GEOLOGICAL STUDIES FOR THE PLANNING AND CONSTRUCTION OF TUMUT 2 UNDERGROUND POWER STATION

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AUTHOR'S STATEMENT

This thesis contains no material previously submitted by me for a degree at any University.

To the best of my knowledge it contains no material previously written or published by another person, except when due reference is made in the text.

D.H. Stapledon

22nd August, 1961.

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SUMMARY

Tumut 2 Power Station is being constructed in a group of large chambers, excavated in granitic rocks 800 feet underground beneath the eastern side of the Tumut River gorge.

Geological work associated with the power station has been carried out in three more or less distinct stages:

(1) Reconnaissance and Preliminary Planning Stage

(2) Detailed Investigations for Final Location and Design

(3) Detailed Investigations during Construction.

All these stages are mentioned, but only stages (2) and (3) are described in detail.

In the first stage the geological conditions at several alternative power station sites were investigated by means of surface geological mapping, seismic refraction surveys, and limited amounts of diamond drilling. Geological defects including wide fault zones and extensive rock weathering caused rejection of some sites. This stage ended with the adoption of one of the alternative sites. The adopted site is in granitic gneiss, intruded by a granite sheet 70 to 100 feet wide. Porphyry and dolerite dykes intrude both the granite and gneiss. All of the rocks are extensively jointed and are intersected by several minor transcurrent faults.

Investigations at the second stage commenced with detailed surface mapping on a scale of 1 inch = 50 feet. An exploratory tunnel was driven from the ground surface into the site, which was then further explored by means of several diamond drill holes drilled from the end of the tunnel. The detailed rock structure, and the mechanical properties of the rock mass were assessed from the results of this exploration. The positions and orientation of the main chambers were chosen to make best use of the rock at the site. The rocks were tightly jointed and relatively impervious, and ground water flows into the exploratory

tunnel were small, but at high pressures. The water table is close to the ground surface. Experience from the completed Tumut 1 Power Station had shown rock bolts to be suitable support for similar hard, jointed rock. Rock bolts were therefore adopted as the primary form of support for Tumut 2.

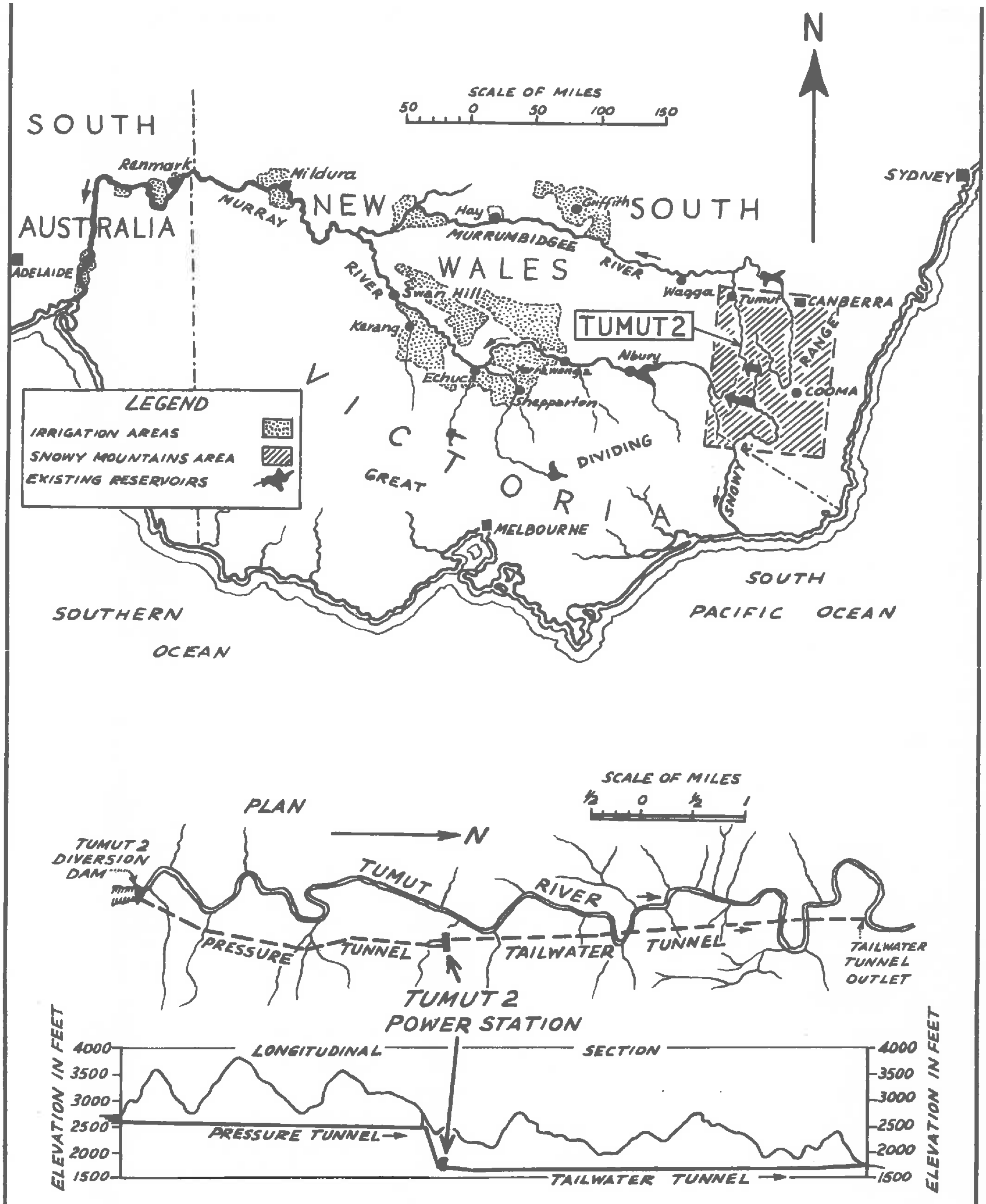
During the preliminary planning and detailed investigation stages attention was also given to the location of materials suitable for use as concrete aggregates. Surface sites for quarrying of bedrock were rare, and mostly proved unsatisfactory due to extensive chemical weathering, and steep, potentially unstable slopes. Suitable deposits of natural sands and gravels were absent. Rock spoil from the excavations was adopted for crushing for coarse aggregate, and sands of Tertiary age were chosen for fine aggregate.

Geological studies during construction confirmed generally that the rock structure and ground water conditions were as assumed for the designs. The excavations were made with no major rock falls, and using rock bolts for support almost throughout. The geological record made during construction has provided valuable data for use when investigating sites for future underground power stations.

INTRODUCTION

The Tumut 2 underground power station is located in the Snowy Mountains of South-eastern Australia (Figure 1). It forms part of the Upper Tumut Development of the Snowy Mountains Scheme, which will develop the water resources of the mountains for hydro-electric power and irrigation. The Upper Tumut works of the Scheme have been described by Campbell, Pinkerton, Bray and Frost (1956).

The station is situated beneath the lower part of the Eastern wall of the Tumut River valley. It is 800 feet vertically below the ground surface, and 350 feet below the level of the Tumut River, which is 1000 feet to the west of the site. Figure 5 and Plate 2 show the



LOCATION AND PROJECT LAYOUT

FIGURE 1

arrangement of the station. The largest chamber is the machine hall, which is 320 feet long and 51 feet wide, and has a maximum height of 120 feet. Access to the station from the surface is by means of a sloping tunnel 3600 feet long.

The first underground power station constructed for the scheme is Tumut 1, which is located 5 miles upstream, and has been in operation since 1958. By means of a 150-foot high concrete dam, the water discharged from Tumut 1 is to be diverted 3 miles through a 21-foot diameter pressure tunnel, and then down steeply inclined twin pressure shafts to Tumut 2 Power Station. When discharged from Tumut 2 turbines, the water will be returned to the Tumut River through a tailwater tunnel 21 feet in diameter and 4 miles long.

Extensive detailed geological investigations have been carried out during the planning and construction of the dam, tunnels, and power station. This paper will describe only the work associated with the underground power station.

The Station was to be constructed within a group of large excavations, the individual size and shape of which were governed largely by the nature of the mechanical and electrical equipment to be installed within them. Unlike most large excavations made in mines, those at Tumut 2 were required to be permanently stable, and it was necessary for design purposes to know what kind of excavation methods would be feasible, and what type of temporary and permanent supports would be required. It was also imperative that the station be located in rock of sufficiently good quality that the cost of excavation, and of temporary and permanent supports could be kept to a minimum.

The location of a sufficiently large mass of suitable rock in which to make the excavations was therefore of primary importance. A detailed knowledge of the geology of this rock mass was then required, so that its behaviour in the excavations could be accurately predicted, and also to ensure that the shape, orientation, and arrangement of the openings could be chosen to make best use of the natural conditions.

ACKNOWLEDGMENTS

This thesis is submitted with the kind permission of the Snowy Mountains Hydro-electric Authority. The views expressed herein are those of the author, and not necessarily of the Authority. All of the geological work for the project was carried out under the direction of Mr D. G. Moye, Head of the Authority's Engineering Geology Branch. Other geologists of the Authority who were engaged on the preliminary investigations of Tumut 2 Project are Mr D. Svenson and Mr C. J. Zilverschoon, and Dr C. C. Wood.

SCOPE OF GEOLOGICAL WORK

The geological work on which this paper is based was carried out as an integral part of engineering studies during the site selection, design, and construction stages of the whole project. Only the work associated with the power station will be described in detail, and this will be dealt with in three main sections, covering the above-mentioned stages. The site selection and design were carried out intermittently from early 1952 until early 1957 at which time the author was only partly occupied on Tumut 2 Project. During construction, which commenced in July 1958 and is still in progress (November 1960), the author has been engaged mainly on geological work for this project.

Laboratory and field studies of the natural state of stress in the rock at the site are briefly mentioned. These were not carried out by the author, but formed an essential part of the design and construction stage studies.

REGIONAL GEOLOGY

Snowy Mountains

The geology of the Snowy Mountains Area has been briefly described by Moye (1955). He has also made more detailed descriptions of the geology of the Upper Tumut Region (1958 and 1960).

The mountains are composed mainly of Paleozoic granitic and metamorphosed sedimentary rocks, which form several dissected plateaus at various elevations between 4000 and 7000 feet above sea level. The present topography results from the uneven uplift of an ancient peneplain, probably of Cretaceous age. The uplift, which was accompanied by considerable faulting and flexuring of the rocks, probably occurred in several stages and culminated in late Tertiary time. On several of the plateau surfaces there occur remnants of nearly horizontal basalt flows, which overlie and are in some places intercalated with river and lacustrine sediments of early Tertiary age. The extrusion of the basaltic lavas appears to have been connected with tectonic disturbances during an early stage of uplift.

Escarpments which separate the dissected plateaus trend generally North or North-east, and are considered to be the surface expressions of the faulting. The main rivers commence in the undulating plateau country, but rejuvenation which followed the uplift has in most cases caused them to become deeply entrenched within a few miles downstream from their sources.

On the plateaus and on the valley slopes the granitic rocks are generally variably weathered to depths of 100 to 200 feet below the ground surface. The depths of weathering are usually least near the valley floors, because the streams are youthful and actively downcutting, and commonly expose fresh or slightly weathered rock along their banks.

Tumut 2 Area

The Tumut River flows generally northwards in the Tumut 2 Project area, and is entrenched 2000 to 2500 feet into the western margin of the Kiandra Tableland, as shown on Plate 1 and Figure 2, Section A-B. The river valley is straight in broad outline, but in detail the river course is quite sinuous, consisting of a series of short, straight sections, sharp bends, and loops. It has been suggested (Moye, 1958) that the ancestor of the Tumut River was located two miles to the East of the present course, in an ancient valley now occupied by early Tertiary sediments and the Eight Mile Basalt. This would indicate that the present valley has been formed since early Tertiary time. Its youthful nature is shown by Plate 1. In the power station area and further upstream the side slopes range from 35^{o} to 50^{o} near the river, becoming flatter at higher levels. The river flows generally as rapids through a boulder-filled channel from 70 to 150 feet wide, with fresh or slightly weathered outcrops forming its banks, and commonly extending as cliffs for 100 feet or more upslope. Fresh or partly weathered rock outcrops are also abundant in the rocky, V-shaped gorges cut into the valley sides by the main tributary creeks. Signs of increasing maturity downstream from the power station area include slightly flatter side slopes, and widening of the valley floor, which is partly occupied by terraces of boulder gravels. Outcrops of fresh rock are much less abundant there than in the upstream section.

Rock Types and Their Occurrence

Figure 2 shows granitic rocks of several types forming most of the project area. The most abundant of the rocks is a group of granitic gneisses and granulites known as Boomerang Creek Granitic Gneiss, believed to be of Ordovician age or older. A second group, termed Happy Valley Granite, includes several lens or sheet-like masses, which appear to have intruded the granitic gneisses. In a one third to one half mile wide zone adjacent

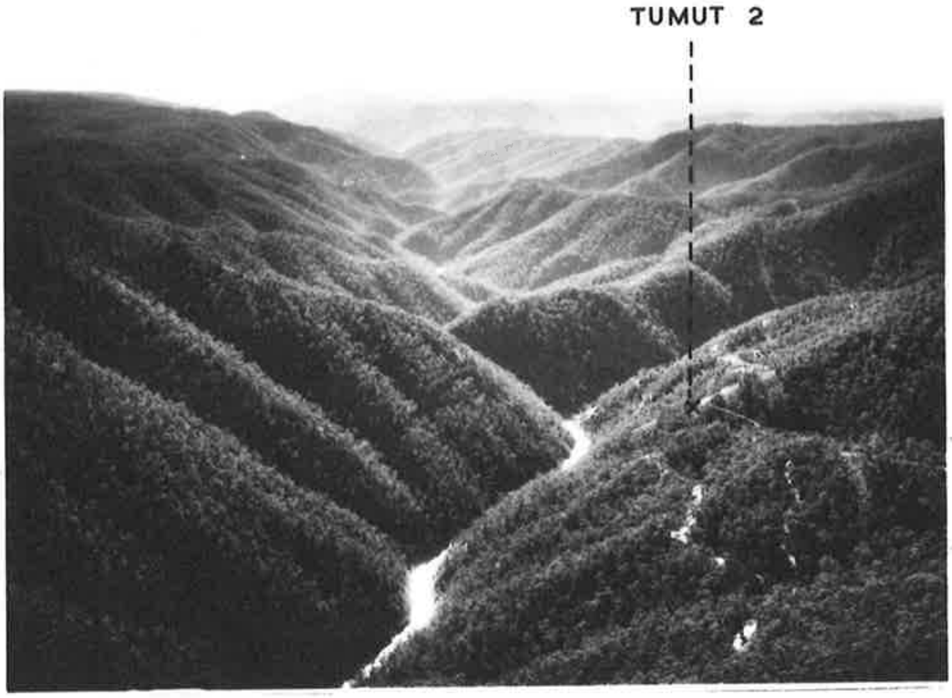


PLATE I. View of Tumut 2 Power Station site, looking North along the valley of the Tumut River. The station is located 600 to 800 feet vertically below the ground surface, in the area marked X.

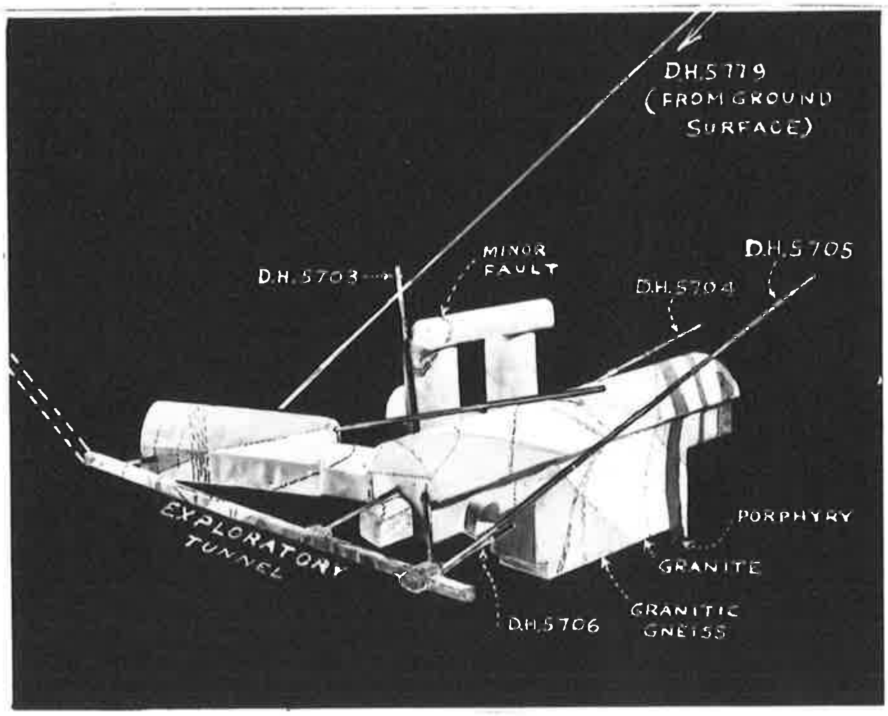


PLATE 2. Painted cardboard model, used during design stage exploration for Tumut 2 Power Station.

to their eastern boundary, the rocks of both groups have developed a pronounced gneissic foliation, and are intruded by coarse to very coarse grained gneissic granite. Highly folded sedimentary and volcanic rocks of Ordovician age, which include slate, siltstone, tuff, and andesite, lie to the East of this gneissic zone.

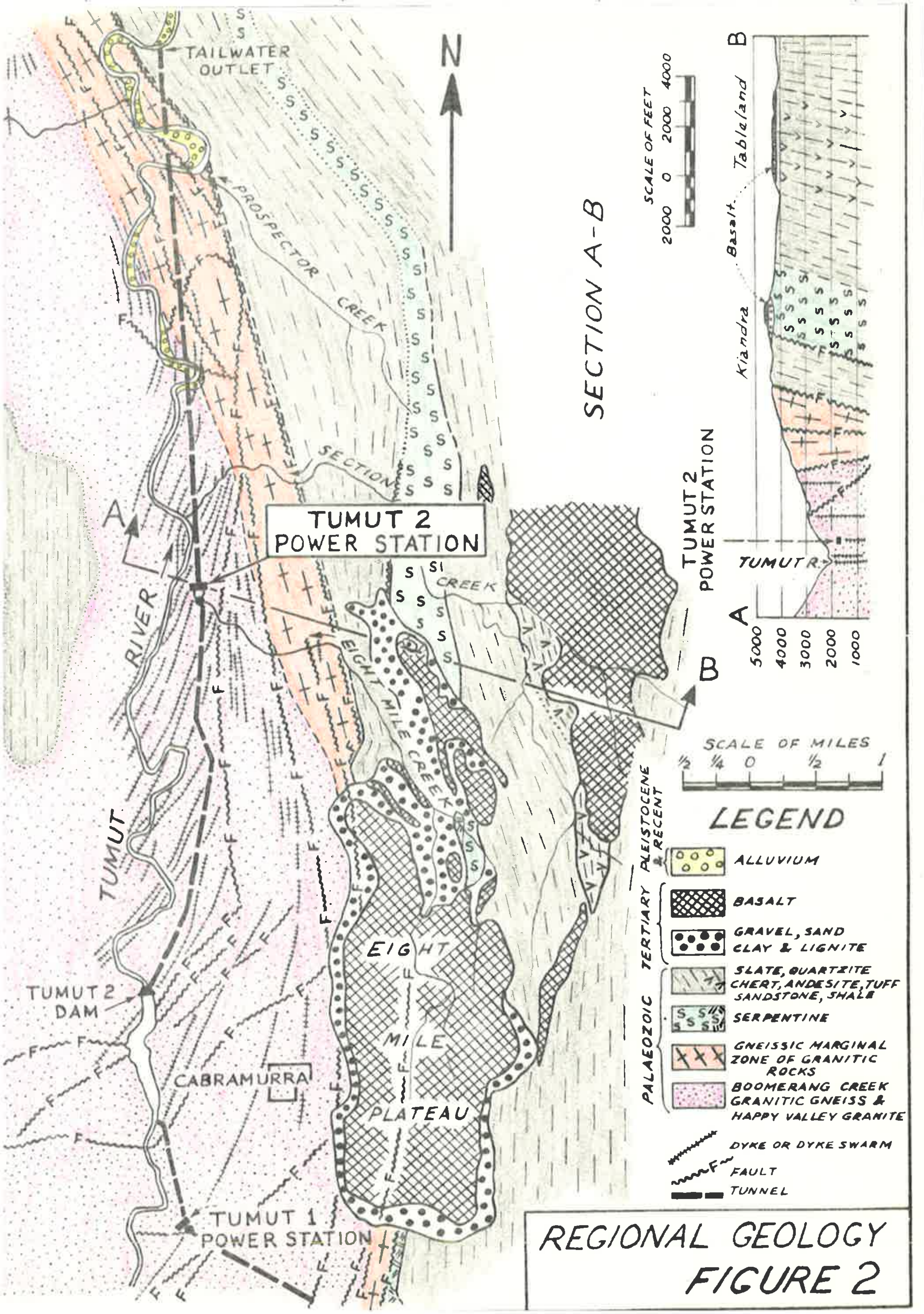
Numerous nearly vertical dykes have intruded all of the granitic rocks. The dykes are remarkably parallel, and occur either singly or in swarms. They mostly strike N.20^{o} to 40^{o}E, in the southern part of the area, curving to N.20^{o} to 30^{o}W in the Northern part. The dykes are of several rock types. Porphyries of acidic composition are the most abundant, and appear to be the oldest group. They appear similar macroscopically to lavas to Devonian age which have a widespread occurrence 10 to 15 miles to the North, in the Talbingo area. Fine grained intermediate and basic dykes are also common and fall into two groups. The older dykes appear to be contemporaneous with the porphyries, and the younger ones, which are mainly of basaltic composition, are possibly related to the Tertiary basalts on the Kiandra Tableland.

Geological Structure

The area is intersected by many faults, only the largest of which are shown on Figure 2. The gneissic margin of the granitic rocks is a zone of intense shearing and faulting. Several types of structure present in it suggest that deformations have occurred at widely spread intervals of time. Veins of aplite and aplitic granite, some deformed, and others not deformed, occur both along and across the foliation direction in parts of the coarse gneissic granite. These suggest that the first deformation may have been during its emplacement. Intensely foliated zones, hard siliceous mylonites, and zones of unconsolidated, crushed rock and breccia are evidence of several later stages of deformation.

Two main types of faults are present:

(a) Transcurrent faults, which are steeply dipping and



REGIONAL GEOLOGY

FIGURE 2

generally almost parallel to the eastern boundary of the granitic rocks.

(b) Overthrust faults, which mostly strike almost at right angles to the transcurrent ones, and dip at low angles towards the South.

Throughout the entire area, even at great distances from the major faults, the rocks are intersected by numerous slickensided joints and minor faults. The displacement directions shown by the overthrust and transcurrent faults, and also by the majority of slickensided joints, suggest that in the most recent tectonic phases a series of granitic blocks have been thrust upwards and towards the North. The blocks appear to be separated from one another, and also bounded on the eastern side by transcurrent faults.

PRELIMINARY INVESTIGATIONS AND SITE SELECTION

There are many alternative arrangements for the development of the Tumut River downstream from Tumut 1 power station (Olsen, 1955). In the preliminary investigation stage, the geological aspects of each alternative were studied, in order to establish the geological practicability of each of its features, and to determine the nature of the geological problems involved. It was necessary to obtain sufficient geological information about the alternative proposals to enable a realistic comparison to be made between them.

The various arrangements investigated for Tumut 2 project included several with underground power stations, and one with a surface station. The sites for these were located on both sides of the Tumut valley over a distance of about 7 miles. Geological reconnaissance mapping on a scale of 4 inches = 1 mile was carried out over the whole area covered by the alternatives, and full advantage was taken of the fresh rock exposures along and near the banks of the Tumut River, by mapping these on a scale of 1 inch = 200 feet. Geological mapping on this scale was also carried out on the ground surface above several of the power station sites, and in nearby creeks where rocks are exposed. Four of the power-station sites were explored directly by means of diamond drilling, and trenches cut by hand or by bulldozer were used to provide additional surface exposures in areas of few outcrops. The seismic refraction method was used to determine the depths of the weathered mantle at three of the surge tank sites.

These geological studies provided an important part of the data used to determine the most favourable alternative. The principal geological problems recognized were (a) the great depths of weathering of the rocks beneath many of the slopes, particularly beneath spurs, and (b) the presence of numerous major faults in the area, mostly in or near the deformed zone adjacent to the meta-sedimentary rocks. Several of the layouts investigated were geologically inferior due to extensive weathering at their surge tank sites, and in others the underground station sites were located in or near zones of faulting, and rock of suitable quality for economic construction was not present.

Figure 2 shows the arrangement which was adopted. The power station is located on the eastern side of the Tumut valley near the mouth of Eight Mile Creek. The Tumut River in this area flows in a channel 30 to 100 feet wide, with cliffs rising 50 to 100 feet above water level on both banks. Above the cliffs the slopes become flatter (30^{o} to 45^{o}) and are mainly covered by soil and vegetation, and deeply weathered. Eight Mile Creek which has cut a deep, V-shaped gorge into the valley side, flows down to the Tumut River as a series of rapids and waterfalls. It has cut through the zone of intense weathering, and as a result the depths of weathering in the sides of its gorge are less than in most parts of the Tumut Valley at equivalent levels. Fresh rock outcrops almost continuously along the creek banks near water level.

This site for the power station offered several advantages. The gorges of Eight Mile Creek and the Tumut River provided almost continuous rock exposures, from which it was possible to obtain a good initial picture of rock conditions in this area by means of detailed geological mapping (Figures 3 and 4). The rocky nature of Eight Mile Creek channel made it possible to design a surge tank of the overflow type, discharging into the creek. The underground surge chamber was located on the left bank of the creek, taking advantage of the relatively shallow depth of weathering in the gorge. An ancient boulder- and gravel-filled channel near the mouth of the creek provided a relatively flat terrace suitable for a cableyard site.

