rare in the areas of tight rock lower down each abutment.

Grout was lost from right thrust block foundations and from the foundations of Blocks 10 and 11 by leakage at the cliff face on the downstream side of the right spillway apron, a leakage path of up to 150 feet. Very high curtain grout consumption in the 0 to 30 foot and 30 to 50 foot stages was a feature of this section, and was generally greater per foot than consolidation grout consumption, because all the "B" holes are below the sedimentary breccia bed which marks the downward limit of open jointing, whereas some of the higher stages of the "A" holes are above the sedimentary breccia bed and penetrate rock with very open joints.

A high-angle reverse fault with a throw of 4 feet in the left thrust block foundations was responsible for high grout consumption in holes adjacent to it. The fault zone consists of a band of very closely jointed rock with little brecciation, and varies in width from 2 to 4 feet. Some of the grout injected into rock near the fault ultimately leaked out along the upstream wall of the excavation in Blocks 2 and 3.

The area of moderately high grout consumption in the river channel is due to a zone of openly jointed rock which may have controlled the course of the river at this point. Core from drill hole 3 shows that although the rock is quite broken by joints, there is no recognizable fault zone under the river.

In general the results of consolidation grouting show that rock with a joint permeability less than about 400 feet/year accepts very little grout, not more than 10 cu. ft. of cement per 100 sq. ft. of rock surface, and that rock with a joint permeability of up to about 900 feet/year can be treated with moderate cement consumption, not more than about 50 cu. ft/100 sq. ft. of rock surface for 30 foot holes. Beyond this point cement consumption increases rapidly with relatively small increments in joint permeability and is of the order of 90 cu. ft/100 sq. ft. for 1200 feet/year.

EFFECTIVENESS OF GROUTING

The present effectiveness of grouting at Bendora Dam (Dec. 1964) is shown in part by the absence of any extensive leakage at the ground surface immediately downstream from the dam. Minor seepages, each less than a gallon per minute, have developed at several points in the right abutment near river level. Water apparently enters joints and bedding planes in the vicinity of the right thrust block and percolates downwards along main joints until it collects at the impervious sedimentary breccia bed which crops out at the base of the cliff. Minor seepage in the left spillway

apron area totals about 5 gallons per minute. From time to time, a drain hole near the bottom of the left spillway apron produces a pulsating discharge, indicating that a pressure connection exists in the foundation. The pulsations may be due to pressure fluctuations in the stilling basin a few feet lower down when the spillway is operating. No seepage is apparent on either side of the valley away from the dam; it is possible that some is occurring beneath the soil cover without being evident at the surface.

Unfortunately there is no means of observing the behaviour of sub-surface water at the dam. Water-level measurements in exploratory drill holes were discontinued before the dam was completed and there are no instruments for recording uplift pressures in the foundations. Measurement of water table may be resumed in a wing hole on the right abutment.

During consolidation grouting of the foundations four holes were drilled to obtain cores of grouted rock. The results varied from core in which joints showed no evidence of grout to core in which about 50% of the joints were grouted. Most of the grout recovered was still in paste form, having failed to set in the period of about one month between grouting and coring, and much may have washed away with drilling water. No specific reason for the failure to set was discovered - it could have been due to the thin grout mix, type of cement used, or the cold weather prevailing at the time. The four cored holes were later grouted with takes of 1, 37½, 8½ and 7½ bags of cement respectively. Surface leaks occurred from one hole and connections took place between two of the cored holes and other grout holes.

After these attempts, no further coring was done. Reliance was placed initially on water tests after grouting and later on cement consumption of closure holes to indicate the effectiveness of treatment.

ROLE OF ENGINEERING GEOLOGY IN GROUTING

The main results of applying engineering geology to the grouting programme at Bendora Dam were as follows:

1. On completion of exploratory drilling and permeability testing at the site a broad qualitative estimate was made that leakage through the foundations would be low to moderate and that it could be remedied without excessive consumption of grout.

2. Examination of the regularity, density, and continuity of jointing resulted in grout hole spacing being varied to suit geological conditions in different parts of the foundations. The area per hole was increased from about 90 sq. ft. to 115 sq. ft. with a consequent saving in cost.

3. In the initial stages of "B" hole grouting, holes were planned to intersect special geological features such as faults, large joints, beds isolated by impermeable inter-beds, etc.

4. Geological sections were drawn through certain holes in order to investigate the reasons for unusually high grout takes. In some cases geological structures existed which probably accounted for the losses.

5. Cement consumption was analysed in relation to site geology and joint permeability after completion of grouting to define more accurately patterns already indicated, to establish grout consumption rates for different categories of rock, and to relate grout consumption to joint permeability in a quantitative manner. Some of this information may be applicable to other dam sites in similar geological environments.

ACKNOWLEDGMENTS

Approval of the Director General, Commonwealth Department of Works to use information on grouting, and of the Commissioner, Snowy Mountains Hydro-Electric Authority, to reproduce the permeability formula, is gratefully acknowledged.