Granitic gneiss and granite are the most abundant rocks, and are intruded by numerous nearly vertical dykes, which are parallel to one another and to the Tumut River direction. The dykes increase in number and in width as the mouth of Eight Mile Creek is approached. The majority of joints and minor faults exposed in the rocks are vertical or almost so, and strike in two main directions, namely: (i) parallel to the dykes and to the Tumut River direction, and (ii) almost perpendicular to the dykes, and roughly parallel to the valley of Eight Mile Creek. The contacts between the dykes and granitic rocks were in many cases sheared. Although the upper slopes over the site were mainly soil covered and extensively weathered, the quartz-rich porphyries were resistant to chemical weathering, and usually formed outcrops or else were indicated by loose, angular fragments at the ground surface. By objectively mapping all of the surface geological features, as on Figure 3, it was possible to infer several dislocations of the dykes by minor faults of direction (ii). The detailed mapping also confirmed the presence of a 400-foot wide major fault zone several hundred feet to the East of the site. This zone, indicated on Figures 2 and 3, had been suspected from the results of earlier reconnaissance mapping.

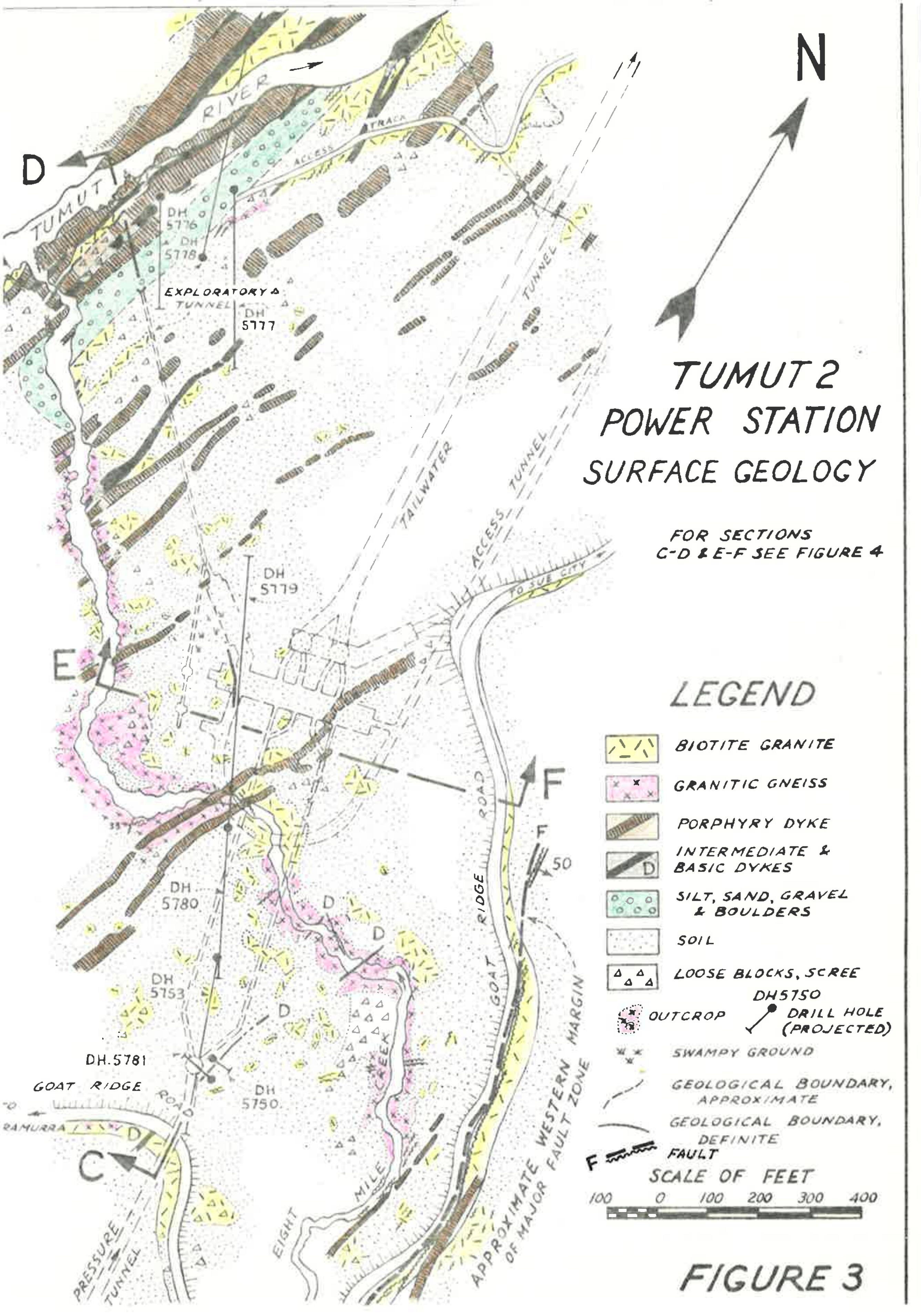
Diamond drilling was carried out to explore two possible sites in this area. The first site, 400 feet beneath the ancient channel near the creek mouth, was explored by three holes, D.Hs. 5776, 5777, and 5778. The other site, 600 to 800 feet beneath the right bank, and 1000 feet upstream from the creek mouth, was explored by only one drill hole (D.H. 5779) from the surface. The projected positions of the drill holes are shown on Figures 3 and 4. The rock in most of the holes was tested for permeability, by isolating with a rubber packer each 20-foot long section drilled, and pumping in measured quantities of water under measured pressures. The core recovery was 100 percent, or almost so, throughout most of the holes. The cores showed the rock at power station levels at each site to be mainly fresh and strong, but intersected by numerous slickensided and coated joints, and several minor faults. The permeability tests gave very promising results. In the case of D.H. 5779, a summarized log of which is shown on Figure 6, most 20-foot long sections of the hole below the top 100 feet were watertight, and other sections showed small leakages of up to a maximum of 0.8 gallons per minute with a pressure of 200 pounds per square inch applied at the surface. This was taken to indicate that most of the joints were tightly closed, even though they were slickensided and in some cases coated with soft materials.

The site 1000 feet upstream from the creek mouth was adopted, partly because the rock was more uniform and contained fewer dykes than that near the creek mouth. There were also several engineering advantages.

DESIGN

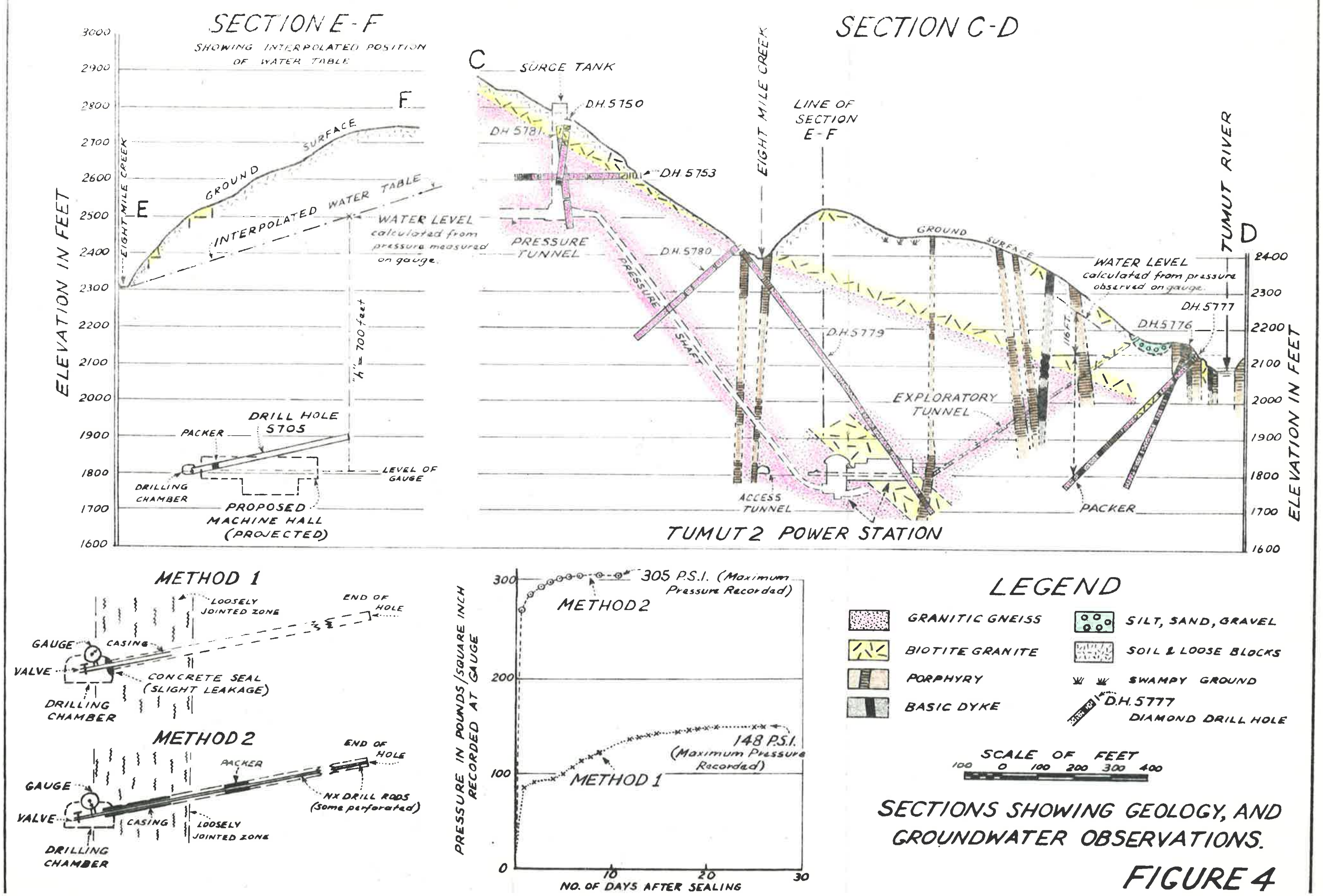
The Preliminary Layout

J. A. S. McLeod (1958) classifies underground hydro-electric power stations into two types, based on the cross-sectional shape of the machine hall. The conventional type has a machine hall with high



N
TUMUT 2
POWER STATION
SURFACE GEOLOGY
FOR SECTIONS
C-D & E-F SEE FIGURE 4
LEGEND
BIOTITE GRANITE
GRANITIC GNEISS
PORPHYRY DYKE
INTERMEDIATE & BASIC DYKES
SILT, SAND, GRAVEL & BOULDERS
SOIL
LOOSE BLOCKS, SCREE
OUTCROP
DH5750
DRILL HOLE (PROJECTED)
SWAMPY GROUND
GEOLOGICAL BOUNDARY, APPROXIMATE
GEOLOGICAL BOUNDARY, DEFINITE
FAULT
SCALE OF FEET
100 0 100 200 300 400
TUMUT RIVER
ACCESS TRACK
EXPLORATORY TUNNEL
DH 5776
DH 5778
DH 5777
DH 5779
DH 5780
DH 5753
DH.5781
DH 5750
TAILWATER
ACCESS TUNNEL
TO SUE CITY
GOAT RIDGE ROAD
GOAT RIDGE
EIGHT MILE CREEK
PRESSURE TUNNEL
APPROXIMATE WESTERN MARGIN OF MAJOR FAULT ZONE
RAMURRA
D
E
F
C
50

FIGURE 3



SECTIONS SHOWING GEOLOGY, AND GROUNDWATER OBSERVATIONS.

FIGURE 4

vertical walls and an arched roof. The other type which is less common, has a machine hall of roughly semi-circular section, and was developed primarily for construction in poor rock where the construction of high vertical walls was not feasible.

At Tumut 2 the results of investigations carried out during selection of the site indicated that it would probably be feasible to construct a chamber of the vertical-walled type. This was very desirable from the engineering viewpoint. The design stage work commenced with the preparation of a preliminary design or "layout" for a station of this type, using all of the relevant geological data obtained during the earlier work. Thus the positions of the chambers in this preliminary layout were selected after careful consideration of both the surface exposures and the core of D.H. 5779, and of the correlation between them. Likewise the orientation of the main chambers was chosen so as to be best suited to the joint pattern in the rocks. Because of the lack of persistent and uniform foliation in the granitic gneiss, it was not possible to determine the true orientation of joints and minor faults in the core of D.H. 5779. The orientation of the chambers was therefore chosen mainly on the basis that the two nearly vertical joint sets (i) and (ii) seen in surface exposures would be the most prominent ones at the power station levels. It was considered undesirable that the vertical walls of the chambers should lie parallel or nearly so to either of these main joint sets, and so the long axes of the chambers were to bisect the roughly 90° angle between them.

Objectives of Geological Work

The preliminary layout was still however of a tentative nature, and it was appreciated that during the detailed design stage it would be possible to make limited adjustments in the power station location and orientation, and in the arrangement of the various chambers. Thus the main objective of the geological work was to obtain very detailed information on the rock structure, so that the design finally adopted could make best use of the rock in the limited area available at this stage. The detailed knowledge of the rock was also necessary for the design of the engineering structures, particularly of the temporary and permanent supports for the chambers, and of means for ground water drainage.

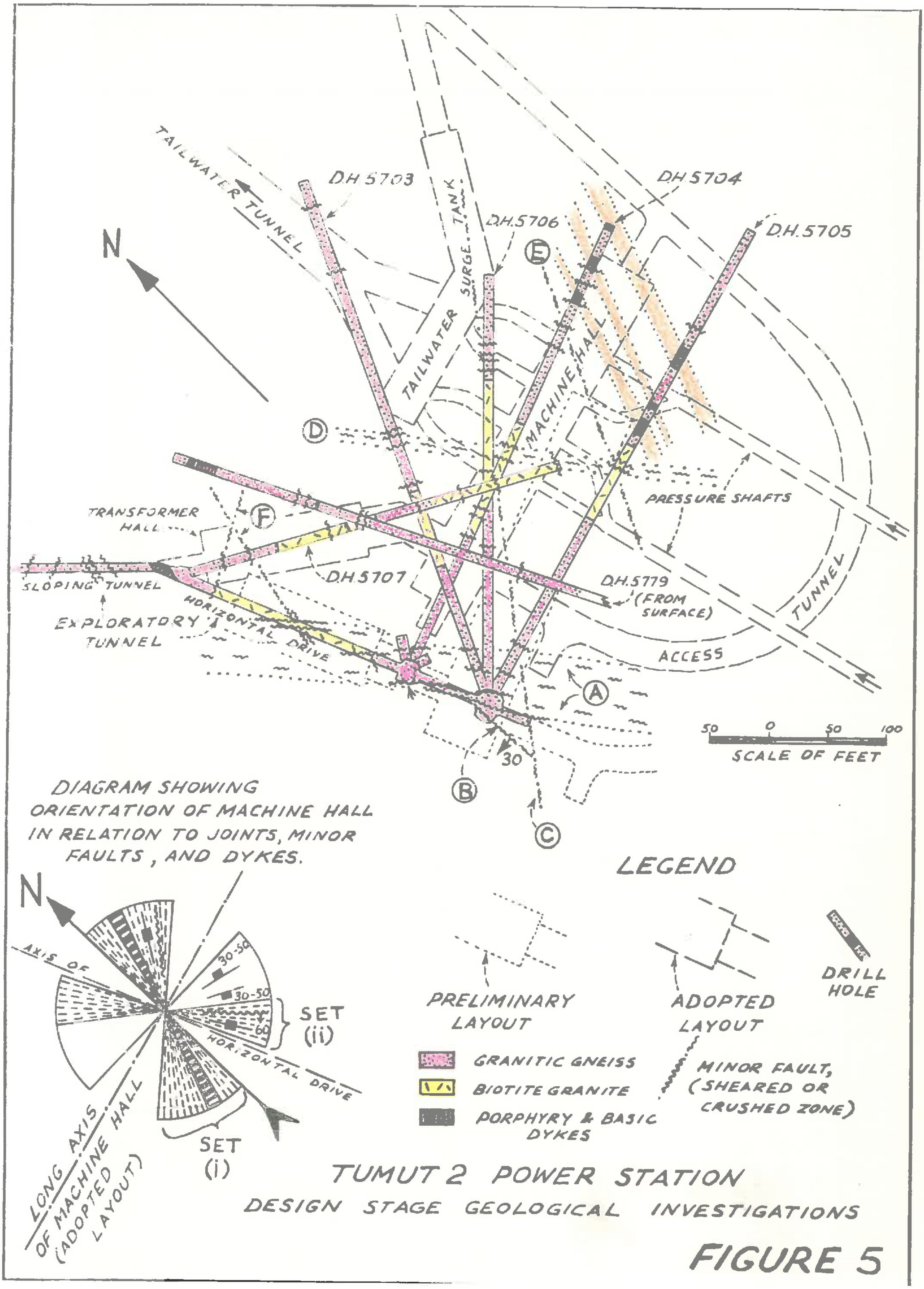
Detailed Exploration

It was decided to drill no further holes from the ground surface into the site, partly because of the great lengths of the holes required, and partly because of the extremely difficult access to drilling sites in Eight Mile Creek gorge. An exploratory tunnel, 8 feet high and 6 feet wide, was commenced from the cableyard site near the mouth of Eight Mile Creek, and driven into the transformer and machine hall sites of the preliminary layout as shown on Figure 5. It was proposed that the exploratory tunnel should serve as a pilot excavation for the cable and ventilation tunnel of any layout adopted at the site. The main advantages of the tunnel as a means of exploration were as follows:

(i) In a tunnel driven into the site the actual behaviour of the rock when excavated by blasting could be observed.

(ii) The attitude of individual joints and minor faults, and their persistence, could be observed. This is usually not possible from diamond drill cores. The tunnel also showed the character of the joint surfaces, the presence and nature of soft joint coatings and very narrow crushed seams, and any separation of the joint planes. These features of the rock mass, which were most important in any consideration of the rock behaviour in a large opening, were difficult to assess from diamond drill cores alone.

(iii) A better assessment of the sources and quantities of ground water inflows could be made from the tunnel than from diamond



TUMUT 2 POWER STATION

DESIGN STAGE GEOLOGICAL INVESTIGATIONS

FIGURE 5

drilling, and actual measurements of groundwater pressures at the power station levels could be made.

(iv) Measurements of the natural state of stress of the rock at the site could be made in the tunnel.

(v) The exploratory tunnel provided access into the actual rock mass in which the excavations were to be made, which could therefore be inspected by representatives of tenderers for the construction of the station. It was considered that such inspections would reduce undertainties in the minds of tenderers when assessing rock and groundwater conditions at the site.

It was undesirable to extend the exploratory tunnel too far into the site while the final positions of the proposed large chambers were not fixed. To extend the geological picture obtained from the exploratory tunnel and D.H. 5779, five gently sloping diamond drill holes, D.H.s 5703 to 5707 (inclusive) were drilled into the site from chambers excavated near the end of the tunnel. The positions of the holes are shown on Figure 5 and Plate 2. One hole, D.H. 5706, was sloped downwards at 11^{o}, and the remaining four were sloped upwards at angles between 7^{o} and 12^{o}.

To assist in the interpretation of the geological data from surface mapping, diamond drilling, and the exploratory tunnel, a geological model of the area on a scale of 1 inch = 50 feet was constructed. The geology was plotted onto thin aluminium rods which represented the diamond drill holes, and also onto thin perspex sheets which showed the proposed engineering structures. Plate 2 shows another model on a scale of 1 inch = 16 feet, which was initially constructed by the design engineers. The factual geology from the diamond drill cores was plotted onto the rods which represented the drill holes, and the interpreted geology was painted onto the cardboard scale model of the proposed chambers. As well as providing a useful means for the interpretation of the detailed geology, this model allowed the design engineers to see a clear three-dimensional picture of the geology, and of its relation to the complicated group of openings proposed.

Exploratory Tunnel

The exploratory tunnel, shown on Figures 3 and 4, consisted of a 644-foot long sloping section, and a 336-foot long horizontal drive. It passed mainly through granitic gneiss, intruded in two places by biotite granite, and also intruded by several porphyry and dolerite dykes. The first granite met was in the top 150 feet, and formed part of a thick sheet of granite which occurs near the ground surface over most of the site. The contact between this granite and the gneiss below it was obscured by a porphyry dyke which intruded both the gneiss and granite. The other section of granite was met in the horizontal drive, when this passed through the proposed transformer hall of the preliminary layout. The contacts between the granite and gneiss in this section were poorly defined and gradational over several feet. Several of the dykes had tight, intrusive contacts with the granitic rocks, and showed narrow irregular "stringers" passing into the latter. In other places the rock along and near the contacts was flaky due to shearing. All of the dykes and most of the minor faults met by the tunnel were found to correlate directly with exposures mapped on the ground surface above, and along the banks of Eight Mile Creek.

Below the top 20 feet the rocks were almost entirely fresh. They were intersected by numerous joints, but as suggested already by diamond drilling in the area, the joints were mostly tightly closed, and cemented by slickensided veneers of chlorite and calcite. The majority of joints belonged to the nearly vertical sets (i) and (ii), which were prominent in the surface exposures. The diagram on Figure 5 shows these two main joint sets, and their relation to the horizontal drive portion of the exploratory tunnel. Very few gently dipping joints were observed in the tunnel. Several minor faults which were met were parallel to joint set (i), i.e. parallel to the dykes and almost at right angles to the tunnel. These were mostly sheared zones of the flaky but firm type shown on Plate 11, and caused little or no overbreak or instability in the tunnel. The last 250 feet of the horizontal drive was intersected by several minor faults parallel to joint set (ii). These faults, which are shown in plan on Figure 5, consisted of narrow crushed seams, of the type indicated on Plate 12. Water seepages and very small flows issued from these seams, and the adjacent granitic rocks were generally partly softened due to chemical alteration. The soft nature of the material forming the seams, and the shallow angles at which they intersected the drive, caused this section of the tunnel to be more loosened by blasting and to have greater overbreak than most of the sloping section.

Except for the short weathered section near the portal, the tunnel was driven entirely without support. In most places, scaling down of the surface layer of loosened rock after blasting revealed compact rock with tightly closed joints, which could not be scaled further with a pick or crowbar.

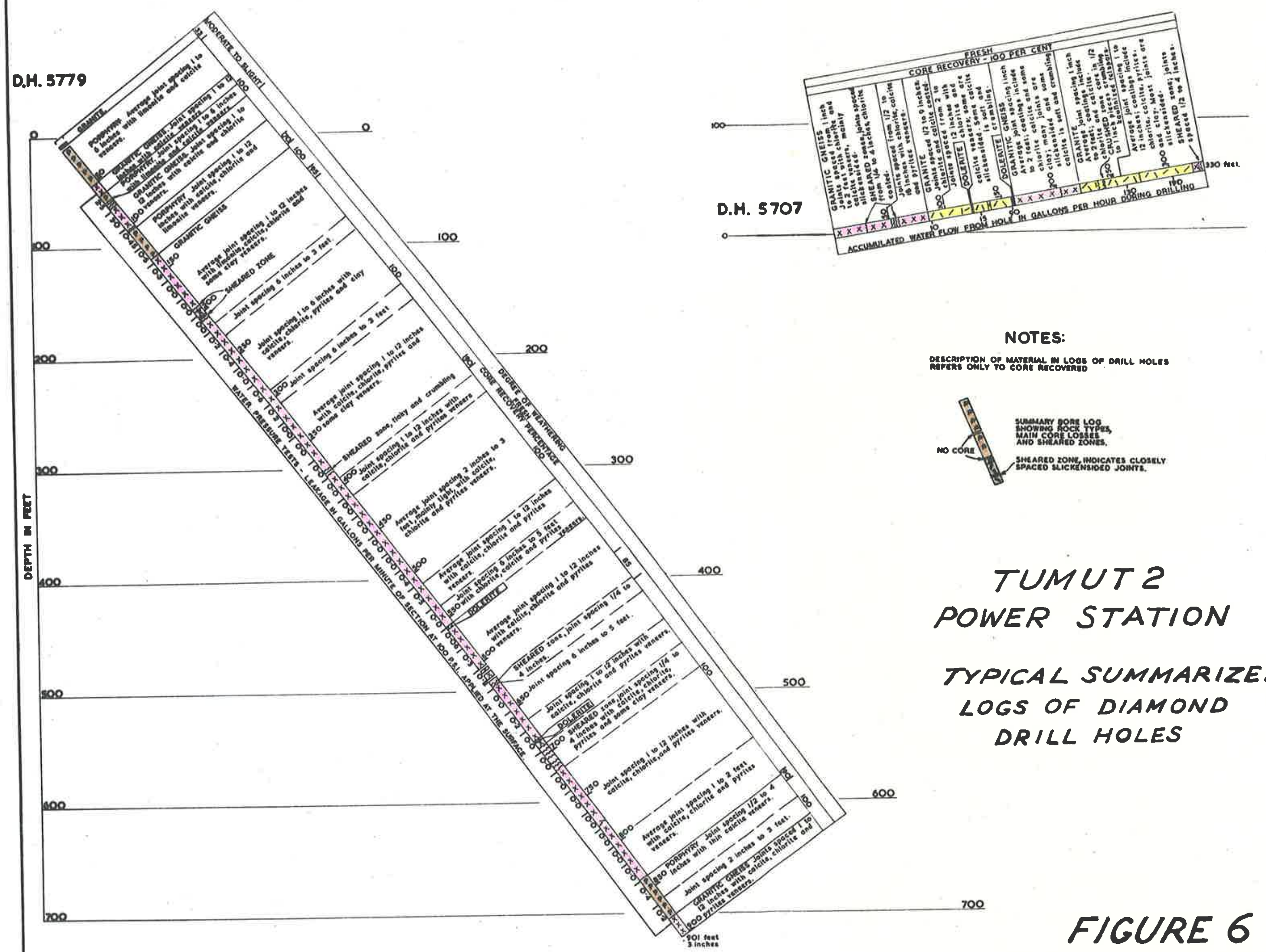
Detailed Geology from Design Stage Work

The exploratory tunnel and diamond drill cores were logged in detail on a scale of 1 inch = 10 feet. Summarised logs, of which those shown on Figure 6 are typical, were also prepared. The geological picture obtained from the results of diamond drilling, as well as from the tunnel, is shown on Figure 5 and Plate 2.

Rock Types

The most abundant rock was granitic gneiss. A 70- to 100-foot wide sheet of biotite granite passed obliquely through the site, DRILL HOLE 5779, sloped at 52° downwards from the left bank of Eight Mile Creek, passed over the roof of the proposed Machine Hall.

DRILL HOLE 5707, sloped 10° upwards from the Exploratory Tunnel, passed through the proposed Transformer Hall.



TUMUT 2 POWER STATION

TYPICAL SUMMARIZED LOGS OF DIAMOND DRILL HOLES

FIGURE 6

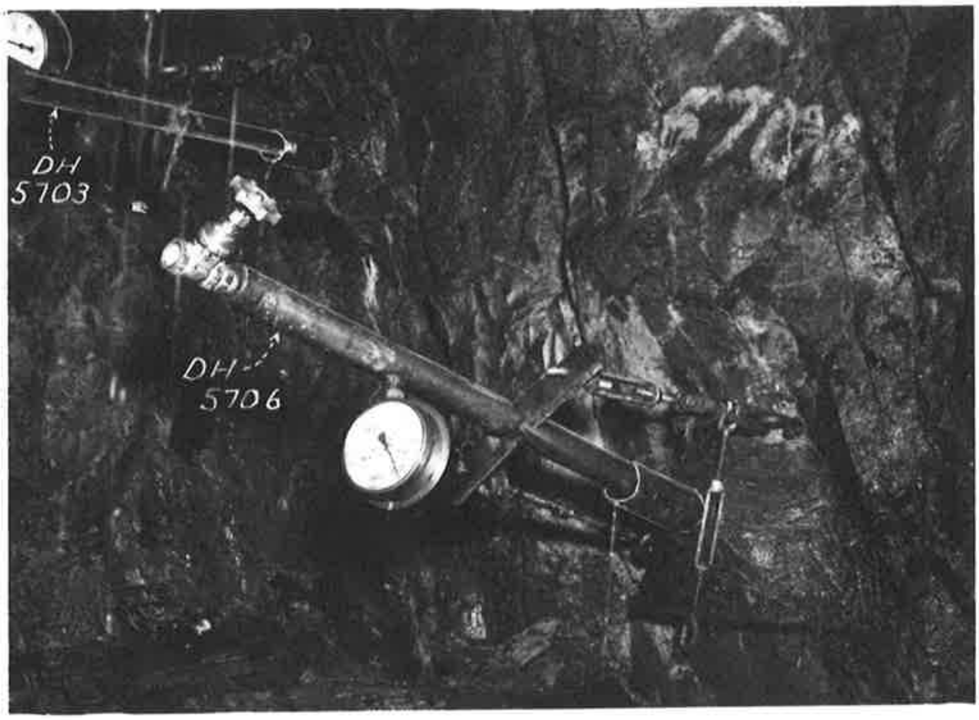


PLATE 3. Drill Holes 5703 and 5706, sealed off by means of rubber packers and fitted with valves and pressure gauges.

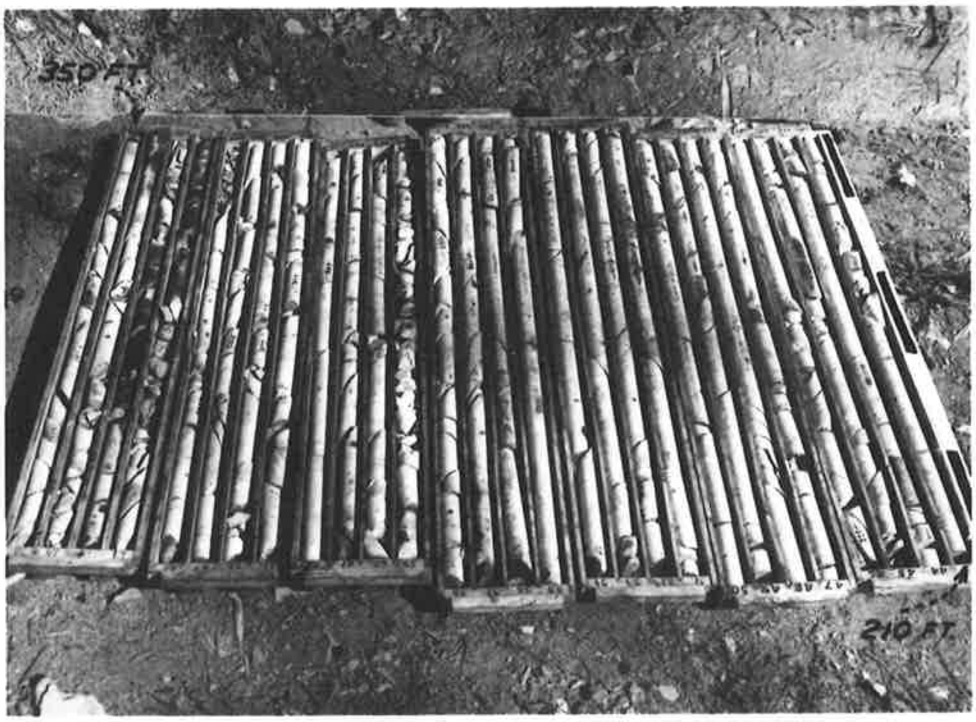


PLATE 4. Core from Drill Hole 5704 between 210 and 350 feet. This hole passed through and close to the roof of the Machine Hall. The staff on the right hand edge is graduated in feet.

and was dipping approximately 50^{o} towards the north. The boundaries between the granitic gneiss and granite were gradational over distances of 2 to 10 feet. Three vertical porphyry dykes passing through the south-eastern part of the site were intersected by D.H.s 5704 and 5705. These dykes ranged in width from 5 up to 15 feet, and in most places in the cores their contacts with the granitic gneiss were intrusive. Another porphyry dyke, 10 to 15 feet wide, passed through the bend in the exploratory tunnel, and was also intersected near the bottom of D.H. 5779, the hole drilled previously from the ground surface. Two vertical dolerite dykes, 6 to 12 inches and 1 to 2 inches wide respectively, were intersected by the exploratory tunnel near the collar of D.H. 5704, and also by D.H.s 5707 and 5779. Petrographic descriptions of the rocks, and tables showing their physical properties are in Appendix I and II respectively.

Jointing

All of the rocks were intersected by numerous joints. It was evident from both the exploratory tunnel and the drill cores that the character and spacing of the joints were related to a large extent to rock type. In both the granite and granitic gneiss most joints were tightly closed, cemented by slickensided coatings of chlorite, or calcite, or some of each. Some of the calcite coatings were partly crushed, causing them to be very soft and powdery. Thin veneers of crushed rock were also present on some joints. Plate 7 shows a face of a typical joint in granitic gneiss. In the granitic rocks the joints were spaced between 1 inch and 2 feet apart, predominantly about 6 inches in the granitic gneiss, and about 12 inches in the biotite granite. Closer spacings occurred in sheared zones (Plate 11) and adjacent to some crushed seams. Although many of the joints were slightly curved, and intersected by neighbouring joints, most of them appeared to persist for considerable distances, as far as could be judged from the exploratory tunnel. In the porphyry

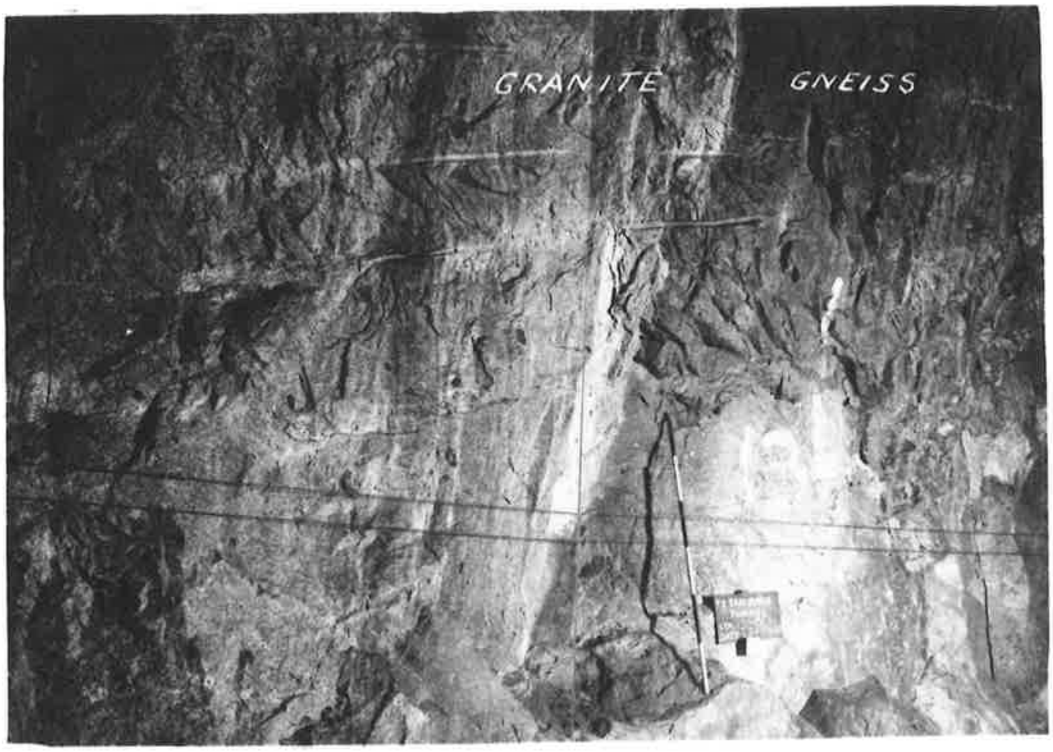


PLATE 5. Schlieren in Happy Valley Granite near its contact with Boomerang Creek Granitic Gneiss, exposed in the Tailwater tunnel.

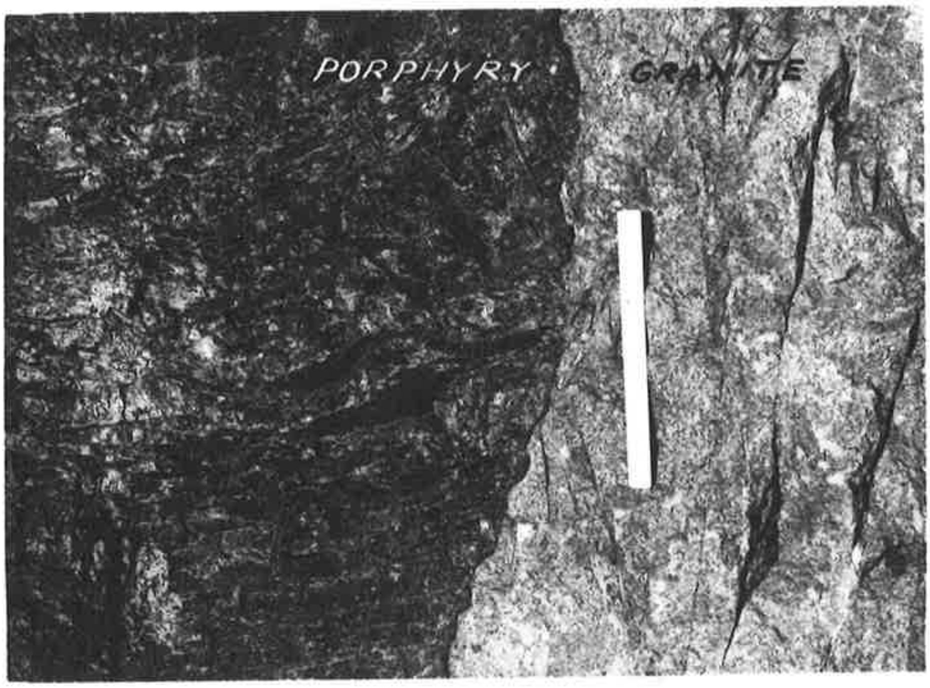


PLATE 6. Intrusive contact between porphyry dyke and Happy Valley Granite, exposed in Penstock Tunnel No. 6.

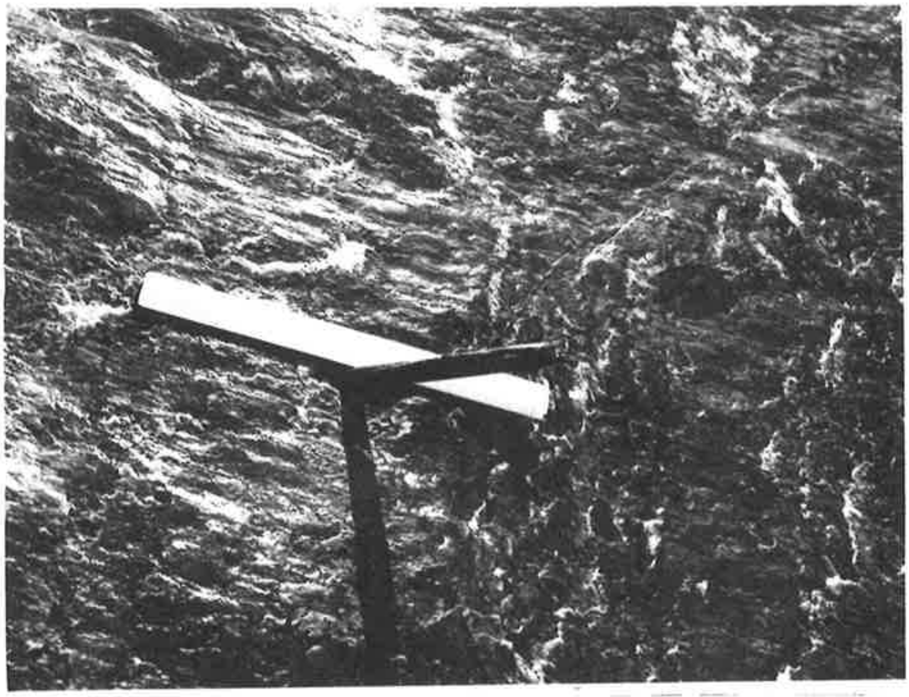


PLATE 7. Face of typical slickensided joint in granitic gneiss. The joint surface is coated with as much as 0·2 inch of calcite and chlorite. Ridges of calcite, on the right hand side, indicate the sense of the most recent movement.

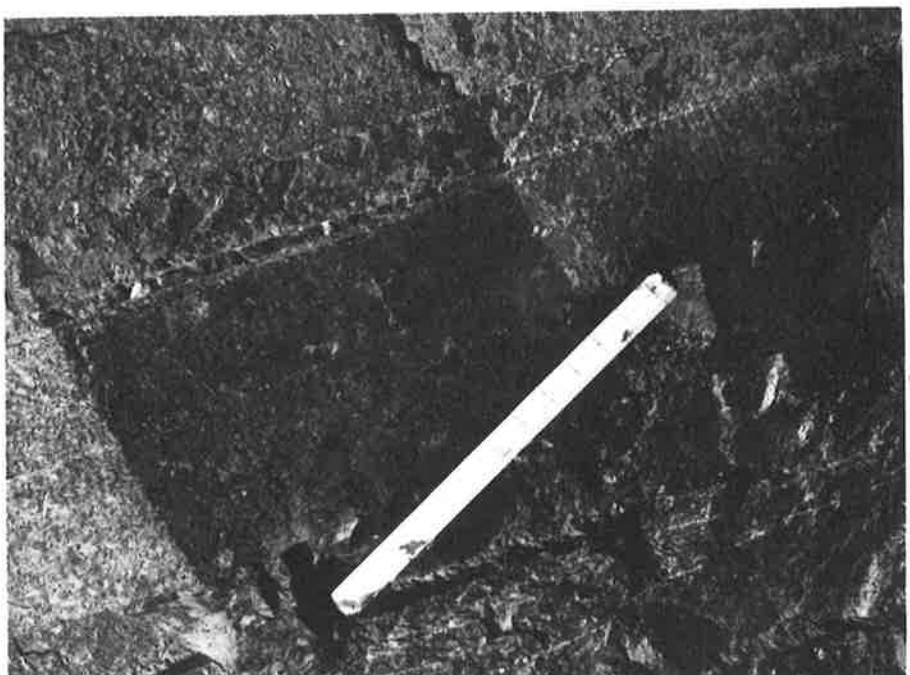


PLATE 8. Typical joints in a porphyry dyke. The joints are very flat and straight and have rough surfaces, which are coated with as much as 0·05 inch of calcite.

(Plate 8), most of the joints were tightly closed, and their surfaces were almost planar when viewed broadly, but in detail were quite rough. Slickensiding was far less common than in the granitic rocks. Joints were usually cemented by calcite, which commonly contained fine clusters of pyritic grains, and a few were limonite stained. They were spaced between 0.5 and 6 inches apart, predominently 2 inches, and most of them terminated near the dyke contacts (Plate 6).

The chlorite and calcite coatings made relatively weak cements, especially when the joints were slickensided, and hence the rocks tended to break along the joint planes rather than across their own fabric. This was evident in the exploratory tunnel, where in most sections more than 50 percent of the excavation surfaces were joint planes. This "percent breakage along joint planes" was considered to be a useful factor for assessing and describing the mechanical properties of the rock mass, and it was recorded during geological logging of the exploratory tunnel, and of all of the later tunnels and excavations for the power station. In the tightly jointed granitic rocks at Tumut 2, this factor was controlled to a large extent by the joint spacing and orientation relative to the excavation surfaces. Due to the jarring received during drilling and on removal from the core-barrel, the rock cores had already parted along many of the weakly cemented joint planes by the time they were in core-boxes ready for logging. It was usually possible however to find "sticks" of core 1 to 2 feet long which would break along one or more weakly cemented joints when struck with a hammer.

Minor Faults

Each diamond drill hole intersected several minor faults, some being of the sheared flaky type, and others being narrow crushed seams. Particular care was taken during drilling to recover as far as possible

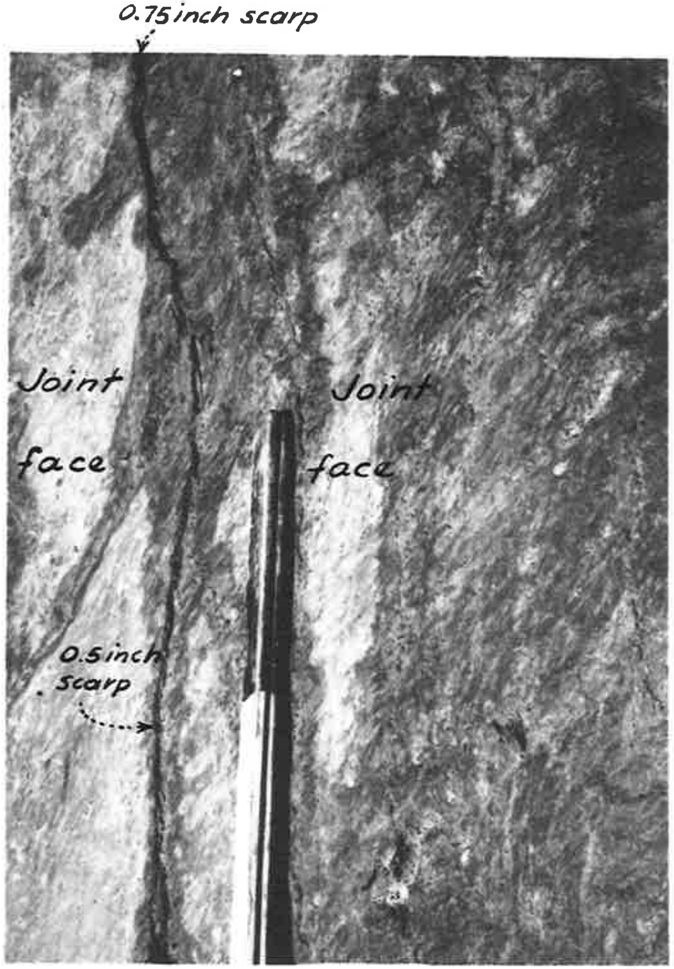


PLATE 10. Relative movement of adjacent joint blocks in granitic gneiss, indicated by the 0.5 to 0.75 inch displacement of a prominent slickensided joint. The staff is graduated in feet.

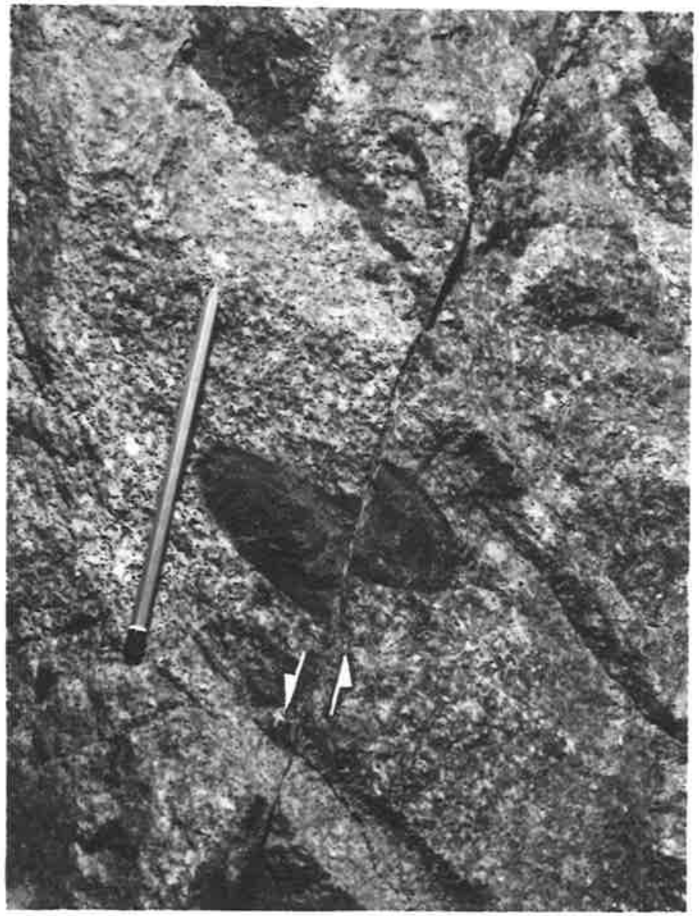


PLATE 9. Relative movement of adjacent joint blocks in granite, indicated by the displacement of a xenolith.

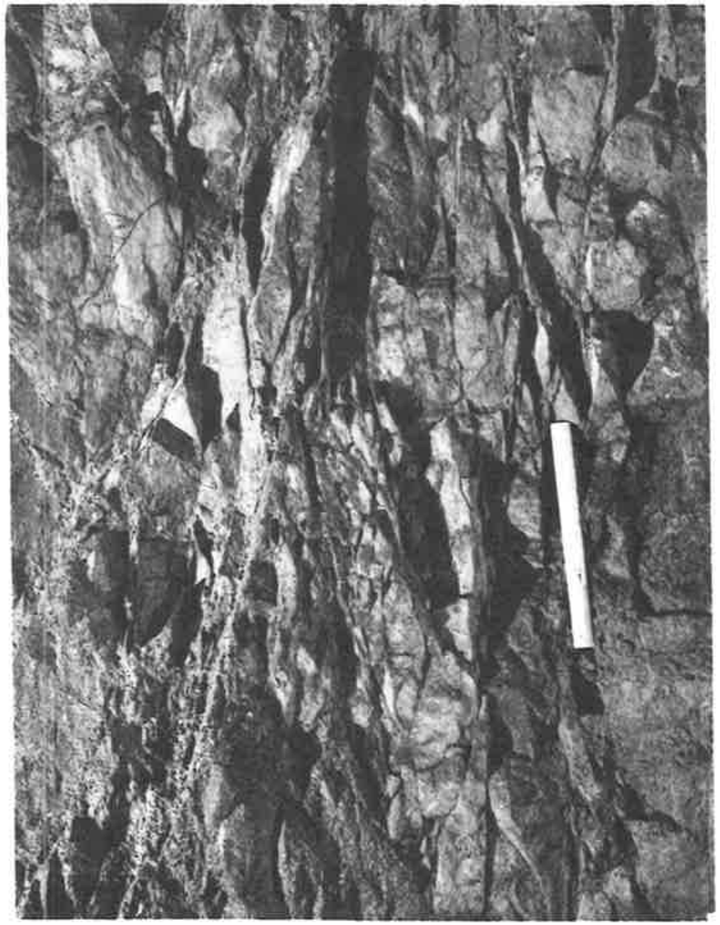


PLATE 11. Sheared zone in granitic gneiss, exposed in wall of Transformer Hall.

SHEARED ZONE: Zone, planar or almost so, in which the rock is intersected by closely spaced, nearly parallel joints, usually slightly curved and intersecting, giving rise to thin, platy or wedge-shaped joint blocks. The joint surfaces are generally slickensided or smooth, and are commonly coated with chlorite or calcite or both.

In most sheared zones at Tumut 2, the joints were tightly closed, weakly cemented by chlorite. The rock in the sheared zones was therefore flakey, but firm and compact, unless loosened during excavation. Behaviour of such zones depended to a large extent upon their orientation in relation to the excavation surface.