The sections on grouting technique and grout consumption are based on information supplied by the following officers of the Department of Works, Canberra - Messrs. K.G. Harding, C.A. Tonissen, and J.C. Purcell. The author desires to thank especially Mr. Purcell for his ready and efficient co-operation in collecting and supplying this information.

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A P P E N D I X 1

APPENDIX 1

TABLE 1

"A" Hole Curtain Grouting - Cement Consumption Per Lineal Foot

Section grouted	Total lineal feet grouted[1]	Number of stages or equivalents grouted	Cement Consumption[2] Per lineal foot		
			Total	Mean	Maximum
(Block No.)	(ft.)		(cu.ft.)	(cu.ft.)	(cu.ft.)
1	2080	81	1439	0.69	10.15
2	330	13	50	0.15	1.22
3	225	9	70	0.31	0.98
4	300	12	50	0.17	0.42
5	325	12	41	0.13	0.70
6	480	18	109	0.23	0.97
7	1150	34	242	0.21	2.00
8	275	11	38	0.14	0.38
9	400	14	114	0.28	4.30
10	375	13	100	0.27	2.55
11	465	17	233	0.50	4.15
12	1875	65	3196	1.70	29.30
Crevices near Block 12	0	0	769	-	-
Entire site	8280	299	6451	0.78	29.30

1. Only the newly drilled and grouted section at each stage is included in the total: i.e. 0-30 ft., 30-50 ft., 50-75 ft., 75-100 ft.

2. The volume of one bag of cement has been taken as approximately one cubic foot.

TABLE 11

"A" Hole Curtain Grouting - Comparison of Cement Consumptions for 0-50 ft. and 50-100 ft. Stages

Section grouted	Depth	Area of Curtain	Total lineal feet grouted	Cement Consumption		
				Total	Per 100 Lin.ft.	Per 100 sq. ft.
(Block No.)	(ft.)	(sq.ft.)	(ft.)	(cu.ft.)	(cu.ft.)	(cu.ft.)
1	0-50	10,780	1,780	1,194	67.0	11.0
2	"	2,750	280	47	16.8	1.7
3	"	2,750	200	68	34.0	2.5
4	"	2,750	250	38	15.2	1.4
5	"	2,750	250	29	11.6	1.1
6	"	4,380	380	83	21.8	1.9
7	"	4,380	800	94	11.8	2.1
8	"	2,750	250	30	12.0	1.1
9	"	2,750	350	112	32.0	4.1
10	"	2,750	350	98	28.0	3.6
11	"	2,750	390	170	43.6	6.2
12	"	7,800	1,350	2,638	195.4	33.8
Entire Site	"	49,340	6,630	4,601	69.4	9.3
1	50-100	9,550	300	244	81.3	2.56 ~~2.00~~
2	"	2,750	50	3	6.0	0.19
3	"	2,750	25	2	8.0	0.07
4	"	2,750	50	11	22.0	0.40
5	"	2,750	75	12	16.0	0.44
6	"	5,160	100	26	26.0	0.50
7	"	5,160	350	149	42.6	2.89
8	"	2,750	25	8	32.0	0.29
9	"	2,750	50	2	4.0	0.07
10	"	2,750	25	3	12.0	0.19
11	"	2,750	75	63	84.0	2.29
12	"	11,040	525	558	106.3	5.07
Entire Site	"	52,910	1,650	1,081	65.5	2.04

TABLE 111

"A" HOLE CURTAIN GROUTING - CEMENT CONSUMPTION OF CLOSURE HOLES

Block Number	Cement Consumption			Closure Stages	
	Non-closure stages (cu.ft. per lin.ft.)	Closure stages (cu.ft. per lin. ft.)	Closure Ratio	Number of stages	Footage grouted (ft.)
1	1.00	0.13	0.13	28	730
2	0.22	0.04	0.18	5	130
3	0.30	0.33	1.10	4	100
4	0.15	0.19	1.27	6	150
5	0.14	0.10	0.71	4	100
6	0.26	0.11	0.42	4	100
7	0.31	0.03	0.10	8	400
8	0.14	0.13	0.93	6	150
9	0.42	0.06	0.14	4	150
10	0.35	0.03	0.09	2	100
11	0.73	0.07	0.10	5	160
12(a)	13.33	9.81	0.74	3	90
12(b)	0.82	0.16	0.20	14	425
Entire site including 12(a)	0.82	0.43	0.52	93	2785
" " excluding 12(a)	0.61	0.11	0.18	90	2695

(a) - holes in open-jointed rock above sedimentary breccia bed

(b) - holes in tight-jointed rock below sedimentary breccia bed

TABLE IV

"A" Hole Curtain Grouting - Hole Density and Spacing

Block Number	Hole Density (lin.ft. per 100 sq.ft. of curtain)		Average spacing of 0-50 ft. holes[1] (ft.)
	0-50 ft.	50-100 ft.	
1	16.5	3.1	6.8
2	12.0	1.8	
3	7.3	0.9	10.0
4	9.1	1.8	
5	9.1	2.7	
6	8.7	1.9	6.3
7	18.2	6.8	
8	9.1	0.9	
9	12.7	1.8	8.0
10	12.7	0.9	
11	14.2	2.7	
12	17.3	4.8	4.2
Entire Site	13.5	3.1	7.3

1. The average spacing of stages for the entire site given at the foot of this column was calculated by dividing the total number of 0-50 ft. stages or equivalents into the length of grout curtain measured at the surface (920 ft. approximately).