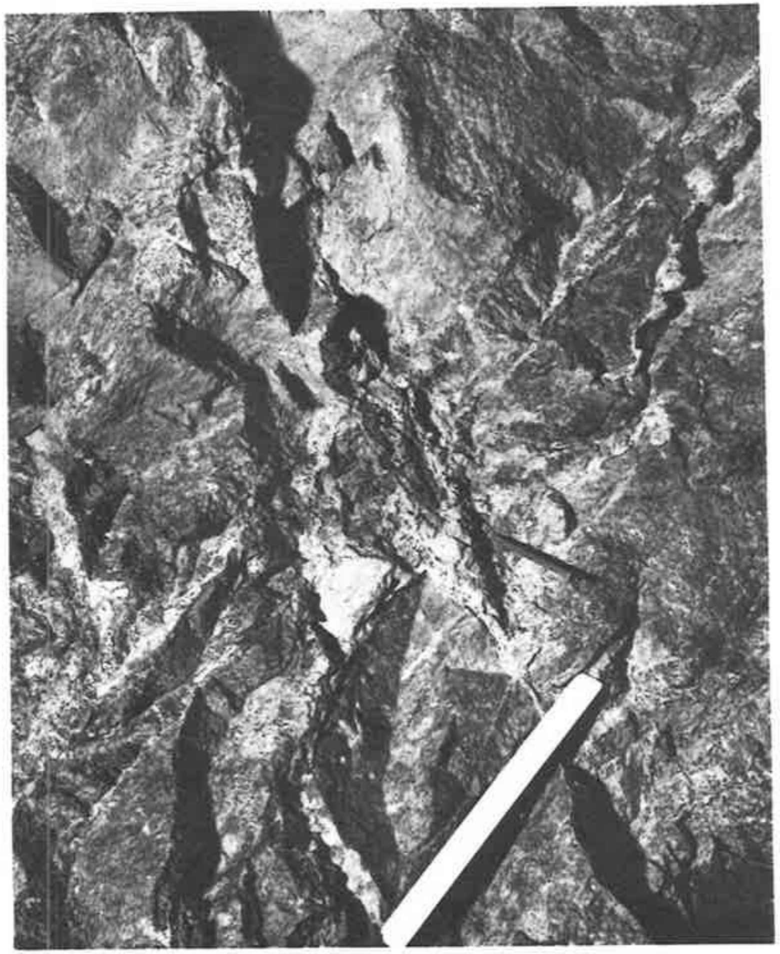
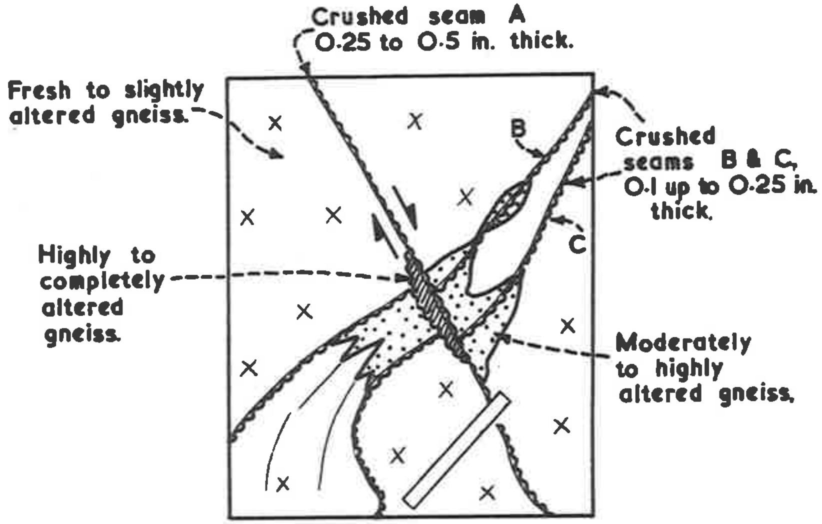


PLATE 12. Intersection of three crushed seams, ranging from 0.1 to 0.5 in., exposed in Tumut 2 Tailwater Tunnel. The gneiss near the seams and their intersection is softened due to chemical alteration.

EXPLANATORY SKETCH
SCALE: 1:16



CRUSHED SEAM OR ZONE: Seam or zone, planar or almost so, in which the rock has been mechanically ground into very small fragments.

Crushed seams in the granitic rocks at Tumut 2 consisted of soft, silty and clayey material, containing very few rock fragments up to 0.1 in.

all the material in the zones and seams. As drilling progressed, the intersections of these minor faults were plotted objectively in plan and on sections, and painted on the aluminium rods on the model shown on Plate 2. At the same time work proceeded on correlation of the seams and zones from one hole to another, and between all of the drill holes and the exploratory tunnel.

It was not possible to orient reliably the drill cores, due to the lack of any sufficiently pronounced and persistent foliation or other directive structure in the fabric of the rocks. However D.H.s 5703 to 5706 were drilled from 60° to 90° to the planes of joints and seams of set (ii), and it could be fairly reliably assumed that structures making angles between 60° and 90° to the axis of their cores belonged to that set. Similarly D.H. 5707 was drilled almost normal to the structures in set (i), which could therefore be distinguished readily in the cores from this hole. As every hole drilled from the tunnel sloped gently, the attitudes of structures intersecting a core at acute angles to the axis of the hole could not be determined reliably. These structures could be either gently dipping, or steeply or moderately dipping, striking at small angles to the drill hole direction. A factor which assisted in the correlation was the rather marked difference in character of the minor faults in sets (i) and (ii). This difference had been seen in the exploratory tunnel exposures.

Chemical alteration of the granitic rocks tended to hamper correlation. In the exploratory tunnel, irregular patches of soft, altered rock occurred at the intersection of narrow seams of set (ii) with other joints and seams, and it was evident that a hole passing through the intersection area would give the impression of a single, wide soft zone rather than the true picture. Such a situation is illustrated by Plate 12.

Final Layout of the Power Station

It was possible eventually to obtain a logical correlation from about 70 percent of the minor faults met by the drill holes and exploratory tunnel, as shown on Figure 5 and Plate 2. The remaining 30 percent which could not be logically correlated included several narrow partly crushed and sheared zones in the porphyry dykes met in D.H. 5704. These zones were limonite stained. The porphyry dykes were quite intact and free of limonite staining, where met 100 feet away in D.H. 5705. The six minor faults or zones of minor faults which could be readily correlated are labelled A to F on Figure 5. With the exception of Zone B, all the structures were steeply dipping or nearly vertical. Zone A, the widest and worst zone, was intersected along the first 40 to 60 feet of D.H.s 5703 to 5706 and along the last 250 feet of the horizontal drive. In the preliminary layout this zone passed along most of the proposed transformer hall, and through one end of the machine hall, and the connecting tunnel between these two chambers. To avoid this zone, and to place the deep central portion of the machine hall in rock virtually free from minor faults, the "adopted layout" shown on Figure 5 was chosen. It was appreciated that it would not be possible to choose any layout which would avoid all of the zones of minor faulting.

Orientation of the Machine Hall

The orientation of the Machine Hall in the adopted layout was chosen after consideration of the directions of both jointing and faulting in the exploratory tunnel, and of the directions of minor faults determined by correlation between drill holes. The diagram on Figure 5 is a replica of one produced for the designers, and shows the adopted machine hall axis in relation to the two main sets of structures and to two other less prominent sets. It was realised that the transformer hall of the adopted layout was oriented almost parallel to set (ii) structures.

However exposures in the exploratory tunnel near the end of the transformer hall, and the results of diamond drilling indicated that the likelihood of set (ii) structures causing major instability in the transformer hall walls was not great.

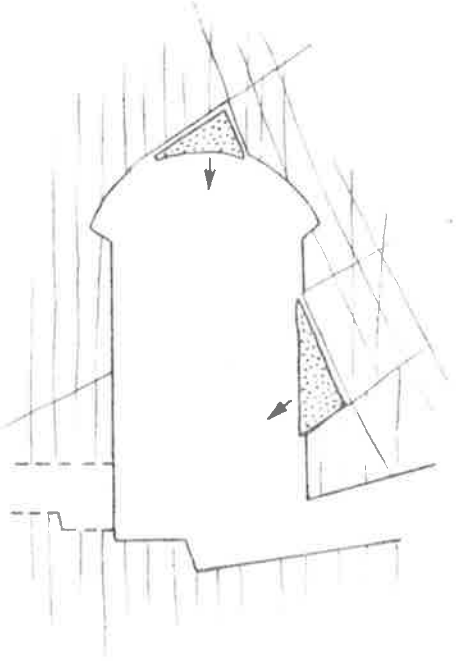
Prediction of Rock Behaviour

An accurate knowledge of the physical properties of the rocks, and a detailed understanding of the structure and mechanical properties of the rock mass, were not in themselves sufficient to enable the designers to predict behaviour of the rock mass around the proposed openings. It was necessary in addition to consider the type and magnitude of the forces which would act upon the rock mass when excavations were made in it. The two main types of force recognised were as follows:

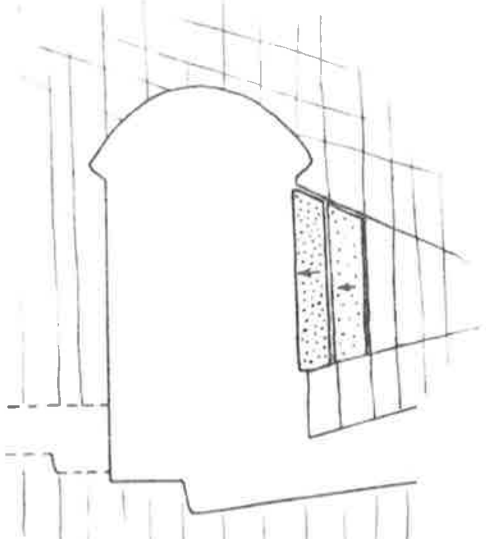
(a) The direct force of gravity acting on loosened rock exposed at the excavation surface: Individual joint blocks could separate from the rock mass when jarred by blasting. If such blocks did not have sufficient interlock with adjacent blocks, they could fall under their own weight. Figure 7, Diagram A, illustrates failure of this type. The block in the roof may be entirely free to fall, but the block in the wall would have to slide or rotate against the joints at its sides and base before fall-out could occur.

(b) Stresses present in the rock prior to excavation: The following notes are based on considerations by D.G. Moye (1958, Reference 16, and 1960, Reference 26) who describes the probable nature of these forces, and their likely effects during construction of a large excavation in jointed rock. He considers that the rock stresses would include those developed due to the weight of overlying rock, and possibly also ones of tectonic origin, and also assumes that before excavation commences, the site would be in a state of equilibrium.

POSSIBLE TYPES OF ROCK FAILURE

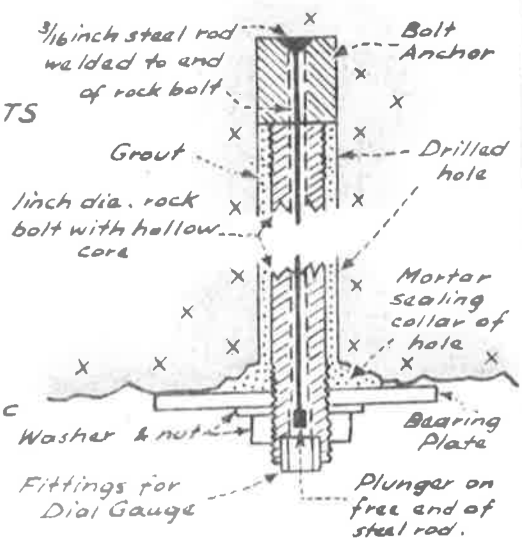


A. FALLOUT OF BLOCKS ISOLATED FROM ROCK MASS DUE TO RUPTURE OF JOINT CEMENTS

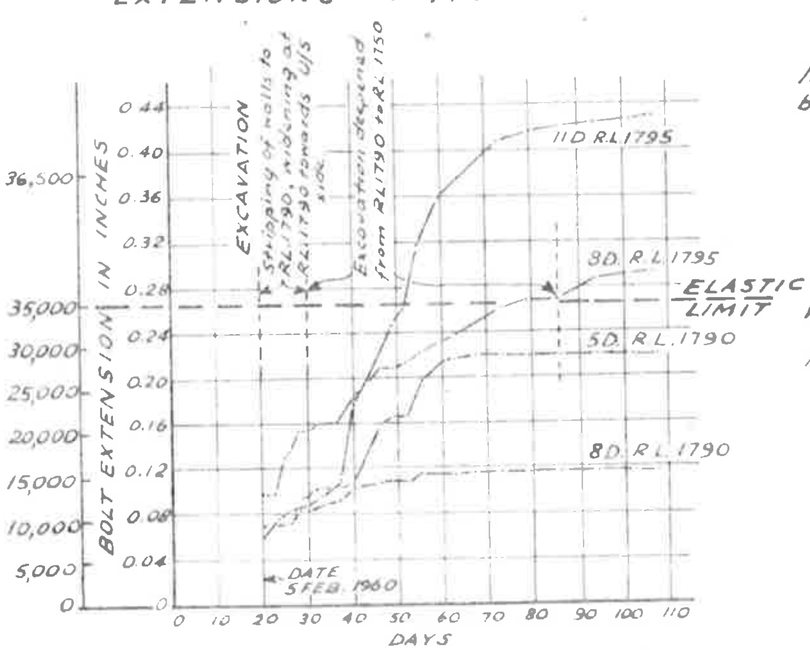


B. PROGRESSIVE PARTIAL FAILURE OF JOINT BLOCKS ADJACENT TO EXCAVATION SURFACE

D. MEASURING ROCK BOLT.



C. ROCK MOVEMENTS IN MACHINE HALL WALLS - INDICATED BY EXTENSIONS IN 14 FT. LONG ROCK BOLTS



NOTE: Zero reading before tightening
All bolts installed at 250 ft.-lb torque

ROCK BEHAVIOUR

FIGURE 7

When a large opening is made the rock near the excavation surface would be weakened to some extent by blasting, and further weakened as it became partly unconfined. This rock would be left to carry the load previously carried by the rock removed. The magnitude and pattern of the stresses developed in this rock around the opening would depend largely upon the initial or primary state of stress, and the shape and size of the opening. The ability of the rock to withstand these stresses would depend upon its strength and mechanical properties. Complete failure by rupturing of a section of rock at the excavation surface could cause a rock-fall as described in (a) above, and additional load to be transmitted to the rock immediately behind. If a section partially failed (Figure 7, Diagram B), but remained interlocked with adjacent sections, it would continue to support some load, and less load would be transmitted to the section behind. If this section could adequately support its share of the load without failure, then equilibrium would have been reached. If not, then it would also fail, (but to a lesser extent than the section in front of it), and this progressive partial failure would continue until equilibrium was reached.

With the generally tightly jointed rocks at Tumut 2 it was felt that failure due to direct gravity effects on individual blocks could be prevented or controlled by the use of rock bolt supports. However, the extent to which elastic deformation and "progressive" failure of the type shown in (b) might occur could not be assessed until the probable magnitude and pattern of the stresses were known. At Tumut 1 Power Station, located 1200 feet underground in similar rocks to those at Tumut 2, stress concentrations during construction had caused strong compression to develop in the arch ribs in the roof, and slight rotation of the abutments, which led to some spalling of the concrete arch ribs and abutment beams. Inward movements of the high vertical walls occurred during excavation and were observed by precise survey methods

(T.A. Lang, 1957 and D.G. Moye, 1958). Stress measurements made at Tumut 1 during construction of the large chambers indicated vertical compressive stresses of up to 5,500 p.s.i., and horizontal ones of up to 3,050 p.s.i.

At Tumut 2 advantage was taken of the small exploratory tunnel of simple (rectangular) shape, and measurements of rock stress were made well before commencement of construction of the main chambers. The measurements were made by officers of the Authority's Physical Sciences Branch, using the flat jack method developed by M.E. Tincelin (1952). Their results, shown on Table 1, showed that natural compressive stresses at the site were considerably greater than could be expected from the weight of overburden alone. Alexander and Worotnicki (1959) who described the work, showed a stress concentration effect related to the surface topography to account for the high vertical stress at the site, but suggested that the high horizontal stresses were probably of tectonic origin, related to past activity in the region.

TABLE 1

Primary Compressive Stresses at Tumut 2 Power Station Site

	Computed from Flat Jack Observations	Computed on Basis of Overburden Depth
Vertical stress	1500 p.s.i.	840 p.s.i.
Horizontal stress (perpendicular to drive direction)	1800 p.s.i.	100 to 250 p.s.i.
Horizontal stress (parallel to drive direction)	1900 p.s.i.	

Photoelastic studies were carried out by Worotnicki (1958, References 24 and 25) to determine the possible stress pattern around the proposed Machine Hall, and also to assess the changes in the stress pattern which would occur during the progress of the excavation. His studies showed that zones of tension would probably occur in the upper parts of the high vertical walls during deepening of the excavation, and that considerable losses of compression would occur at lower levels. Another tension zone was indicated in the rock floor of the station on the downstream side.

Worotnicki also showed that the arch roof of the station would be strongly compressed during deepening of the excavation below roof abutment level, and that an appreciable portion of the final compressive stress in the roof would be developed during excavation of the rock in the first 20 feet below roof abutment level.

Application of Rock Behaviour Studies

The types of temporary and permanent supports to be used in the large excavations were decided upon by the designers after consideration of the probable behaviour of the rock mass during and after construction. Predictions which were made were based partly on the geological and other studies described above, and to a large extent also upon the experience obtained during the construction of Tumut 1, which is of similar dimensions and is located in generally similar rock. (T.A. Lang, 1957, and D.G. Moye, 1958, Reference 16, and 1960, Reference 26).

It was considered that rock bolts would provide adequate support for the rock during excavation in most places. These were prescribed as the main form of construction support. Provision was made for smaller amounts of conventional steel I-beam supports to be used in any areas in which rock bolts alone were inadequate. Throughout most of the works the rock bolts were to be grouted in place, and hence would form part of the permanent support.

A close pattern of grouted rock bolts was envisaged for both the temporary and permanent support of the greater part of the high vertical walls of the machine hall. In the arch roofs of the machine and transformer halls, lightly reinforced concrete ribs were proposed for permanent support, in addition to patterns of rock bolts grouted in place. The roof-ribs at Tumut 2 were designed to be nominally 2 feet thick, that is, only half the nominal thickness of those at Tumut 1. The ribs were 10 feet wide and quite independent of each other, as there were to be no reinforced concrete abutment beams of the type at Tumut 1. The thinner ribs were designed to be more flexible, and being independent of one another they were less prone to damage by differential rock movements, than interconnected ones. Should the abutment rock for an individual rib break away, or be loosened to such an extent that a suitable foundation for the rib could not be obtained at the nominal abutment level, then the abutment could be excavated deeper locally until a suitable foundation was obtained, and the rib extended downwards to this foundation. To protect the concrete ribs from damage due to compressive forces developed during deepening, it was specified that no concrete placement for the ribs could commence until the excavation had progressed to 20 feet below abutment level.

Groundwater Observations

Both the granitic and dyke rocks have a very low porosity and are virtually impervious in hand specimens, but it was appreciated that groundwater could exist in open joints and jointed zones in the rocks, and could prove to be a construction hazard in several possible ways. Large quantities of water flowing from open joints into the main excavations could make working conditions unpleasant, and could cause flooding, or else make expensive pumping facilities necessary. Water flows could wash out the soft material in minor faults, or else cause softening of joint coatings, thus reducing the bond between adjacent joint blocks.

Water in slightly open joints could exert high pressures on the rock, which in turn could result in failure of the rock near excavation surfaces, particularly near the proposed high vertical walls. Consideration had to be given to groundwater pressures and quantities in the design of concrete linings or structures cast against the rock surface, and in the design of steel lining for the pressure shafts.

Diamond drill holes put down in and near the site during the site selection stage gave early indications that most joints in the rocks were tightly closed, and that groundwater would be present only in very few slightly open joints, and possibly along and near some of the minor faults. Permeability tests on D.H. 5779, where it passed close over the roof of the adopted machine hall, showed most 20-foot long sections to be watertight. The remaining few sections showed small leakages of up to 0.8 gallons per minute (48 gallons per hour) with water pressures of 200 p.s.i. applied at the ground surface. Similar results were obtained during permeability tests on other holes drilled near the site. After the completion of drilling the groundwater level in every hole was close to the ground surface. In D.H. 5777, which was sloped at 45° downwards into the Tumut Valley wall, a small artesian flow was struck at 470 feet, that is, 526 feet vertically below the ground surface. The water inflow section was isolated with a rubber packer and its pressure was measured by means of a gauge at the hole collar. The measured pressure of 50 p.s.i. is equivalent to a water head of 116 feet above the gauge level, which suggested that the water table above this point in the hole is very close to the ground surface, as shown on Figure 4. Another hole, D.H. 5780, sloped into the steeply rising left bank of Eight Mile Creek, also encountered a small artesian flow, but the pressure was not recorded.

The exploratory tunnel confirmed the impermeable nature of the average rock, as indicated by the diamond drilling. The tunnel walls and roof were predominantly damp or dry, with water seepages from slightly open joints in some of the dykes, and from minor faults. The only measurable water flows occurred from several minor faults in Zone A, (Figure 5) which was met in both the horizontal and sloping sections of the tunnel. The total water inflow into the tunnel on completion of its excavation was 1500 gallons per hour.

Diamond drilling into the site from the exploratory tunnel gave additional information on both the permeability of the rock mass, and groundwater pressures. As each hole passed through the site, measurements were made of the rate at which water flowed from it. Steady small increases in the rates of flow occurred as drilling progressed and additional water-bearing structures were intersected. In some places sudden larger increases in the rates of flow indicated the main water-bearing joints or seams. These observations were plotted on the geological logs of the drill holes, as shown on the summarized log of D.H. 5707, on Figure 6. In this hole the flow increased slowly but steadily to 15 gallons per hour until the hole reached 157 feet, where a sudden increase to 90 gallons per hour occurred. The core from this section of the hole showed a narrow crushed seam and dolerite dyke 6 inches wide. Other sharp increases in the flow occurred near 260 feet and 303 feet, but the core did not indicate any unusual structures in these sections.

D.H. 5705 was the first hole drilled, and on completion water was flowing out at the rate of 120 gallons per hour. The hole was sealed off, firstly by concreting a short length of casing into its collar, and attaching a valve and Bourdon gauge (Method 1, Figure 4). The maximum pressure recorded was 148 p.s.i., and was reached approximately 20 days after sealing. The reason for this delay is not known, but it may have been due to the gradual expulsion of air trapped in the hole. Slight leakage occurred around the edges of the concrete seal, and the seepage rate from nearby joints in the tunnel increased noticeably. Considering the vertical cover of 600 to 800 feet, and assuming a water table close to the ground surface, it was concluded that the pressure recorded was unrealistic, probably due to leakage into the tunnel through a zone of slightly open joints and minor faults (Zone A, Figure 5). To avoid this zone, which intersected the first 45 feet of the drill core, the hole was sealed by means of a rubber packer placed 70 feet from the collar, in a section where the core indicated sound rock (Method 2, Figure 4). A few hours after sealing by this method, the pressure had reached 268 p.s.i. and after 13 days a steady pressure of 305 p.s.i. was reached, indicating a static water head of 700 feet above the gauge level. The four other holes drilled through the site were also sealed off by this second method, and their maximum pressures ranged from 205 to 260 p.s.i., all less than that of the first hole. Tests carried out by opening the holes one by one and recording the changes in pressure in other holes indicated that there were groundwater connections of varying degrees between all the holes, and this being so, it was logical to assume that they also existed between the tunnel and the drill holes. It was therefore concluded that the highest pressure recorded, 305 p.s.i. in D.H. 5705, would approach the initial groundwater pressure, i.e. that which existed before the area was disturbed by the exploration.

An indication of the pressure gradient in the tightly jointed rocks was given by two holes, 10 feet and 20 feet long respectively, which were drilled into the wall of a drilling chamber in the exploratory tunnel. The hole 10 feet long passed through rock of very low permeability and produced a water flow of only 1 gallon per hour, which when sealed off with a packer gave a maximum pressure of 167 p.s.i. The 20 foot long hole gave a higher water flow of 55 gallons per hour, and a maximum pressure of 145 p.s.i. when sealed off 6 feet from its collar.

Because such high groundwater pressures and steep pressure gradients could exist in the tightly jointed rocks, provisions for groundwater drainage were an important feature of the Tumut 2 design. No pressure grouting of the rock to control or prevent water inflows was provided for, and it was proposed that pressure grouting for rock consolidation purposes should be kept to a minimum. The concrete roof-ribs for the machine and transformer halls were cast against the rock, but separated from each other by drainage slots 2 feet wide exposing the rock. From the slots a pattern of drainage holes was to be drilled 20 feet into the rock over the roof, to provide further relief to groundwater pressures. The vertical rock walls of the chambers were treated in a similar manner, by means of a pattern of holes spaced 20 feet apart vertically and horizontally, drilled 20 feet into the rock at angles of 5^{o} to 10^{o} downwards from the horizontal. All drainage holes were to be drilled as soon as possible after exposure of the rock, but after grouting in place of rock bolt supports. The steel lining for the pressure shafts was designed to withstand external water pressure based on a groundwater column extending to the ground surface above the line of the shafts.

Concrete Aggregates

The location of suitable materials for both coarse and fine aggregates for the Tumut 2 Project presented a difficult problem for several reasons. Because of the youthful nature of the gorges of the Tumut River and its tributary creeks, deposits of natural sand and gravel occur rarely along them, and very few of the deposits which do exist are sufficiently large to be considered as a materials source for the project. Several deposits of gravel along the Tumut River 4 to 5 miles downstream from the power station site were explored and found to contain excessively large quantities of material which had become softened by chemical weathering in the period since deposition. Petrographic examination of the main granitic and dyke rocks indicated that with the exception of material from the deformed marginal zone these would probably produce suitable coarse aggregates when crushed. Suitable surface sites for quarrying the desired amounts of fresh rock were rare, however, due to the extensively weathered nature of most of the slopes and plateau country. A few areas of steep cliffs adjacent to the Tumut River presented the only possible surface sites, and with these were the problems of difficult and expensive access, and lack of working space. In addition, there was the likelihood that quarrying operations would cause the extensively weathered slopes above to become unstable.

Another problem associated with construction of the project was the location of suitable areas for the disposal of rock spoil from the excavations. Because of the steep sided, narrow nature of the gorges, there were very few suitable areas near the power station site, and each of these was of limited capacity. The use of rock spoil as a source of concrete aggregate was therefore seen as a useful solution to the aggregate problem. Detailed exploration of the power station site indicated that the rock spoil obtained from this area would probably be suitable for crushing for coarse aggregate. Trial concrete mixes and standard aggregate tests carried out by the Authority's Materials Branch proved the suitability of the material, which was adopted as the main coarse aggregate source for the project. The tests also showed that fine aggregate produced from the granitic rocks at the site was suitable for limited use, blended with the natural sand proposed as fine aggregate.

After considerable exploration by means of percussion drilling and trenching by bulldozers, extensive deposits of sand of early Tertiary age were found beneath basaltic soil and talus at the southern end of the Eight Mile Plateau. The geological work was carried out and described by K.R. Sharp (1957). These deposits were adopted as the main source of fine aggregate.

CONSTRUCTION

Construction of the power station has been carried out by contract, under the supervision of the Snowy Mountains Authority.

Scope and Objectives of Geological Work

The chief object of the geological work during construction was to provide assistance with day to day construction problems, most of which have some connection with the geology, especially during the excavation stages. To provide this service it was necessary to have at all times an up-to-date picture of the geology as exposed in the excavations, and to be sufficiently familiar with both the designs and the construction techniques to fully understand the problems which arose. To obtain the geological picture and also the essential familiarity with the job, detailed geological logs or maps were made of the excavations as they progressed, and the geology as exposed was reviewed and correlated with the geological data obtained earlier during exploration of the site.

Another aim of the geological work was to check that the geological conditions actually encountered corresponded with those assumed in the designs, so that if any significant discrepancy was apparent the works could still be constructed to suit the rock as exposed. A scale model used by the designers for this purpose consisted of a perspex outer shell which displayed the interpreted geological picture shown on Plate 2. When the proposed excavation procedure was known in detail, wooden blocks were made which represented the sections of rock to be removed at the successive stages of construction. As each detailed geological log was received from the site, the main geological features were plotted onto the appropriate wooden block, which was inserted into its place in the model. This gave a direct comparison between the predicted and actual geology.

Together with photographs the geological logs provided a permanent record of rock which was later covered by steel or concrete linings, or by concrete walls. These records were to be used if any difficulty of a geological nature arose at a later stage of construction, or during the operation of the station.

During the geological logging it was attempted to relate the geological detail in individual sections of the rock mass to their behaviour during and after excavation, in particular, to their support requirements. The results of the work were recorded whenever possible in the form of geological reports. These reports were of two types. Short reports were prepared for action on the day-to-day problems, and longer, comprehensive reports were prepared as records of the construction. As well as their immediate use during construction, it was intended that these reports and logs be used in the future by personnel engaged in the planning or construction of underground works.

Geological Logging

The geological logging was mostly done under conditions which were, to say the least, most unsuitable for scientific work. Except when use could be made of "off-shift" periods, logging was carried out amid the incessant noise and activity of high-speed mining operations. There were numerous interruptions. The logging method adopted was therefore designed to make the geologist's field work straight-forward and rapid. This was achieved by means of careful office and field preparatory work, carried out mainly by assistants.

The Authority's standard tunnel-log form was used almost exclusively as a basis for the logs. The form is essentially a sheet of tracing linen of standard size, printed with title block and legend.

Also printed on the sheet are numerous headings and blank columns. The headings cover the complete range of tunnelling and geological data to be recorded. Figures 9 to 12 were each produced on such a form, with some adaptions. They have been reduced photographically for inclusion in this thesis. The original scale of all of the Tumut 2 logs was 1;240, or 20 feet to 1 inch.

Using the appropriate design drawings for dimensions, the draftsman or assistant produced on the standard log sheet a suitable outline drawing or projection of the excavation surface or surfaces to be logged. In the case of Figure 9 the tunnel was depicted as three parallel strips, as shown on the explanatory diagram at the extreme left. Figure 10 was a plan view, and Figures 11 and 12 were elevations.

The field preparatory work consisted of marking up the excavation surfaces with vertical lines or marks at regular chainages, usually every 10 feet. In high or sloping excavations a series of lines of known level was also employed. Brightly coloured crayons or paint were used for the marks. The positions of these lines and marks were plotted in coloured pencil on a dyeline print of the previously prepared log sheet. This print was termed a field sheet, and was clipped onto a rigid board for use in the field.

Having the field sheet and the excavation marked up with equivalent lines, almost forming a "grid", the geologist was able to plot the traces of the geological features largely by eye. A metallic tape and ranging pole were employed where measurement was necessary in the areas between marked lines. A Brunton transit was used for dip measurements, but was unsuitable for strike measurements because of the presence of construction machinery, and iron drainage and ventilation pipes. To measure strikes the field sheet was held horizontal and oriented so that the line of section, plan, or elevation, on the drawing was parallel to the equivalent line in the actual excavation. A drawing scale held flat on the field sheet was pointed along the observed strike direction, and the strike line plotted directly onto the sheet. Because of the slightly curved, irregular nature of many joints, it was often necessary to view them broadly, rather than taking the strike along small joint faces.

It was appreciated that a graphic log made from one viewpoint, and showing only the rock types and traces of geological structures, would be of very limited value. Graphic logs produced in plan view, or as three parallel strips as on Figure 9, were therefore accompanied by cross sections and logs of working faces. Those produced as elevations were accompanied by plans at certain levels. Every graphic log had an adjacent strip plan showing the true strikes and dips of geological structures.

This graphic picture of the excavation surface or surfaces was supplemented by notes on the geology, type of rock breakage, and ground water. On all logs, and throughout all stages of the geological work, the weathering products of granitic rocks were classified according to Table 2, which was devised by Moye (1955). Also on the logs were records of excavation progress, support during construction, photographs, and the final support, lining, or treatment. On logs of tunnels (Figures 9 and 10), individual sections were classified according to a qualitative table of "Rock Condition" (Table 3). This table was devised by Moye (1958). It is based partly on work of Talobre (1957), and Terzahgi (1946), and partly on experience with the Authority's tunnels.

Geology of the Completed Excavations

The completed geological logs have been used to compile Figure 8. The plan and sections on this drawing demonstrate that the final geological picture is substantially the same as the interpreted one, shown on Plate 2 and Figure 5.

TABLE 2

Weathering Products of Granitic Rocks

Name	Definition	Engineering Properties
Completely Weathered Granite:	Granite completely decomposed by weathering in place, but still possessing a recognizable granitic fabric; the original felspars are completely decomposed to clay minerals which remain as grains of clay; biotite mica may be decomposed to varying degrees; it will disintegrate into a mass of sandy clay when immersed in water and cannot often be recovered as cores by ordinary diamond drilling methods; often stained brown with limonite.	Can be excavated by hand, and by ripping, without the use of explosives. Unsuitable as foundations for concrete dams or large concrete structures; may be suitable for foundations of earth dams (permeability of 5 to 100 feet per year); unstable in high cuttings at angles steeper than 1 : 1; may be suitable for rolled earth fill for earth dam construction (permeability 0.1 to 5 feet per year); readily eroded by water and frost and requires protection against erosion.
Highly Weathered Granite:	Intensely weathered granite, weakened to the extent that pieces the size of NX drill core ($2\frac{1}{8}$ inches in diameter) can be broken and crumbled in the hands; does not disintegrate when soaked in water and can often be recovered as cores by careful diamond drilling, but is often lost, often stained reddish brown due to limonite.	Very similar to those of completely weathered granite.
Moderately Weathered Granite:	Granite considerably weathered throughout, but possessing strength such that pieces the size of NX drill core cannot be broken by the unaided hands; often stained reddish brown with limonite.	Can be excavated with difficulty without the use of explosives; may be suitable for foundations of small concrete structures; its stability in cuttings depends largely on jointing, the rock itself being capable of standing vertically; mostly crumbles under 'dozer tracks; may be suitable for semi-pervious fill for dam construction; although the rock itself is practically impervious, the rock mass is often highly permeable due to the presence of open joints.
Slightly Weathered Granite:	Granite distinctly weathered throughout the fabric of the rock, as shown by slight limonite staining, and some decomposition of the felspars; but its strength approaches that of fresh granite.	Requires use of explosives for excavation; often suitable for foundations for concrete dams; practically impervious but the rock mass is often highly permeable due to the presence of open joints; unsuitable as concrete aggregates.
Fresh Granite:	Granite, not discoloured in any way with limonite, and of normal hardness and strength. Fresh granite lying immediately beneath the various types of weathered granite frequently shows limonite stains along joints; this is described as "fresh granite with limonite stained joints".	Requires use of explosives for excavation; suitable for foundations for concrete structures; practically impervious but the rock mass is often highly permeable due to the presence of open joints; may be suitable as concrete aggregate.

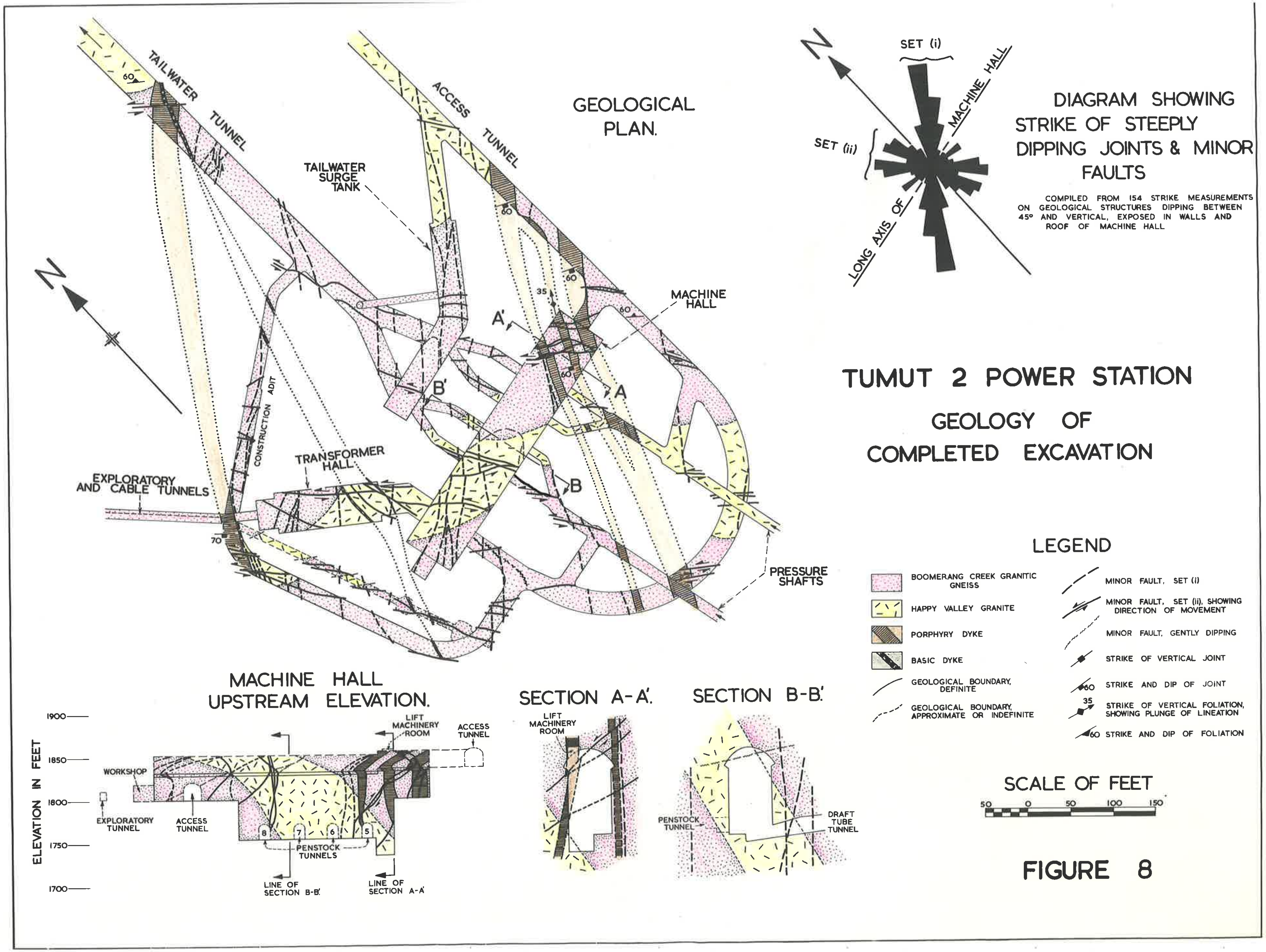
TABLE 3

Rock Condition Classification

Classification	Characteristics
5 Good	Rock mass is perfectly sound, and very compact.
5S	Rock mass is perfectly sound, and very compact. Spalling or popping occurs.
4 Moderately good	Rock mass is sound, but somewhat loosened along joints. Support usually not required.
3 Mediocre	Blocky and seamy, considerably loosened along joints. Considerable support may be required.
2 Poor	Very blocky and seamy, much loosened along joints, requiring continuous support.
1 Bad	Exerts considerable pressure on the supports.
0 Very bad	Exerts considerable pressure on the sides as well as the top of the supports. Includes swelling and squeezing ground. Very difficult tunnelling, special methods often being necessary.

Granitic Rocks

The excavations are located mainly in Boomerang Creek granitic gneiss which is intruded by two steeply dipping sheet-like masses of Happy Valley Granite. Both rocks are grey in colour and usually medium grained, but except in their gradational contact zones are readily distinguished from one another at the site. The granite tends to be coarser grained and lighter grey in colour than the gneiss. Macroscopic examination of the granite reveals distinct individual grains of grey,



TUMUT 2 POWER STATION
GEOLOGY OF COMPLETED EXCAVATION
GEOLOGICAL PLAN.
TAILWATER TUNNEL
ACCESS TUNNEL
TAILWATER SURGE TANK
MACHINE HALL
CONSTRUCTION ADIT
TRANSFORMER HALL
EXPLORATORY AND CABLE TUNNELS
PRESSURE SHAFTS
A
A'
B
B'
N
DIAGRAM SHOWING STRIKE OF STEEPLY DIPPING JOINTS & MINOR FAULTS
COMPILED FROM 154 STRIKE MEASUREMENTS ON GEOLOGICAL STRUCTURES DIPPING BETWEEN 45° AND VERTICAL, EXPOSED IN WALLS AND ROOF OF MACHINE HALL
SET (i)
SET (ii)
LONG AXIS OF MACHINE HALL
LEGEND
BOOMERANG CREEK GRANITIC GNEISS
HAPPY VALLEY GRANITE
PORPHYRY DYKE
BASIC DYKE
GEOLOGICAL BOUNDARY, DEFINITE
GEOLOGICAL BOUNDARY, APPROXIMATE OR INDEFINITE
MINOR FAULT, SET (i)
MINOR FAULT, SET (ii), SHOWING DIRECTION OF MOVEMENT
MINOR FAULT, GENTLY DIPPING
STRIKE OF VERTICAL JOINT
STRIKE AND DIP OF JOINT
STRIKE OF VERTICAL FOLIATION, SHOWING PLUNGE OF LINEATION
STRIKE AND DIP OF FOLIATION
SCALE OF FEET
50 0 50 100 150
MACHINE HALL UPSTREAM ELEVATION.
ELEVATION IN FEET
1900
1850
1800
1750
1700
WORKSHOP
EXPLORATORY TUNNEL
ACCESS TUNNEL
LIFT MACHINERY ROOM
PENSTOCK TUNNELS
8 7 6 5
LINE OF SECTION B-B'
LINE OF SECTION A-A'
SECTION A-A'.
SECTION B-B'.
PENSTOCK TUNNEL
DRAFT TUBE TUNNEL

FIGURE 8

glassy quartz, and white or greenish felspar, and flakes of black biotite. A few small muscovite flakes are usually also visible. In the gneiss, which is usually finer grained and darker in colour, muscovite and altered cordierite are present in appreciable amounts, in addition to quartz, feldspar, and biotite. Lack of contrast between the individual grains and the presence of dark grey-green clusters of biotite and altered cordierite, result in a rock of dull and often blotchy appearance. Petrographic descriptions of both the granite and gneiss are given in Appendix I.

The narrower granite mass ranges from 70 to 100 feet in width and passes through most of the Transformer Hall, and through the deep central portion of the Machine Hall. The other mass which is several hundred feet in width is exposed in the Tailwater and Access tunnels on the north-eastern edge of the site. Both range in strike from N.110°E to N.140°E, and dip 50° to 70°NE.

Primary Structures in the Granitic Rocks

Contacts between the granite and gneiss are in most places very poorly defined, and are often gradational over distances of up to 30 feet. The sharpest contact is in the Tailwater tunnel downstream (north) from the site, where pronounced schlieren and numerous xenoliths occur in the granite, as shown on Plate 5. Most of the xenoliths are lying with their long axes parallel to the schlieren direction, but a few make angles of up to 45° to it. The schlieren pattern around both the parallel and non-oriented xenoliths is strongly suggestive of one produced by viscous flow. Only 8 to 10 feet away from this contact the granite appears quite massive, but the few xenoliths which occur in it mostly have their long axes parallel to the contact.

The granitic gneiss is in several places banded or layered, apparently due to alternate layers which differ slightly from one another in mineral composition. The banded appearance, and the mineralogical composition (Appendix I) suggest that the gneiss is a metamorphic rock of sedimentary origin. The layering in the gneiss is parallel in most places to the granite - gneiss contacts, and hence also parallel to any schlieren or foliation in the granite. Xenoliths occur also in the gneiss, and are oriented parallel to the plane of layering. Because of the indefinite nature of many of the contacts, and the close structural relationship between the two rocks, it is considered that the gneiss and granite may be joint products of metamorphism and partial mobilization of older rocks, probably of sedimentary origin.

Porphyry Dykes

Three nearly vertical porphyry dykes pass through the eastern or Control Building end of the Machine Hall. Another which passes through the north-western edge of the site is exposed in the Tailwater and Cable tunnels, and in the Tailwater tunnel construction adit. In the average dykes which are 10 to 20 feet in width the porphyry is a brittle, compact rock, dark grey-brown in colour, and micro-crystalline, containing varying amounts of lath-shaped feldspar phenocrysts which range from 1 to 5 mm. in the longest dimension. Small clusters of ferromagnesian minerals are usually present, and appear as dark green to black specks or streaks. In dykes 10 feet and less in width the porphyries appear darker and finer grained, and phenocrysts are rare. Both the wide and narrow dykes have chilled margins as much as several inches in width, in which the porphyry is dark grey to black and in parts is glassy in appearance.

The porphyries appear to have been intruded along nearly vertical fissures parallel to or belonging to the Set (i) minor faults. Figures 3 and 4 show that their boundaries are very straight and regular when viewed over some distance, but several detailed exposures in the power station excavations show irregularly shaped contacts. Near the junction of the Cable tunnel with the Transformer Hall an irregular 1 to 2-inch wide vein or "stringer" of porphyry passes from a 10 to 15 feet wide porphyry dyke into the adjacent gneiss. Plate 6 shows a porphyry-granite contact which is straight when viewed broadly but irregular in detail. This dyke, which is only 5 feet wide, contains few phenocrysts and is very dark grey and brittle. In several places at the site the original intrusive contacts have been obscured due to the later development of flaky sheared zones or coated joints along them.

Figure 8 shows that the dykes are in part composite in nature, there in fact being only two dykes exposed in the excavations adjacent to the machine hall. The third or central dyke seen in the machine hall branches off from one of the two dykes exposed 100 feet away in the Access tunnel. The dyke exposed in the penstock bifurcate is clearly a multiple one, and is partly separated into two dykes by a 1 foot thick lens-shaped mass of granite. The correlation shown between dykes is based directly on exposures near the machine hall roof level, and only indirectly on the penstock exposures, as these are 60 to 80 feet lower.

Primary Structures in Porphyry Dykes

A planar gneissic structure parallel to the contacts is displayed to some extent by all of the porphyry dykes. It is most marked close to their contacts, and is shown by the planar arrangement of phenocrysts and clusters of ferromagnesian minerals. A linear arrangement of the phenocrysts and clusters in the foliation plane has also been observed. Eleven field measurements made in the dykes at the site have all shown lineations plunging between 20° and 40°N. It is considered that both the planar foliation and the lineation are primary flow features, although later shearing has in some places caused secondary structures to be superimposed on them.

The lineation is approximately normal to the most prominent set of joints in the dykes. These joints strike almost at right angles to the dyke contacts, and range in dip from 50° to 70°S. They have rough surfaces and are commonly very slightly open, and water seepages from them cause most of the dykes to be damp or wet in the underground exposures. They appear to have been formed due to stretching at the time of emplacement of the dykes, and are therefore considered to be equivalent to cross joints, as defined by Balk (1958).

The individual joints of this set terminate at or near the dyke contacts, and are entirely different in character than any joints in the granitic rocks, as shown by Plates 6, 7 and 8. Many joints and minor faults of set (ii) (Figure 8) pass through both the granitic and dyke rocks, but these are of the slickensided type shown on Plate 7, and hence are easily distinguished from the cross joints.

Dolerite Dykes

Two dolerite dykes which range from 0.25 up to 18 inches in width pass through the north-western part of the site. The dykes are dark-grey to black, and fine grained. They appear to have been intruded along joints of set (i) although in several places they follow set (ii) joints for as much as 12 inches. Tightly welded contacts are characteristic of the dykes but in several places sheared or crushed contacts have also been observed. The two dolerite dykes appear to join and form one wider dyke, which was intersected by the Tailwater tunnel downstream from the power station. In the Tailwater tunnel exposure, this dolerite dyke cuts intrusively through one of the porphyry dykes.

Geological Structures

Figure 8 shows that the site is intersected by two sets of minor faults. In the first set most of the structures are vertical or almost so, and strike between N.20°E and N.60°E. The second set includes structures dipping between 45°S.E. and vertical, and ranging in strike from N. 120°E to N. 170°E. The rocks are also extensively jointed. The majority of joints are steeply dipping and are parallel or nearly so to the two main sets of minor faults. Comparison of the geological structure diagram on Figure 8 with the one on Figure 5 shows that these two main sets of minor faults and joints correspond closely with sets (i) and (ii) indicated by the design stage work. The individual minor faults met in the excavations included all except one of those predicted from the exploration, and in addition several others which were intersected by the diamond drilling, but could not be reliably oriented or correlated at that time. There were no faults met that had not been intersected and recognised during the diamond drilling.

Gently dipping joints were quite rare at the site and were estimated to make up less than 5 percent of the total number of joints present. The gently dipping minor fault 'B' indicated on Figure 5 was predicted to pass through the roofs of the Machine Hall and connecting tunnel to the Transformer Hall. Several gently dipping joints were met in these areas, but no well defined sheared zone. The only gently dipping minor fault met in the excavations was parallel to this predicted one, but 100 feet further to the east. It was dipping between 20° and 30° S.W. and passed through Machine Hall roof over the deep central portion, as shown on Figure 8. In most places it consisted of a crushed seam less than 0.5 inch wide, and was accompanied by several gently dipping tightly closed joints.

Set (i) Structures

Every minor fault of this set was a sheared zone in which the rock was compact and firm, but flaky due to closely spaced slickensided joints, slightly curved and intersecting as shown and described on Plate 11. It appears that appreciable relative movements have occurred between adjacent fault blocks, but that in most cases the strain has occurred as numerous very small movements along slickensided joints. In a few cases crushed seams less than 0.5 inches thick were present. The zones were commonly damp or wet, due to seepages from slightly open joints, or from crushed seams. The zone shown on Plate 11 is the widest at the site, and approaches 4 feet in width. Most of the others range between 1 and 12 inches, predominantly 6 inches. Several of these narrower zones passed through the western end of the Machine Hall, and were exposed also in the Tailwater Surge Tank and Draft Tube tunnels. Another very firm but flaky zone of this set passed close to and partly along a porphyry dyke contact at the eastern end of the Machine Hall. On the downstream side this zone was 3 feet wide, but towards the upstream side it divided into two zones less than 6 inches wide, and several slickensided joints. The Transformer Hall was also intersected by several sheared zones of this set.

Most joints of set (i) were tightly closed, and water seepages from them were rare. The joint surfaces were somewhat irregular, and invariably slickensided, as shown on Figure 7. Chlorite was the most common joint filling, and occurred in veneers generally less than 0.05 inches thick. Thicker coatings, including both chlorite and calcite, with patchy films of pyrites, were present on some of these joints. When viewed for several feet most of the joints were seen to be slightly curved, and it was common in the more closely jointed areas (joint spacings less than 6 inches) for joints to persist only 10 to 20 feet before ending at their intersections with other joints of the same set. In more broadly jointed areas (joint spacings of the order of 1 to 2 feet), most of the joints were seen to persist for distances of 40 to 100 feet.

On both the minor faults and joints of set (i), slickensides were mostly either horizontal or plunging at shallow angles (less than 40°). Only two cases of actual displacement of adjacent rock masses along joints or faults of this set were seen in the excavations. Plate 9 shows one of these known displacements, indicated by a displaced xenolith in granite. The other example is in the Tailwater tunnel 500 feet downstream from the station, where one minor fault of this set displaces another fault of the same set through a horizontal distance of 3 feet. In both cases the movement has occurred in the same sense; the north-west side has moved southwards in relation to the south-east side. Although no other actual displacements have been found, it is considered that the minor faults are transcurrent ones, and that small movements, mainly transcurrent in nature, have occurred along the set (i) joints.

Set (ii) Structures

Minor faults of set (ii) were narrow crushed seams, which consisted of soft, silty and clayey material, containing very few rock fragments, mostly less than 0.1 inch (Plate 12). Such crushed seams appear to have been formed when movement of adjacent joint blocks occurred along a single rupture plane, rather than along many closely spaced fractures, as in the case of the set (i) sheared zones. The seams often contain partly crushed calcite, which appears to have at one stage been present as a coating or filling on the fracture planes. The set (ii) faults were the main source of water inflow at the site, and very small flows or seepages occurred from most of them.

One group of these faults passes through the Machine Hall almost directly over Penstock Tunnel No. 8, as shown on the plan and upstream elevation on Figure 8. This group corresponds with Zone D predicted from the exploration (Figure 5 and Plate 2). One of this group of seams branches away towards the west and was met in the south-eastern corner of the Transformer Hall. Another very narrow seam (less than 0.5 inches thick) passes through the Machine Hall further to the west, and continues into the Transformer Hall. Two other seams, both less than 0.1 inch thick, were also met in the Transformer Hall.

Another group of these faults passed through the eastern end of the Machine Hall. In the gneiss they consisted of crushed seams ranging in width from 0.1 inch up to 2 inches, and were in some places accompanied by several inches of partly altered gneiss. In the porphyry they formed zones of up to 12 inches wide, consisting of very closely jointed (0.1 to 0.5 inches spacing) and partly crushed or brecciated porphyry. Very little soft silty or clayey material was present. In most places these zones were seeping water, and were limonite stained. The partly brecciated material was firm and compact but broke readily into small fragments when struck with a hammer or pick. It was the presence of these wider brecciated zones in the porphyry dykes which led to some difficulty during correlation between diamond drill holes through the site. They were recognised as faults in D.H. 5704, but in D.H. 5705 several of the very narrow crushed seams in the gneiss were partly or completely washed away during drilling. Those which were recorded in D.H. 5705 at that time appeared insignificant and were not correlated with the wider zones seen in the porphyry dykes met in D.H. 5704.

Set (ii) joints are fewer in number at the site than those of set (i). They are spaced usually between 6 and 12 inches apart where most abundant, and are entirely absent from the rock in a few areas. They are mostly curved and irregular, and persist for distances of 20 up to 100 feet. The joint surfaces are usually slickensided, but in addition have an irregular, undulating appearance. Calcite, which is the most common coating on them, is in many places partly flaky, or else soft and crumbling, apparently due to crushing since deposition. The coatings range from traces up to 0.4 inch in thickness, and commonly contain pyrites, either as thin patchy films or as small crystals partly filling small cavities in the calcite.