TABLE V

"A" Hole Curtain Grouting - Concrete Load and First Stage Injection Pressure for each Block

Block Number	Gauge injection pressure 0-30 ft.	Approximate concrete load on foundation rock	
		At start of grouting	On completion of block[1]
	(p.s.i.)	(p.s.i.)	(p.s.i.)
1	45	20	20
2	45	25	25
3	45	30	40
4	50	40	45
5	50	55	60
6	55	80	90[2]
7	55	85	90
8	50	55	60
9	50	40	45
10	45	40	40
11	45	25	30
12	45	20	30

1. The weight of the roadway across the top of the dam and the piers supporting it is not taken into account in these figures.

2. Concrete loads for Blocks 6 and 7 do not include weight of the valve tower which is situated on a gravity block at the heel of the dam.

TABLE VI

"A" Hole Curtain Grouting - Gauge Injection Pressures

Block Number	Depth of Stage			
	30 ft.	50 ft.	75 ft.	100 ft.
	(p.s.i.)	(p.s.i.)	(p.s.i.)	(p.s.i.)
1	45	70	95	150
2	45	70	95	150
3	45	70	100	No holes
4	50	70	110	175
5	50	75	120	175
6	55	75	150	200
7	55	75	150	200
8	50	75	120	No holes
9	50	70	110	175
10	45	70	100	No holes
11	45	70	95	150
12	45	70	95	150

TABLE VII

"B" Hole Consolidation Grouting - Cement Consumption [1]

Block No.	No. of holes grouted	Total lineal feet grouted (ft.)	Developed area of block (approx.) (sq.ft.)	Cement Consumption in cubic feet					
				Total	Per Hole	Per lin. ft.	Max. per lin. ft.	Per 100 sq.ft. of rock surface	Per 100 cu.ft. of rock
1	26	780	2000	1009	39	1.29	10.1	50	1.68
2	7	210	630	192	27	0.91	2.7	30	1.02
3	15	390	1020	161	11	0.41	3.0	16	0.53
4	18	358	1420	290	16	0.81	6.7	20	0.68
5	13	350	1770	89	6.8	0.25	2.3	5	0.17
6	16	480	2530	76	4.8	0.16	0.4	3	0.10
7	15	450	2940	184	12	0.41	1.8	6	0.21
8	7	210	1770	23	3.3	0.11	0.3	1	0.04
9	6	180	1420	6	1.0	0.03	0.1	0.4	0.01
10	10	300	1020	200	20	0.67	6.2	20	0.65
11	9	270	630	572	64	2.12	15.0	91	3.03
12	34	720	2000	679	20	0.94	8.2	34	1.13
L	25	670	2330	96	3.8	0.14	0.4	4	0.14
C	64	1920	8450	412	6.4	0.21	3.3	5	0.16
R	16	480	2330	91	5.7	0.19	1.7	4	0.13
Entire site	281	7768	32,260	4080	14.5	0.53	15.0	13	0.42

L = Left spillway apron

C = Centre spillway apron

R = Right spillway apron

1. This table does not include 0 to 30 foot stages, or equivalents of "A" holes. They are, however, included in the plan in Fig. 9.

A P P E N D I X II

APPENDIX II

The joint permeability formula developed by Mr. Chapple of the Snowy Mountains Hydro-Electric Authority and used in this paper is as follows:

$$U = \frac{Q}{H} \times \frac{16,200}{L} \left(1 + 0.825 \times \log_{10} \frac{L}{Ro}\right)$$

where U = joint permeability in feet per year.

Q = flow out of hole in gallons per minute.

H = effective pressure in test section in p.s.i.

L = length of test section in feet.

Ro = radius of hole in inches.

In deriving the formula it was assumed that the properties of rock affecting its permeability were homogeneous and isotropic and that flow was laminar.

Mr. Chapple points out that the formula implies a linear relationship between applied pressure and flow out of the hole. It is often found from actual tests that the relationship is not linear, which indicates the rock conditions are different from those assumed. One possibility is the existence of a few large fissures with non-laminar flow. For fully turbulent flow, discharge would increase as the square root of the pressure. Mr. Chapple suggests that comparison of the test results with theoretical results may thus give an indication of the structure of the rock.