Most of the joints appear tightly closed, but very small water passages must exist along or through the coatings, as they were a common source of seepage in the excavations. From examination of the adjacent irregular surfaces of many set (ii) joints it is hard to visualise that the initial fractures could have been formed by sliding of blocks of rock over one another. It appears that the initial fracture surfaces were irregular and hackly, and have been partly planed off during later sliding movements along the joints. It is therefore considered that they were originally tension fractures. Obviously the partly planed irregular joint surfaces would not fit tightly together after any small amount of sliding had occurred. The accumulation of calcite has in most cases served to fill completely the narrow, lenticular cavities which resulted.

The presence of partly crushed calcite in many of the minor faults of set (ii) suggests that these have also been formed from old tension fractures. This concept of their origin helps to explain the rather extensive chemical alteration in the rock adjacent to many of the set (ii) structures, as such alteration would probably be caused by mineralized waters free to circulate through the rock.

Slickensides on both the joints and minor faults of set (ii) generally plunge within 20^{o} of horizontal. In many places at the site the direction and sense of movements along set (ii) structures were observed, and showed the faults to be strike-slip or transcurrent ones. Small (up to 1 inch) transcurrent movements were shown to have occurred along some of the joints, as shown on Plate 10. Seventeen larger movements, shown on Figure 8, were displayed where the minor faults passed through dykes, or through joints or minor faults of set (i). These displacements ranged from 6 inches up to a maximum of 3 feet, and averaged 1 foot.

All seventeen examples showed nearly horizontal displacements, and sixteen of them showed that the western block had moved southwards in relation to the eastern block. It will be appreciated from Figure 8 that due to repetition these examples represented probably only six main displacements in the power station excavations. There were numerous examples of small displacements such as that shown on Plate 10, and the majority of these showed the same sense of movement as the larger ones. Within the limits of the excavations shown on Figure 8, it is estimated that the total amount of horizontal displacement in this sense exceeds 10 feet.

Age Relationships

From the foregoing discussion of rock types and geological structures, it is evident that the granitic rocks are the oldest at the site. The granite is considered to be a little younger than the gneiss, as it shows rather definite intrusive relations to it.

The porphyry dykes are younger than both the granite and gneiss, and have been intruded along structures of set (i) direction. Presumably these structures were tension fractures at the time of intrusion of the dykes.

The set (ii) structures appear to have been first formed after intrusion of the porphyry dykes, but before intrusion of the dolerites. In several places the dolerite dykes locally follow the set (ii) direction, and hence some or all of the structures in this direction may have been present at least as tension fractures, at the time of intrusion of the dolerite. Transcurrent movements along the minor faults of set (ii) appear to be the youngest feature at the site. In an exposure in the Tailwater tunnel 600 to 700 feet downstream (north) from the power station, a minor fault of this set displaces both a porphyry dyke, and a dolerite dyke which intrudes the porphyry.

No special study has been made to determine the directions and magnitudes of the tectonic forces which have caused the existing geological structure to develop. It is interesting, however, to reflect on the possible relationships between the youngest structures present and the primary stresses measured at the site (Table 1). If, as is quite possible, the dolerite dykes are of Tertiary age, then the latest transcurrent movements along the set (ii) minor faults could be of Tertiary age or younger. Such movements would require unbalanced horizontal compressive forces, and vertical stresses of intermediate magnitude. The theory that the abnormally high horizontal compressive stresses are "fossil" or residual stresses of tectonic origin therefore appears reasonable.

Ground Water Inflows During Construction

During construction the rock was mostly dry, with small water inflows and seepages coming mainly from minor faults. Several actual flows of up to 10 gallons per hour occurred from set (ii) minor faults in the initial drive along the Machine Hall roof. With later widening of the roof to full size these flows decreased to seepages. No other measurable inflow was met during the excavation of the power station. Numerous small flows met during excavation of the Access Tunnel made up most of the water pumped from the site during excavation. A considerable amount also would have been wasted drilling water. The average quantity pumped daily during the main excavation stage was of the order of 100,000 gallons, and of this it is estimated that 70,000 gallons was derived from the Access Tunnel and wastage at the site. The actual inflow from the main power station excavations was therefore probably of the order of 30,000 gallons per day, or 1250 gallons per hour.

Excavation

Throughout the construction rock bolts were installed as temporary supports, and they were mostly grouted in place, and hence served as or contributed towards the permanent support. In very few isolated areas where some doubt arose as to the adequacy of the bolts for protection during construction, supplementary steel rib supports were installed. Steel ribs were also used together with rock bolts in several of the more complicated junctions between openings, in an attempt to prevent or control overbreak in these places. Descriptions of the rock bolts used, and installation and grouting procedures are given in Appendix III.

Machine Hall Roof

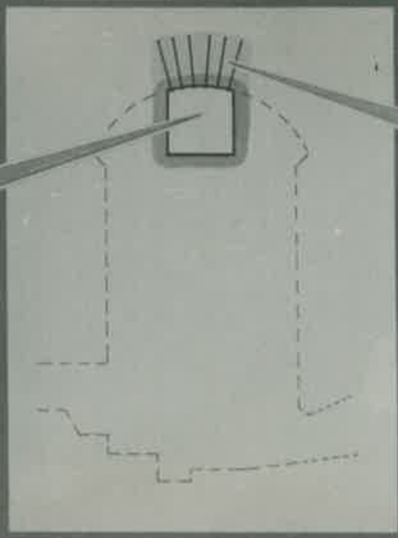
The sequence of excavation of the Machine Hall is shown by means of photographs and diagrams on Plates 13 and 14. As the first stage a "crown drive" 20 feet high and 20 feet wide was driven along crown of the proposed roof arch. As the drive advanced its roof was supported by a close pattern of rock bolts, mostly 14 feet long. These were grouted in place after completion of the drive. Supplementary steel rib supports were installed over the first 50 feet of the drive at the eastern end, where the group of set (ii) minor faults had developed zones of limonite stained breccia in the porphyry dykes.

The pattern of blast-holes used in the drive is shown on Figure 9. The main features of this pattern were the close spacing of lightly charged holes near the roof, and the loading of every alternate hole along the crown. The number and length of loaded holes in individual "rounds" or blasts, together with the quantities of An 60 explosive used, are also shown, in the column labelled "Excavation".

Along most of the drive the rock broke along or very close to the blast holes, and a good shape with very little overbreak resulted. The percentages of rock breakage along joint (or minor fault) planes, are plotted on Figure 9. Except in the slightly inferior rock in the first 20 to 30 feet at the eastern end, these figures appeared to be related mainly to the orientation of the joints or minor faults in relation to the excavation surfaces. In the walls about 50 percent of

1 Excavation of centre drive

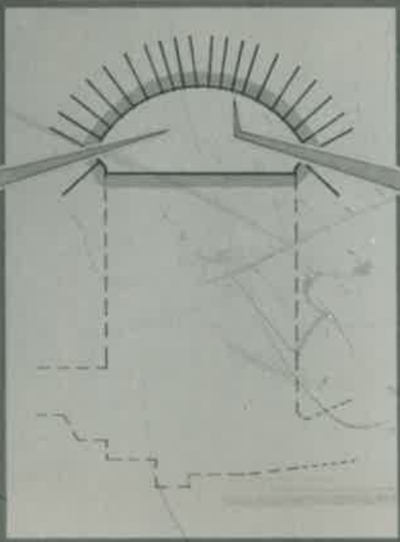
March to April 1959



2 Installation and grouting of rock bolts

3 Completion of excavation of roof

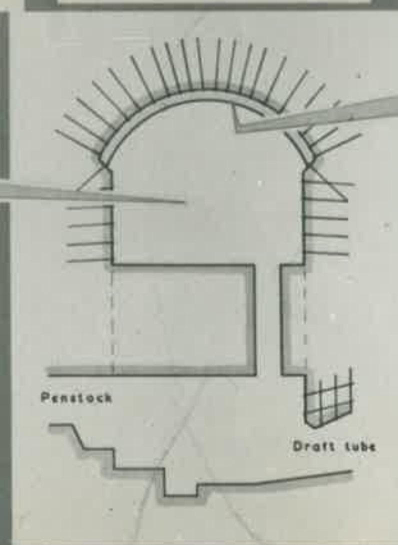
May to June 1959



4 Placing of wire mesh on roof

5 Deepening of excavation to 51 feet from roof and excavation of penstock and draft tube tunnels

June to September 1959



6 Placing of concrete arch ribs in roof

PLATE 13

TUMUT.2 EXCAVATION OF MACHINE HALL

MARCH 1959 TO MARCH 1960

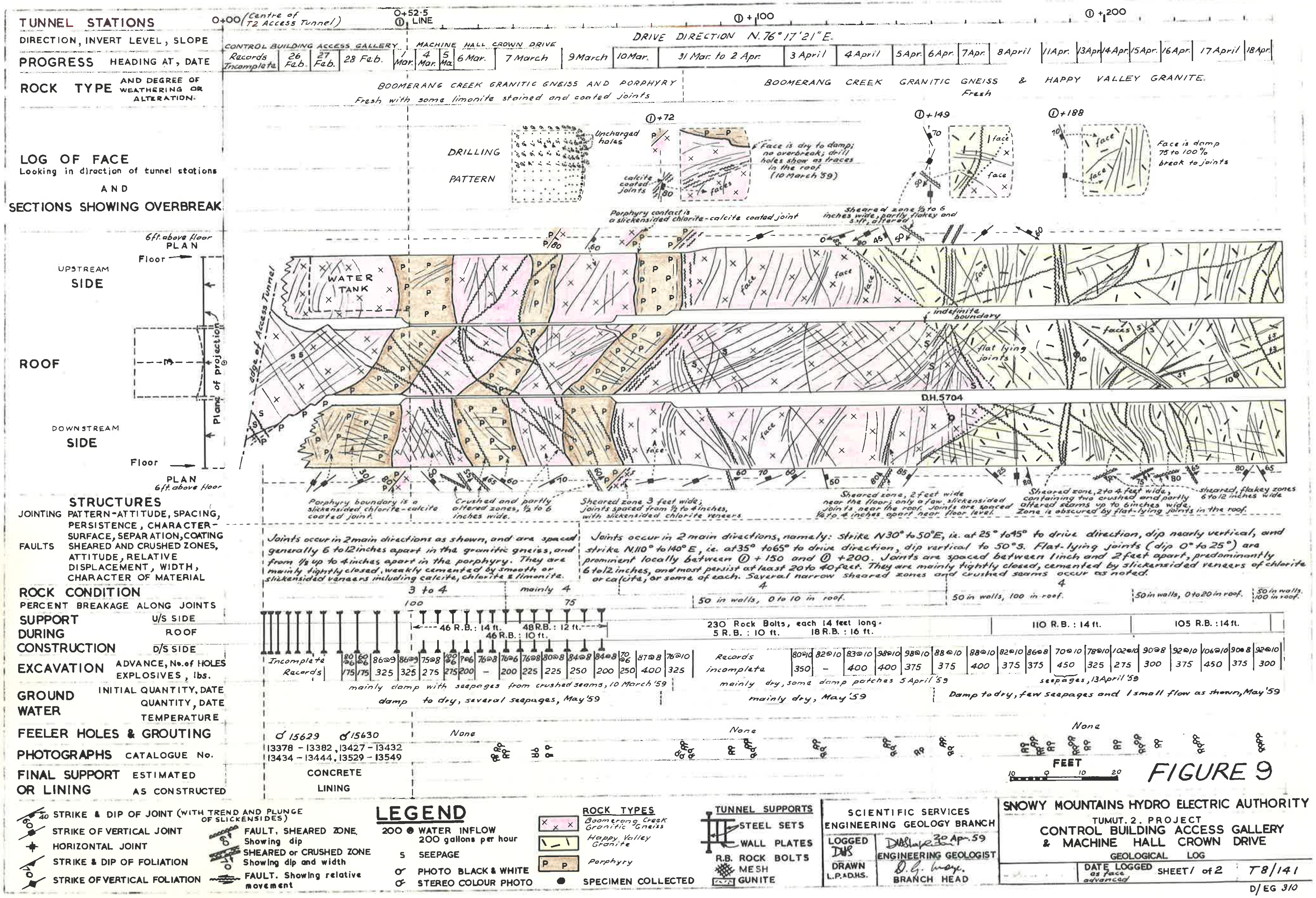

TUNNEL STATIONS
0+00 (Centre of T2 Access Tunnel)
0+52·5 ① LINE
① + 100
① + 200
DIRECTION, INVERT LEVEL, SLOPE
DRIVE DIRECTION N. 76° 17' 21" E.
PROGRESS HEADING AT, DATE
CONTROL BUILDING ACCESS GALLERY
MACHINE HALL CROWN DRIVE
Records Incomplete
26 Feb.
27 Feb.
28 Feb.
1 Mar.
4 Mar.
5 Mar.
6 Mar.
7 March
9 March
10 Mar.
31 Mar. to 2 Apr.
3 April
4 April
5 Apr.
6 Apr.
7 Apr.
8 April
11 Apr.
13 Apr.
14 Apr.
15 Apr.
16 Apr.
17 April
18 Apr.
ROCK TYPE AND DEGREE OF WEATHERING OR ALTERATION.
BOOMERANG CREEK GRANITIC GNEISS AND PORPHYRY
Fresh with some limonite stained and coated joints
BOOMERANG CREEK GRANITIC GNEISS & HAPPY VALLEY GRANITE.
Fresh
LOG OF FACE
Looking in direction of tunnel stations
AND
SECTIONS SHOWING OVERBREAK
DRILLING PATTERN
Uncharged holes
① + 72
calcite coated joints
Face is dry to damp; no overbreak; drill holes show as traces in the roof (10 March '59)
① + 149
① + 188
Face is damp 75 to 100 % break to joints
Porphyry contact is a slickensided chlorite-calcite coated joint
Sheared zone ½ to 6 inches wide, partly flakey and stiff, altered
6ft. above floor
PLAN
Floor
UPSTREAM SIDE
WATER TANK
edge of Access Tunnel
Plane of projection
ROOF
indefinite boundary
flat lying joints
D.H. 5704
DOWNSTREAM SIDE
Floor
PLAN
6ft. above floor
STRUCTURES
JOINTING PATTERN-ATTITUDE, SPACING, PERSISTENCE, CHARACTER-SURFACE, SEPARATION, COATING
FAULTS SHEARED AND CRUSHED ZONES, ATTITUDE, RELATIVE DISPLACEMENT, WIDTH, CHARACTER OF MATERIAL
Porphyry boundary is a slickensided chlorite-calcite coated joint.
Crushed and partly altered zones, ½ to 6 inches wide.
Sheared zone 3 feet wide; Joints spaced from ½ to 4 inches, with slickensided chlorite veneers
Sheared zone, 2 feet wide near the floor; only a few slickensided joints near the roof. Joints are spaced ¼ to 4 inches apart near floor level.
Sheared zone, 2 to 4 feet wide, containing two crushed and partly altered seams up to 6 inches wide. Zone is obscured by flat-lying joints in the roof.
Sheared, flakey zones 6 to 12 inches wide
Joints occur in 2 main directions as shown, and are spaced generally 6 to 12 inches apart in the granitic gneiss, and from ½ up to 4 inches apart in the porphyry. They are mainly tightly closed, weakly cemented by smooth or slickensided veneers including calcite, chlorite & limonite.
Joints occur in 2 main directions, namely: strike N30° to 50°E, i.e. at 25° to 45° to drive direction, dip nearly vertical, and strike N110° to 140°E, i.e. at 35° to 65° to drive direction, dip vertical to 50°S. Flat-lying joints (dip 0° to 25°) are prominent locally between ① + 150 and ① + 200. Joints are spaced between 1 inch and 2 feet apart, predominantly 6 to 12 inches, and most persist at least 20 to 40 feet. They are mainly tightly closed, cemented by slickensided veneers of chlorite or calcite, or some of each. Several narrow sheared zones and crushed seams occur as noted.
ROCK CONDITION
PERCENT BREAKAGE ALONG JOINTS
3 to 4
100
mainly 4
75
4
50 in walls, 0 to 10 in roof.
4
50 in walls, 100 in roof.
50 in walls, 0 to 20 in roof.
50 in walls, 100 in roof.
SUPPORT DURING CONSTRUCTION U/S SIDE ROOF D/S SIDE
46 R.B. : 14 ft.
48 R.B. : 12 ft.
46 R.B. : 10 ft.
230 Rock Bolts, each 14 feet long.
5 R.B. : 10 ft.
18 R.B. : 16 ft.
110 R.B. : 14 ft.
105 R.B. : 14 ft.
EXCAVATION ADVANCE, No. of HOLES EXPLOSIVES, lbs.
Incomplete Records
Records incomplete
GROUND WATER INITIAL QUANTITY, DATE QUANTITY, DATE TEMPERATURE
mainly damp with seepages from crushed seams, 10 March '59
damp to dry, several seepages, May '59
mainly dry, some damp patches 5 April '59
mainly dry, May '59
seepages, 13 April '59
Damp to dry, few seepages and 1 small flow as shown, May '59
FEELER HOLES & GROUTING
None
None
None
PHOTOGRAPHS CATALOGUE No.
15629
15630
13378 – 13382, 13427 – 13432
13434 – 13444, 13529 – 13549
FINAL SUPPORT OR LINING ESTIMATED AS CONSTRUCTED
CONCRETE LINING
FEET
10 0 10 20
LEGEND
STRIKE & DIP OF JOINT (WITH TREND AND PLUNGE OF SLICKENSIDES)
STRIKE OF VERTICAL JOINT
HORIZONTAL JOINT
STRIKE & DIP OF FOLIATION
STRIKE OF VERTICAL FOLIATION
FAULT, SHEARED ZONE, Showing dip
SHEARED or CRUSHED ZONE Showing dip and width
FAULT. Showing relative movement
200 WATER INFLOW 200 gallons per hour
S SEEPAGE
PHOTO BLACK & WHITE
STEREO COLOUR PHOTO
ROCK TYPES
Boomerang Creek Granitic Gneiss
Happy Valley Granite
Porphyry
SPECIMEN COLLECTED
TUNNEL SUPPORTS
STEEL SETS
WALL PLATES
R.B. ROCK BOLTS
MESH
GUNITE
SCIENTIFIC SERVICES
ENGINEERING GEOLOGY BRANCH
LOGGED DWS
DRAWN L.P. & D.H.S.
20 Apr. 59
ENGINEERING GEOLOGIST
BRANCH HEAD
SNOWY MOUNTAINS HYDRO ELECTRIC AUTHORITY
TUMUT. 2. PROJECT
CONTROL BUILDING ACCESS GALLERY & MACHINE HALL CROWN DRIVE
GEOLOGICAL LOG
DATE LOGGED as face advanced
SHEET 1 of 2
T8/141
D/EG 310

FIGURE 9

the rock surface was usually formed by joint faces, and the remaining 50 percent showed newly formed fractures across the rock fabric. This rather uniform breakage was attributed to the uniform pattern of the two main sets of steeply dipping joints which intersect the walls.

In the roof, however, marked difference in the type of breakage were seen from one area to another. Between tunnel stations (1) + 80 and (1) + 155 (75 feet), the roof was intersected only by steeply dipping joints, spaced between 1 and 12 inches apart. The slices or blocks between them remained confined laterally, and were not cut by any other joints at shallow angles close to the roof surface. Breakage in this section therefore occurred almost entirely across the rock fabric, and along the loaded and unloaded blast-holes. In such sections as this, where joints are oriented mainly at or near 90° to the excavation surface, their orientation is described as favourable.

In the adjacent 50-foot long section between stations (1) + 155 and (1) + 205, several tightly closed flat-lying joints and thin soft seams intersected the roof at very shallow angles. Some steeply dipping joints and crushed seams were also present. During excavation the rock broke entirely along flat-lying joints above the line of the loaded and unloaded "crown" holes, resulting in as much as 18 inches of overbreak in some parts. The orientation of the flat-lying joints in this section is described as unfavourable. Rock bolt patterns installed within 10 feet from the working face were adequate to prevent any progressive loosening up along these flat-lying joints during subsequent blasts.

In the next stage the sides of this crown drive were "stripped" outwards, first to form an arch section 51 feet wide, and then widened further to 61 feet, as shown on the geological log, Figure 10. A firing pattern with only alternate "crown" holes loaded was adopted. The "stripping" commenced 70 feet from the eastern or Control

Building end of the roof, and both sides were advanced together towards the Assembly Bay or western end. Support was by means of rock bolt patterns, as shown on Plate 13 and Figure 10.

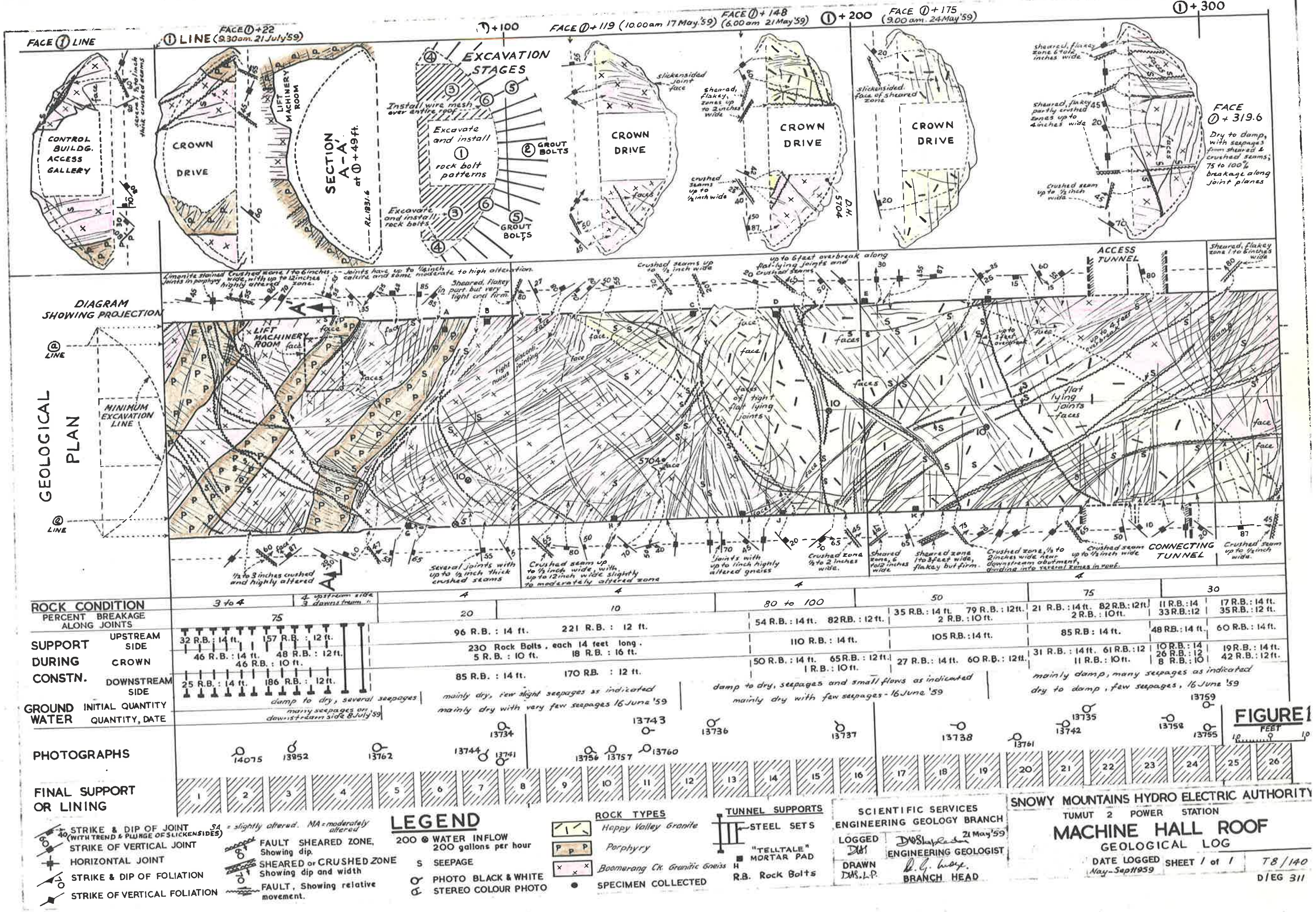
The 70-foot long section at the Control Building end was then widened. During widening of the last 54 feet, steel arch ribs with timber lagging were installed against the roof after the installation of rock bolts. The decision to install steel ribs as a supplement to the rock bolt supports was made after consideration of three main factors. The first was the inferior nature of the rock in this section, when compared with that exposed in the remainder of the roof. The second was that the roof arch in part of this section was to have a 16-foot high hole of rectangular section excavated into it, as shown on Section A-A' on Figure 10. The third was the stipulation that the excavation was to be deepened some 20 feet below the arch abutment level before placement of the concrete rib supports could commence. It was felt that this section of the roof, although excavated and rock bolted without difficulty, could become loosened during the several months in which the excavation was deepened. Huggenberger type strainmeters were installed in the steel ribs so that any build-up of load on them could be observed.

Stripping of the arch to full size presented no difficulty. For the greater part the joints and narrow seams were favourably oriented, and a well-shaped arch with little overbreak was obtained. Although the percentage breakage along joint planes was quite high towards the western end of the roof, many of the drill hole traces were still visible on completion of the excavation, indicating that the rock had not broken along joints above the drill holes, but on ones which almost coincided with the excavation surface. This type of breakage was of course most common in the closely but tightly jointed rock, where the large number of joint planes resulted in a high percentage breakage, but their favourable orientation prevented breakage along them extending very far above the drill holes.

The maximum overbreak in the roof arch was of the order of 6 feet, and occurred in the upstream "shoulder" of the arch near station (1) + 180, as shown on Figure 10. It occurred along the flat-lying joints and steeply dipping crushed seams already mentioned in the section on the crown drive.

The geological log of the completed roof (Figure 10), was commenced by transferring the geological detail from the roof of the crown drive into its appropriate place, and then filling in the detail on either side. The latter was done in small sections as the stripping operation advanced. At the time of excavation of the roof, only two shifts, each of 9½ hours, were being worked by the Contractor. The logging was mostly carried out during the quiet off-shift periods. This enabled a more careful examination of the rock than would have been possible during the noise and activity of the construction, and it was the only time in which the air was sufficiently clear of diesel fumes for photographs to be taken. The roof log was brought up to date every few days, and together with face-logs and cross sections, it was used to keep a constant check on the geology, so that any geological situation endangering the arch stability could have been recognised quickly and appropriate action taken.

An officer of the Authority's Physical Sciences Branch made a further check on the stability of the excavation by observing the frequency of sub-audible rock noises at the site. These observations were made using the listening method developed by the U.S. Bureau of Mines (Obert and Duvall, 1957), and could only be made during the off-shift periods or on Sundays. Observations were commenced in the Access Tunnel some weeks before the start of the Machine Hall excavation, in order to obtain average figures for the frequency of noises in stable rock in



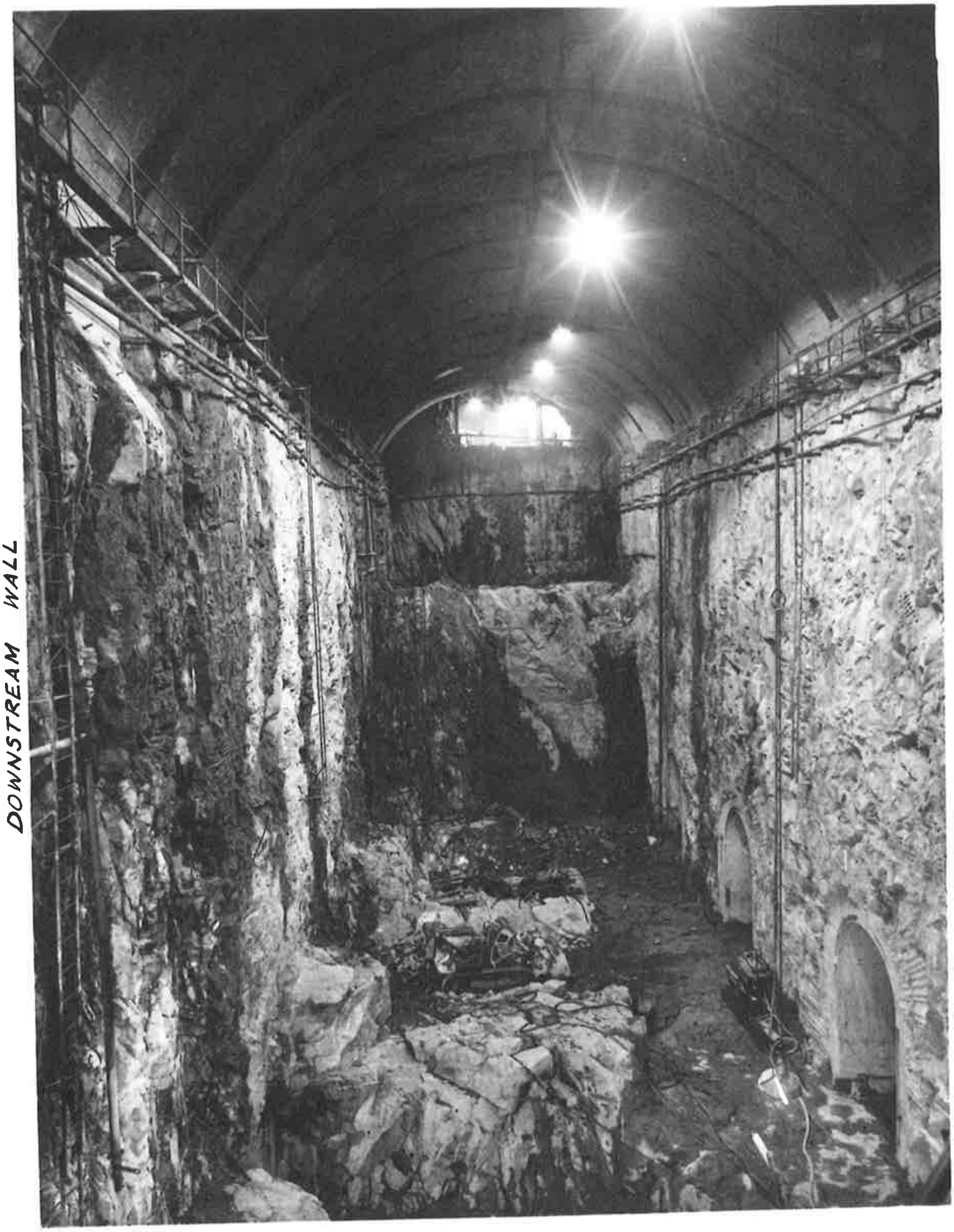
FACE ① LINE
① LINE (9.30am. 21 July '59)
FACE ① + 22
① + 100
FACE ① + 119 (10.00am 17 May '59)
FACE ① + 148 (6.00am 21 May '59)
① + 200
FACE ① + 175 (9.00am. 24 May '59)
① + 300
CONTROL BUILDG. ACCESS GALLERY
CROWN DRIVE
LIFT MACHINERY ROOM
SECTION A-A' at ① + 49 ft.
EXCAVATION STAGES
Install wire mesh over entire roof
Excavate and install ① rock bolt patterns
② GROUT BOLTS
⑤ GROUT BOLTS
Excavate and install rock bolts
CROWN DRIVE
CROWN DRIVE
CROWN DRIVE
FACE ① + 319.6
Dry to damp, with seepages from sheared & crushed seams; 75 to 100% breakage along joint planes
ACCESS TUNNEL
CONNECTING TUNNEL
DIAGRAM SHOWING PROJECTION
GEOLOGICAL PLAN
MINIMUM EXCAVATION LINE
ⓐ LINE
ⓑ LINE
LIFT MACHINERY ROOM
ROCK CONDITION
PERCENT BREAKAGE ALONG JOINTS
SUPPORT DURING CONSTN.
UPSTREAM SIDE
CROWN
DOWNSTREAM SIDE
GROUND WATER
INITIAL QUANTITY
QUANTITY, DATE
3 to 4
75
20
4
10
80 to 100
50
75
4
30
32 R.B. : 14 ft.
157 R.B. : 12 ft.
46 R.B. : 14 ft.
48 R.B. : 12 ft.
46 R.B. : 10 ft.
25 R.B. : 14 ft.
186 R.B. : 12 ft.
96 R.B. : 14 ft.
221 R.B. : 12 ft.
230 Rock Bolts, each 14 feet long.
5 R.B. : 10 ft.
18 R.B. : 16 ft.
85 R.B. : 14 ft.
170 R.B. : 12 ft.
54 R.B. : 14 ft.
82 R.B. : 12 ft.
110 R.B. : 14 ft.
50 R.B. : 14 ft.
65 R.B. : 12 ft.
1 R.B. : 10 ft.
35 R.B. : 14 ft.
79 R.B. : 12 ft.
2 R.B. : 10 ft.
105 R.B. : 14 ft.
27 R.B. : 14 ft.
60 R.B. : 12 ft.
21 R.B. : 14 ft.
82 R.B. : 12 ft.
2 R.B. : 10 ft.
85 R.B. : 14 ft.
31 R.B. : 14 ft.
61 R.B. : 12
11 R.B. : 10 ft.
11 R.B. : 14
33 R.B. : 12
48 R.B. : 14 ft.
10 R.B. : 14
26 R.B. : 12
8 R.B. : 10
17 R.B. : 14 ft.
35 R.B. : 12 ft.
60 R.B. : 14 ft.
19 R.B. : 14 ft.
42 R.B. : 12 ft.
damp to dry, several seepages
many seepages on downstream side 8 July '59
mainly dry, few slight seepages as indicated
mainly dry with very few seepages 16 June '59
damp to dry, seepages and small flows as indicated
mainly dry with few seepages - 16 June '59
mainly damp, many seepages as indicated
dry to damp, few seepages, 16 June '59
PHOTOGRAPHS
14075
13852
13762
13744
13741
13734
13756
13757
13760
13743
13736
13737
13738
13761
13735
13742
13758
13755
13759
FINAL SUPPORT OR LINING
FIGURE 1
FEET
LEGEND
STRIKE & DIP OF JOINT (WITH TREND & PLUNGE OF SLICKENSIDES)
STRIKE OF VERTICAL JOINT
HORIZONTAL JOINT
STRIKE & DIP OF FOLIATION
STRIKE OF VERTICAL FOLIATION
SA = slightly altered. MA = moderately altered
FAULT SHEARED ZONE, Showing dip.
SHEARED or CRUSHED ZONE Showing dip and width
FAULT, Showing relative movement.
200 WATER INFLOW 200 gallons per hour
S SEEPAGE
PHOTO BLACK & WHITE
STEREO COLOUR PHOTO
SPECIMEN COLLECTED
ROCK TYPES
Happy Valley Granite
Porphyry
Boomerang Ck. Granitic Gneiss
TUNNEL SUPPORTS
STEEL SETS
"TELLTALE" MORTAR PAD
R.B. Rock Bolts
SCIENTIFIC SERVICES
ENGINEERING GEOLOGY BRANCH
LOGGED
DRAWN
ENGINEERING GEOLOGIST
21 May '59
BRANCH HEAD
SNOWY MOUNTAINS HYDRO ELECTRIC AUTHORITY
TUMUT 2 POWER STATION
MACHINE HALL ROOF
GEOLOGICAL LOG
DATE LOGGED May-Sept 1959
SHEET 1 of 1
T8/140
D/EG 311

the area. Throughout the excavation of the roof arch the frequency of noises remained close to this low average figure (1 to 5 noises in 10-minute periods) and indicated a stable condition. Higher rates of noise (10 to 50 in 10-minute periods) were recorded during the first 20 minutes after blasting, but in every instance the normal rates were in evidence within 1 hour after each blast.

Deepening of Machine Hall to RL. 1800

On completion of the roof arch excavation, all rock bolts were grouted, and "Cyclone" wire mesh was installed over the entire roof. The mesh was attached to existing rock bolts by means of additional bearing plates, as shown on Plate 13, Stage 4. The excavation was then deepened to R.L. 1800, i.e. 27 feet below roof abutment level, along its entire length. This was several feet deeper than the minimum depth stipulated, but was convenient as it was the final depth of the shallow end portions of the Machine Hall. At the same time excavation of the draft tube and penstock tunnels was in progress (Plate 13, Stage 5).

The deepening to RL. 1800 was carried out by "benching" or quarrying methods, mostly using vertically drilled blast holes. In general it was carried out in 6 to 8-foot deep "lifts", which commenced at the Assembly Bay end, and advanced towards the Control Building end. Vertical "line-drill holes" spaced 4 to 8 inches apart were drilled along or close to the wall lines, and were left uncharged. The closest charged holes to these line-drill holes were usually 12 inches away. The line drilling method was adopted to protect the walls from damage due to blasting close to them. It was also hoped to provide a relatively weakened plane along the wall line, along which the rock would tend to break. Where the rock broke on the excavation side of the line-drill holes (i.e. under-break occurred), the rock had to be trimmed back to the wall line by hand methods, usually by sledge hammers and steel wedges.



DOWNSTREAM WALL

UPSTREAM WALL

PLATE 14. EXCAVATION OF MACHINE HALL (CONTINUED)

STAGE 7. Completion of excavation by benching downwards. Rock spoil was removed through the Draft Tube tunnels, which are at the base of the downstream wall, on the left hand side of the photograph.

The walls were supported by patterns of downwards sloping rock bolts, as shown diagramatically on Plate 13. The bolts were spaced 4 feet apart in horizontal rows, and the vertical distance between adjacent rows was 5 feet. The first few rows below abutment level were mainly of 14-foot long bolts. The remainder of the bolts used were 12 feet long. The bolts were installed as soon as possible after the rock was exposed, and were grouted in place at the time of installation.

Detailed geological logs of the vertical walls were prepared as benching progressed. For each wall the geological detail was plotted on an elevation, which was used in conjunction with strip plans made at several levels. To display the type of rock breakage in the walls, the areas showing the line drill hole traces were delineated. In these areas the rock had broken either across its own fabric as shown on Plate 15 or else partly along closely spaced favourably oriented joints. In the areas not showing line-drill hole traces, the rock usually had broken back along joints, and the rock surface consisted mainly of intersecting joint faces (Plate 16).

Figure 11, the log of the upstream wall, shows a plan 2 feet below abutment level, along line X - X'. This plan shows the extremity of the abutment, together with the (a) Line, or minimum excavation line, for the wall, and also the actual shape in plan of the excavated wall surface at this level. The sloped distance from the (a) Line to the abutment extremity, i.e. the designed maximum width of the abutment, is 4 feet 10 inches. The minimum allowable abutment width was 2 feet 6 inches, i.e. the nominal thickness of the concrete roof-ribs at this point. It can be seen from the plan that the abutment was preserved close to its maximum width for about half the length of the upstream wall. In most of the parts showing the full abutment width, the elevation shows that the wall broke along the line-drill holes. The remaining half broke mainly along joints, resulting in overbreak ranging from several inches up to a maximum of 4 feet. In several very short sections of both the upstream and downstream walls, the required minimum abutment width was not obtained and it was necessary to extend the concrete roof ribs slightly below the nominal abutment level.

Overbreak was usually greatest where joints or seams made shallow angles with the wall line. The worst section was above the Access tunnel portal, where the abutment rock was unconfined on three sides instead of the normal two. It had also been affected by several more close blasts than the rock elsewhere. It was necessary to extend the reinforced concrete portal structure upwards in this area to form the roof abutment. The worst overbreak section in the downstream wall is shown on Plate 14, in the left foreground, and also on Figure 12. As much as 4 feet of overbreak occurred in this area, along three steeply dipping joints which intersected behind the wall and isolated a wedge shaped slab of granite. A 10-foot long section of the wall and abutment was replaced by concrete in this area.

It was observed while logging that many joints in the rock within 10 to 15 feet of abutment level had opened up during the excavation. Many in the abutment itself were open as much as 0.05 inches, and a few as much as 0.2 inches. A high percentage of the rock breakage at these levels was along joint planes, and overbreak was more prevalent than at lower levels.

Some deterioration in the near-abutment rock was to be expected, as it had been subjected to close blast shocks during the excavation of both roof and walls. In addition, it was partly unconfined at the top as well as at the wall surface. Although these two factors certainly contributed to the loosening, the prime cause is considered to be the development of tensile stresses in this rock, as predicted by Worotnicki (1958) from photoelastic studies.

The visual observations near the abutments strongly suggested that progressive partial failure of the type shown on Figure 7, Diagram B, was occurring. Almost without exception the failure appeared to be occurring along existing joint planes. The avoidance of any total failure in the form of rockfall, and the relatively small amounts of total failure as overbreak, are attributed to the pattern rock bolting, performed as soon as possible after exposure of the rock in each lift.

It was observed that the line-drilling was least effective in areas which were intersected by closely spaced vertical joints in the two main sets (i) and (ii). The probable reason for this becomes obvious when the rock in such areas is envisaged as a series of elongated vertical prisms of square cross section, bonded together by the joint cements. Assuming a joint spacing of 6 inches, most prisms would have had one or perhaps two line drill holes vertically through them, and many of the holes would have passed straight down joints or joint intersections. One or two holes vertically down a prism centre were not sufficient to produce a plane of weakness across the rock fabric, especially not one approaching the low strength of most of the slickensided joints present. The rock therefore tended to break back to at least one joint intersection behind the line drill holes. In several cases where sets of vertical joints curved slightly to dip 60^{o} to 70^{o}, breakage began to occur along the line drill holes. It can be seen on Figures 11 and 12 that in most sections showing breakage to line drill holes the majority of joints dip 70^{o} or less. In these sections the joint blocks or prisms were oblique to the drill holes, and hence each was intersected by numerous holes, very few or none of which passed directly down joints.

Geological logs of the vertical end walls of the upper portion of the excavation are shown on Figure 13. The (1) Line or Control Building end wall was composed mostly of tightly but closely jointed

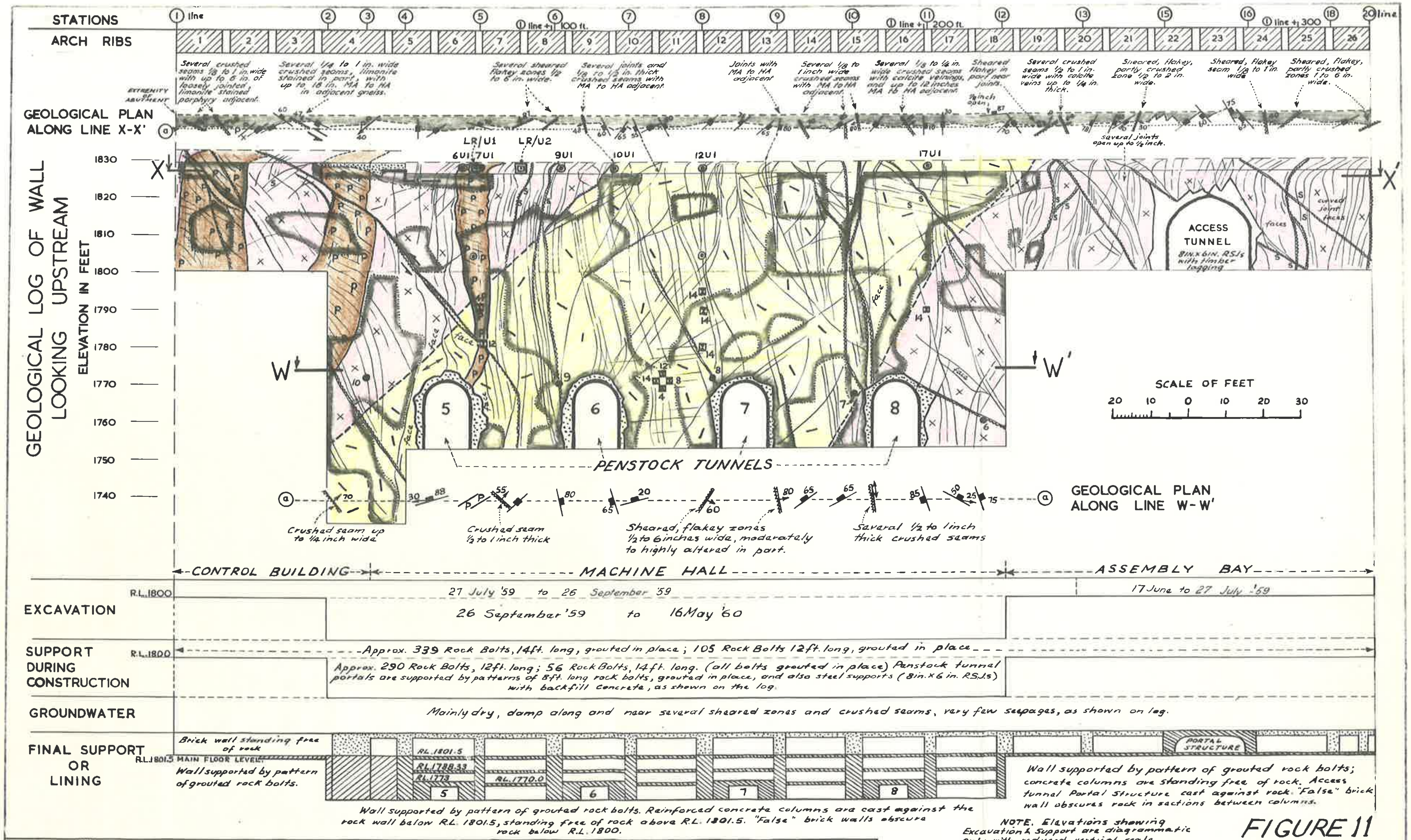


FIGURE 11

LEGEND

- AREA SHOWING BREAKAGE TO LINE DRILL HOLES
- LR/U2 56 FT. LONG MEASURING ROCK BOLT
- 14 14 FT. LONG @ CLINOMETER MEASURING ROCK BOLT
- STRIKE & DIP OF JOINT
- STRIKE OF VERTICAL JOINT
- HORIZONTAL JOINT
- STRIKE & DIP OF FOLIATION
- STRIKE OF VERTICAL FOLIATION
- MA MODERATE ALTERATION
- HA HIGH ALTERATION
- FAULT, SHEARED ZONE Showing dip
- SHEARED or CRUSHED ZONE Showing dip and width
- FAULT, Showing relative movement.
- 200 WATER INFLOW 200 gallons per hour
- S SEEPAGE
- PHOTO BLACK & WHITE
- STEREO COLOUR PHOTO
- SPECIMEN COLLECTED

ROCK TYPES
- BOOMERANG CREEK GRANITIC GNEISS
- HAPPY VALLEY GRANITE
- PORPHYRY
- CONCRETE

TUNNEL SUPPORTS
- BACKFILL CONCRETE
- 8IN.x6IN. R.S.J.

SCIENTIFIC SERVICES ENGINEERING GEOLOGY BRANCH	SNOWY MOUNTAINS HYDRO ELECTRIC AUTHORITY	
LOGGED DHS	D.H.Stapledon 17 June '60 ENGINEERING GEOLOGIST	TUMUT 2 POWER STATION MACHINE HALL UPSTREAM WALL GEOLOGICAL LOG
DRAWN DHS, LP.	BRANCH HEAD	COOMA, N.S.W. — DATE LOGGED AUG. 1959 MAY 1960 — SHEET 1 of 1 — T8/150

porphyry. This broke mainly along joints, but with very little overbreak. The wall was supported by a pattern of downwards sloping grouted rock bolts, mostly 12 feet long and spaced 4 feet apart horizontally and vertically. In the (20) Line or Assembly Bay end wall, considerable overbreak occurred along unfavourably oriented joints of set (ii), striking almost parallel to the wall, and dipping steeply towards the excavation. There was very little breakage along line-drill holes. Further overbreak and loosening of the rock along joints occurred when the excavations for the Workshop and survey niche were made into the wall. Downwards sloping grouted rock bolts, generally spaced 4 feet apart horizontally and vertically were installed as support. Many additional bolts were installed near the Workshop and Survey Niche.

During the benching to RL. 1800, a number of small rock fragments, mainly in the 2 to 4-inch size range, were caught up by the wire mesh placed previously over the roof arch. They were mostly fragments derived from closely jointed areas of rock or from sheared zones. The wire mesh itself was extensively damaged in many places by rock flying up during blasting. To facilitate placement of concrete against the roof, it was necessary to remove the wire mesh before concreting each arch rib. Before concreting of the ribs in the steel supported section at the Control Building end, the timber lagging between the steel arches and the rock was almost completely removed. Regular observations of the strain gauges set in the supports at the time of installation had shown that no build-up of the load had occurred during the deepening to RL. 1800 (Alexander 1959). The timber was removed without incident, and the rock-bolted arch behind the steel supports showed no sign of deterioration.

Completion of the Excavation

While concreting of the roof was in progress, excavation of the draft tube and penstock tunnels was completed. A vertical shaft was risen at each end of the proposed deep central section, to connect with the upper excavation, as shown on Plate 13. The rock within 10 feet of the future portals of the penstock and draft tube tunnels was reinforced, firstly by patterns of grouted rock bolts, and then by collars of concrete reinforced by steel rib supports. These are shown on Figures 11 and 12, and Plate 14.

Benching downwards commenced as soon as concreting of the arch was complete, and was carried out in a similar manner as before, except that rock spoil was removed via the two vertical shafts and the draft tube tunnels. Support was by patterns of grouted rock bolts, in which 12 feet long bolts were spaced 4 feet apart horizontally, and 5 feet vertically. Additional bolts were installed between the (11) and (12) Lines in the downstream wall, where some opening up of joints was observed. Some bolts 14 feet in length were installed above the portals of the penstock and draft tube tunnels.

Visual inspection and geological logging indicated that in general more smooth walls with less overbreak were obtained during this final deepening. More than half of the area of the upstream wall below RL. 1800 showed breakage along line drill holes, compared with about one-third above that level. The downstream wall showed about one third of its area broken along the drill holes, both above and below RL. 1800, but overbreak was noticeably less in the lower part.

The reinforced concrete collars at the portals of the penstock and draft tube tunnels were highly successful in preventing overbreak at these places. The upstream wall was line-drilled radially from within the tunnels, and the smooth breakage which resulted can be seen on Plate 14.

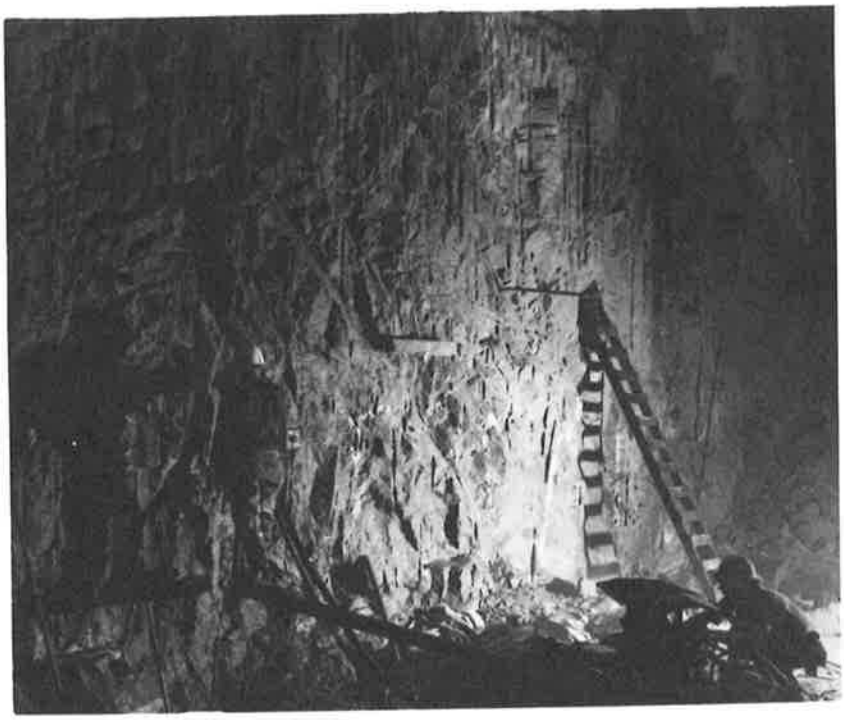


PLATE 15. Granite in the central portion of the Machine Hall downstream wall, showing breakage along line-drill holes and across the rock fabric.

PLATE 16. Granitic gneiss at the Control Building end of the downstream wall. The rock has broken along joints and minor faults, rather than across its own fabric. Few traces of line drill holes remain.

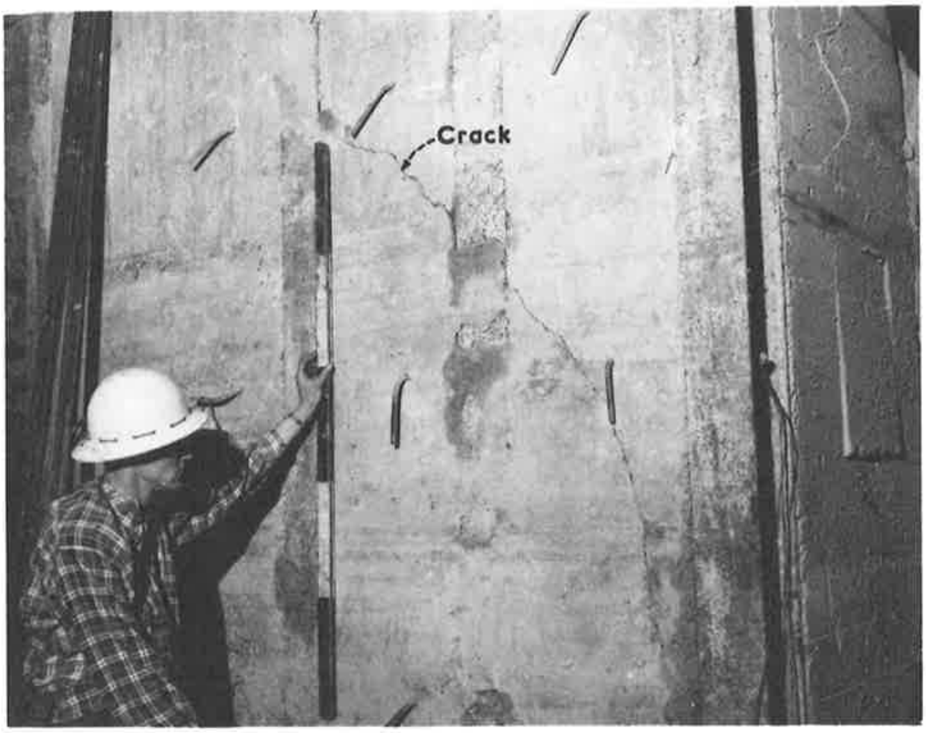
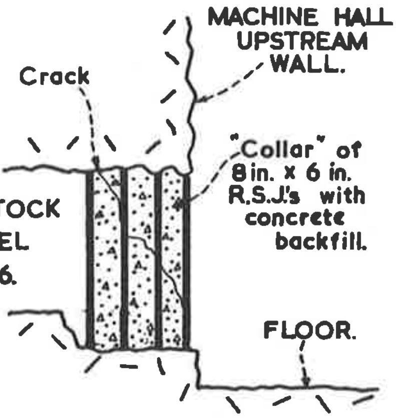


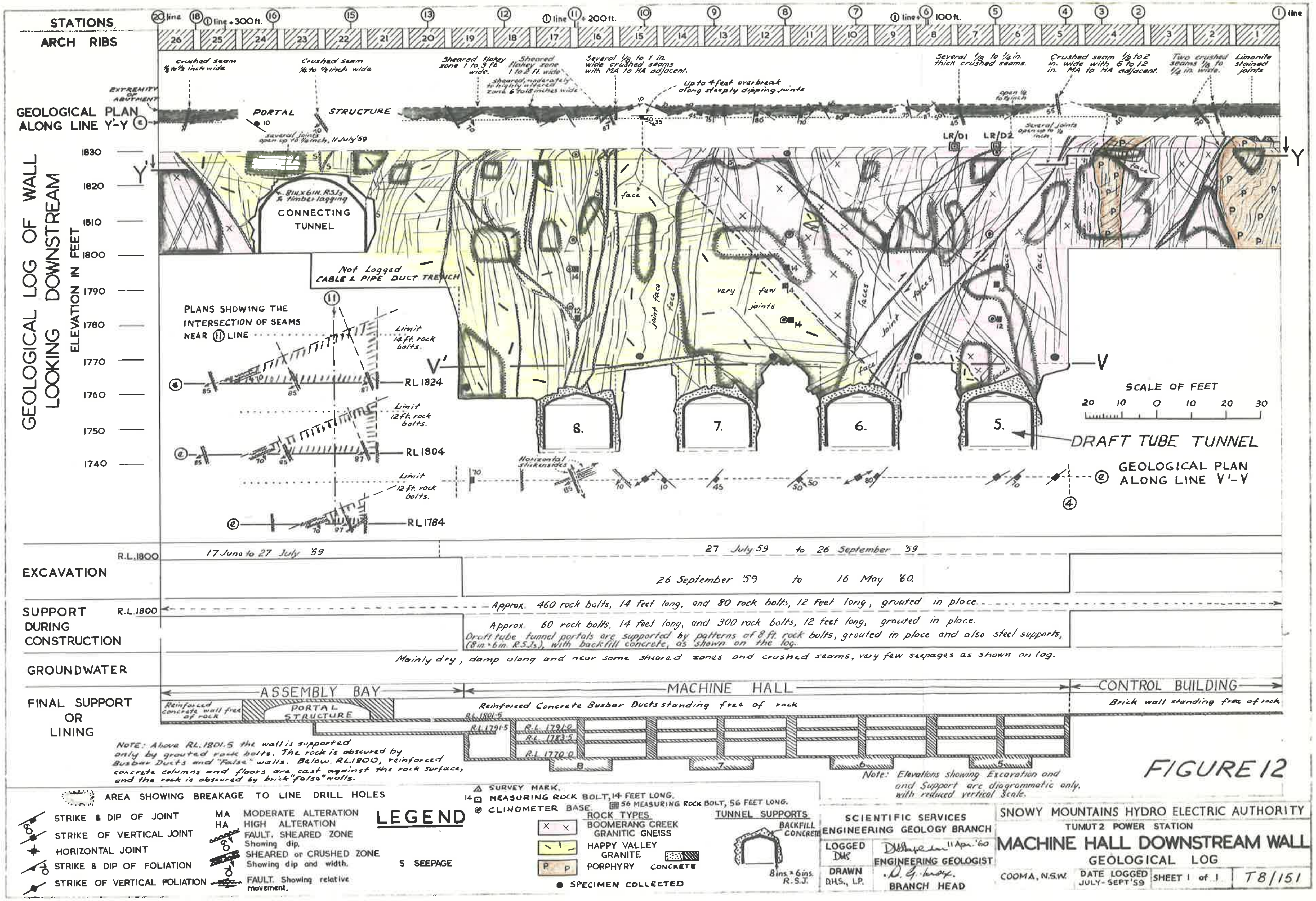
PLATE 17.

Crack open as much as 0.05 inch developed in the reinforced concrete collar of Penstock Tunnel No. 6, during excavation of the Machine Hall.



As the portal of each penstock and draft tube tunnel became exposed by the deepening, cracks similar to that shown on Plate 17 developed in their 10-foot long concrete collars. The worst cracking was seen in Draft Tube No. 8 tunnel, where several cracks with total opening of 0.2 inches occurred. The cracks were in some places en-echelon, and in every instance were dipping steeply (50^{o} to 80^{o}) towards the Machine Hall excavation, and indicated slight rock movements towards the Machine Hall. In some places the bond between the concrete collar and the rock was clearly broken, and hence the concrete movements had probably only reflected part of the actual rock movements which occurred. There was no definite evidence that additional rock failure occurred further back along the tunnels during the deepening. Small movements could have occurred along joints, some of which were slightly open both before and after the deepening. Moye (1958) describes similar cracking in concrete collars in the penstock tunnels at Tumut 1. Rock failure which occurred further back along the penstock tunnels is also described. This occurred both along joints and by spalling across the rock fabric.

The behaviour of the rock walls during deepening was observed by means of instruments installed and read by officers of Physical Sciences Branch. One type of instrument, known as The Measuring Rock Bolt, has been described by Alexander (Reference 4, 1960), and is shown diagrammatically on Figure 7. Several such bolts, mostly 12 and 14 feet long, were installed horizontally at various levels on the (5) and (8) and (11) lines of both walls, as shown on Figures 11 and 12. Each bolt had a hollow core, into which a steel reference rod was inserted and welded to the far end. The bolts were anchored and tightened, and grouted in place along their entire lengths. Extensions which occurred due to inward rock movements were indicated directly by movements of the projecting ends of the bolts in relation to their



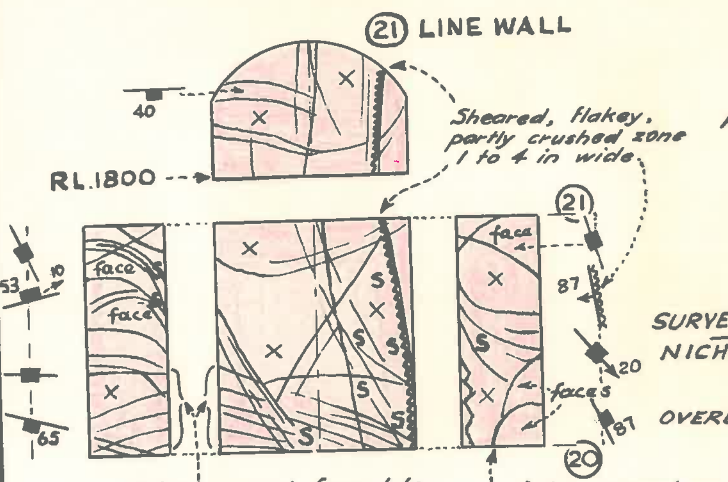
STATIONS
ARCH RIBS
GEOLOGICAL PLAN ALONG LINE Y'-Y
GEOLOGICAL LOG OF WALL LOOKING DOWNSTREAM
ELEVATION IN FEET
1830
1820
1810
1800
1790
1780
1770
1760
1750
1740
PORTAL
STRUCTURE
EXTREMITY OF ABUTMENT
Crushed seam 1/8 to 1/2 inch wide
Crushed seam 1/4 to 1/2 inch wide
Sheared flakey zone 1 to 3 ft. wide.
Sheared flakey zone 1 to 2 ft. wide
Several 1/8 to 1 in. wide crushed seams with MA to HA adjacent.
Up to 4 feet overbreak along steeply dipping joints
Several 1/8 to 1/4 in. thick crushed seams.
Crushed seam 1/2 to 2 in. wide with 6 to 12 in. MA to HA adjacent.
Two crushed seams 1/8 to 1/4 in. wide.
Limonite stained joints
several joints open up to 1/4 inch, 11 July '59
8 in x 6 in. R.S.Js & timber lagging
CONNECTING TUNNEL
Not Logged
CABLE & PIPE DUCT TRENCH
PLANS SHOWING THE INTERSECTION OF SEAMS NEAR 11 LINE
Limit 14 ft. rock bolts.
Limit 12 ft. rock bolts.
RL 1824
RL 1804
RL 1784
very few joints
face
joint face
8.
7.
6.
5.
DRAFT TUBE TUNNEL
GEOLOGICAL PLAN ALONG LINE V'-V
SCALE OF FEET
20 10 0 10 20 30
LR/D1
LR/D2
Horizontal slickensides
EXCAVATION
R.L.1800
17 June to 27 July '59
27 July '59 to 26 September '59
26 September '59 to 16 May '60.
SUPPORT DURING CONSTRUCTION
Approx. 460 rock bolts, 14 feet long, and 80 rock bolts, 12 feet long, grouted in place.
Approx. 60 rock bolts, 14 feet long, and 300 rock bolts, 12 feet long, grouted in place.
Draft tube tunnel portals are supported by patterns of 8 ft. rock bolts, grouted in place and also steel supports, (8 in. x 6 in. R.S.Js), with backfill concrete, as shown on the log.
GROUNDWATER
Mainly dry, damp along and near some sheared zones and crushed seams, very few seepages as shown on log.
FINAL SUPPORT OR LINING
ASSEMBLY BAY
MACHINE HALL
CONTROL BUILDING
Reinforced concrete wall free of rock
PORTAL STRUCTURE
Reinforced Concrete Busbar Ducts standing free of rock
Brick wall standing free of rock
R.L. 1801.5
R.L. 1791.5
R.L. 1791.0
R.L. 1783.5
R.L. 1770.0
NOTE: Above RL.1801.5 the wall is supported only by grouted rock bolts. The rock is obscured by Busbar Ducts and "False" walls. Below RL.1800, reinforced concrete columns and floors are cast against the rock surface, and the rock is obscured by brick "false" walls.
Note: Elevations showing Excavation and Support are diagrammatic only, with reduced vertical scale.
LEGEND
AREA SHOWING BREAKAGE TO LINE DRILL HOLES
STRIKE & DIP OF JOINT
STRIKE OF VERTICAL JOINT
HORIZONTAL JOINT
STRIKE & DIP OF FOLIATION
STRIKE OF VERTICAL FOLIATION
MA MODERATE ALTERATION
HA HIGH ALTERATION
FAULT, SHEARED ZONE Showing dip.
SHEARED or CRUSHED ZONE Showing dip and width.
FAULT. Showing relative movement.
S SEEPAGE
SURVEY MARK.
MEASURING ROCK BOLT, 14 FEET LONG.
56 MEASURING ROCK BOLT, 56 FEET LONG.
CLINOMETER BASE.
ROCK TYPES
BOOMERANG CREEK GRANITIC GNEISS
HAPPY VALLEY GRANITE
PORPHYRY
CONCRETE
SPECIMEN COLLECTED
TUNNEL SUPPORTS
BACKFILL CONCRETE
8 ins. x 6 ins. R.S.J.
SCIENTIFIC SERVICES
ENGINEERING GEOLOGY BRANCH
LOGGED DHS
11 Apr. '60
ENGINEERING GEOLOGIST
DRAWN D.H.S., L.P.
BRANCH HEAD
SNOWY MOUNTAINS HYDRO ELECTRIC AUTHORITY
TUMUT 2 POWER STATION
MACHINE HALL DOWNSTREAM WALL
GEOLOGICAL LOG
COOMA, N.S.W.
DATE LOGGED JULY-SEPT '59
SHEET 1 of 1
T8/151

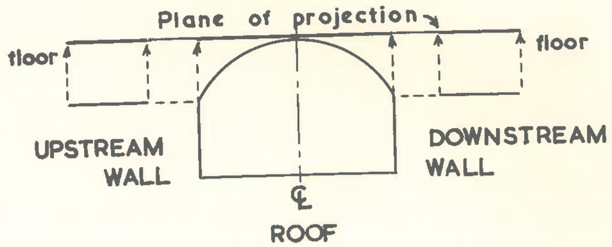
FIGURE 12

GRANITIC GNEISS — PORPHYRY — STRIKE & DIP OF JOINT

S SEEPAGE — AREA SHOWING TRACES OF LINE DRILL HOLES — STRIKE & DIP OF SHEARED OR CRUSHED ZONE

WORKSHOP GEOLOGICAL LOG





EXCAVATED: 30 July to 11 August, 1959

ROCK CONDITION: Mainly 4, some 3 within 4 feet of 20 line.

BREAKAGE TO JOINTS: Roof and 21 line wall 20%, walls 50 to 100%.

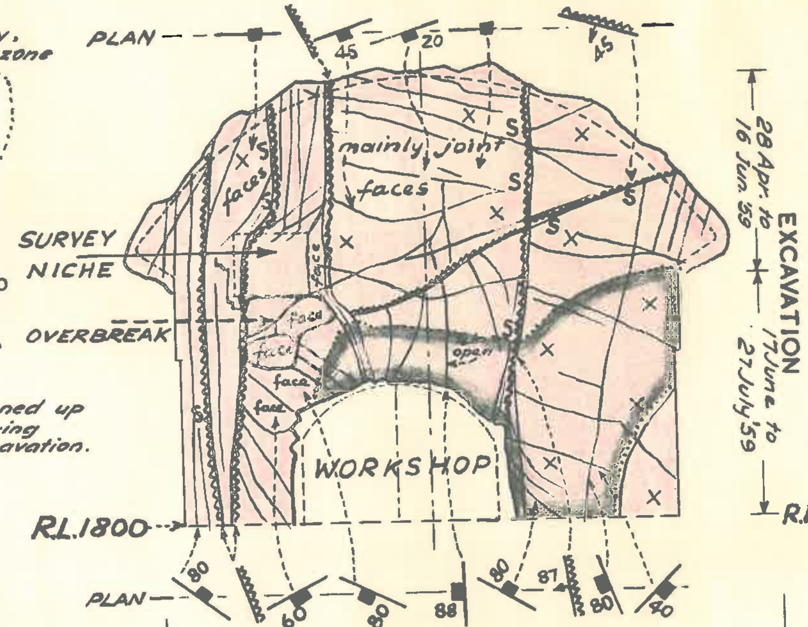
SUPPORT DURING CONSTUCTION: None.

FINAL TREATMENT: Reinforced Concrete Lining.

GROUNDWATER: Mainly dry, seepages as shown.

PHOTOGRAPHS: 14030, 14185

20 LINE WALL GEOLOGICAL LOG



ROCK CONDITION: Mainly 4

BREAKAGE TO JOINTS: 75 to 100%

SUPPORT DURING CONSTRUCTION: 155 rock bolts, mainly 12 feet long, grouted.

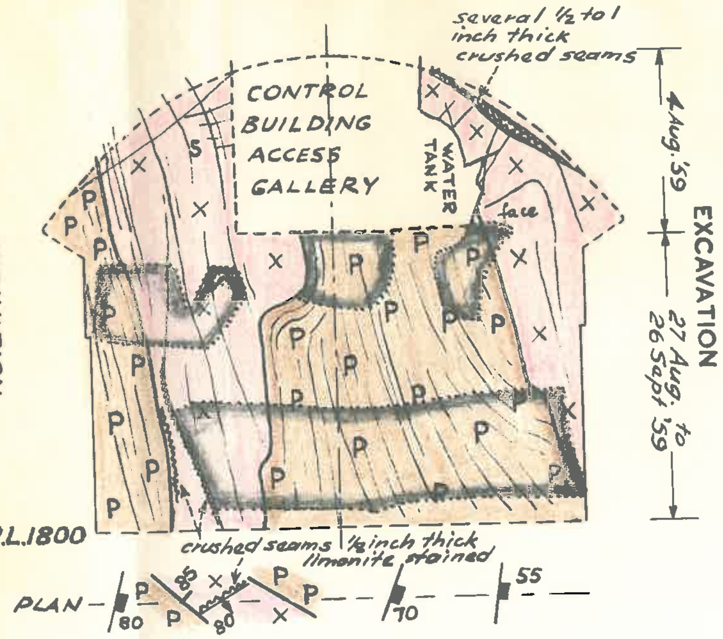
GROUNDWATER: Mainly dry, seepages as shown.

PHOTOGRAPHS: 14030, 13759, 13755, 14178, 14322, 14321

SCALE OF FEET

10 0 10 20 30

1 LINE WALL GEOLOGICAL LOG



ROCK CONDITION: 4

BREAKAGE TO JOINTS: 50%

SUPPORT DURING CONSTRUCTION: 112 rock bolts, each 12 feet long, grouted

GROUNDWATER: Mainly dry.

PHOTOGRAPHS: 14331, 14337.

SCIENTIFIC SERVICES DIVISION

DRAWN DWS, L.P. — TRACED DWS — CHECKED

ENGINEERING GEOLOGIST — BRANCH HEAD — ENGINEER IN CHARGE

COOMA, N.S.W. DATE: 3 SEP. '59

SNOWY MOUNTAINS HYDRO-ELECTRIC AUTHORITY

TUMUT 2 POWER STATION

1 LINE WALL; 20 LINE WALL; WORKSHOP. GEOLOGICAL LOG.

T8/152

FIGURE 13

reference rods. Diagram C on Figure 7 was prepared by L.G. Alexander (1960, Reference 4) to show the large extensions which were obtained in some of these bolts. The four bolts referred to in the diagram were located between RL. 1790 and 1795 in the downstream wall of the Machine Hall, and are shown on Figure 12. It can be seen that large extensions were obtained, indicating loads which in two cases exceeded the elastic limit of the bolts.

Alexander (1959, Reference 2, and 1960, Reference 4) interprets these elongations as elastic strain in the rock due to stress reduction, together with some opening up along joints. The largest extension obtained was in bolt 11D, at RL. 1795 on the (11) line on the downstream wall. A close examination was made of this area when the large extensions became apparent. It appeared that an inverted triangular pyramid of rock was in effect isolated from the surrounding rock by the intersection of several minor faults, as shown by the three diagrammatic plans and the elevation on Figure 12. In many places at the face of this block, cracking of dust and grout films over joints indicated that slight movements were taking place as the excavation was deepened. A fall of rock estimated as one third of a cubic yard occurred from the very loose surface layer during an off-shift period. It was concluded that this block had moved inwards towards the excavation to a greater extent than the rock elsewhere, which mostly had well interlocked joint blocks. The sections on Figure 12 also show the limits of penetration of the rows of 14 and 12-foot long grouted rock bolts which supported this section of the wall. The final elongation recorded on bolt 11D indicated that these bolts were probably stressed close to their yield point, assuming that the strain was evenly distributed along them. As the bolts were grouted in place, it was possible that failure of the grout surround could occur locally and concentrate the strain on short sections of the bolts, and thus result in bolt failure. Additional support in this area was considered, and probably would have been necessary, if the bolt extensions had not virtually ceased on completion of the excavation.

During the final excavation of the Machine Hall floor, three pillars of rock were left between the machine pits, as shown on Plate 14 and Figure 12. Rock bolts sloping downwards were employed to control overbreak and opening up along joints which occurred during excavation around the pillars. The deterioration of this rock was attributed partly to the number of close blasts to which it had been subjected, as it became unconfined on three and partly four sides. The partial development of tensile stresses in these projecting pillars, as indicated by Worotnicki's photoelastic studies, must however have been the main cause.

DISCUSSION

The site selection and design stage investigations for Tumut 2 followed fairly closely the pattern set several years earlier during investigations for Tumut 1 Power Station. Surface geological mapping, diamond drilling, and seismic refraction work, led to the adoption of the most suitable of several possible sites for Tumut 2. To obtain the detailed geological picture necessary for design of the station, this site was then explored by an exploratory tunnel, and by diamond drill holes commenced from the tunnel. The drill holes and exploratory tunnel were used in studies of the permeability of the rock mass, and of groundwater quantities and pressures. Measurements of primary rock stresses were made in the tunnel.

The rock and groundwater conditions proved to be very much as anticipated from the above studies, and the station has been constructed almost exactly as designed. These results demonstrate the value of the geological work, and they also reflect the effectiveness of the pattern adopted for the investigations.

Several features of the work appear to have been particularly useful. Probably foremost among these is the choice of the area near Eight Mile Creek for the site of the power station. This area was adopted after several other areas further downstream had been rejected largely because of unfavourable rock conditions. Construction of the Tailwater tunnel near these rejected areas has since confirmed their inferior nature. Also of note was the choice of orientation for the large openings, particularly the Machine Hall. The pattern of jointing found during excavation was very much the same as that anticipated earlier and assumed in the designs. The Machine Hall was oriented so that its long axis made favourable angles with the two sets of nearly vertical joints which were prominent. This favourable orientation certainly contributed to the overall stability of that chamber, and in particular, to the stability of the high vertical walls. Another feature was the lack of appreciable groundwater inflows at the site, and the dry, tightly jointed nature of most of the rock met during construction. This showed the value of the groundwater and rock permeability studies made during site selection and design.

There are two particular aspects of the Tumut 2 investigations which are considered worthy of further comment, and which suggest slight modifications in future investigations of this type.

The first comment refers to part of the site selection work. It will be recalled that only one diamond drill hole (D.H. 5779) was drilled from the ground surface into the adopted site. In retrospect it would appear that at least one vertical hole should have been drilled in addition to D.H. 5779, before commencement of the exploratory tunnel. D.H. 5779 had shown that several minor faults were present at the site, but being a sloped hole, it gave no reliable indication of the true dips of these structures. A vertical hole would have shown the true dips of the joints and faults it intersected. The presence of persistent flat-lying faults and joints close to the power station roof level would have suggested a significant defect at the site, and if such structures were found in the vertical hole, it may have been possible to make a suitable adjustment to the power station location, before commencement of the exploratory tunnel.

Additional drilling from the ground surface was in fact considered at the time of completion of D.H. 5779, and although the value of such work was appreciated, there were several reasons why it was not carried out. The inaccessable gorge of Eight Mile Creek made drilling difficult and costly, and holes of the order of 1000 feet in depth would have been required. With the funds and time available for completion of the investigation, the expense of this drilling and the delay in commencement of the exploratory tunnel could not have been tolerated.

The second comment refers to part of the design stage work. It is considered that the value of the exploratory tunnel would have been appreciably greater, had it been extended to pass along the whole length of the Machine Hall, several feet below the proposed roof level. Admittedly, it would have been undesirable to have extended the tunnel in this manner before the results of several diamond drill holes from the drilling chamber had enabled the position of the Machine Hall to be firmly decided upon. Such an extension of the exploratory tunnel, after the diamond drilling, would have given a very clear picture of the rock close to the Machine Hall roof level. To the geologist and designers it would have provided a timely check on the interpreted geological structure, and hence on the validity of many of the designs. To prospective tenderers it would have provided much more convincing evidence of the suitability of the rock in this critical area, than any number of diamond drill holes. These matters were considered at the time, and although the advantages of an extension of the tunnel were evident, it was found particularly difficult to justify, after having spent so much time and funds on the underground drilling. In future jobs a useful approach may be to do less diamond drilling, and to put more time and money into extending the exploratory tunnel. The problem of proper location for the extension would remain, however. If rock exposures in the extended tunnel resulted in a decision to move the Machine Hall slightly, or if such a slight move became necessary later due to poor rock exposed during the construction stage, then the tunnel extension could be located in a position where it could interfere with, or endanger the stability of, the re-located Machine Hall roof.

Generally speaking, the geological work during each of the site selection and design stages forms a large and important part of a planned investigation programme. The geologist's role is therefore a prominent one, and his work contributes directly towards such major decisions as choice of site and choice of type of engineering structure. During construction, however, his duties are mainly those of observer, advisor, and recorder, and to some extent the amount of attention his work receives depends upon the number and size of the geological problems which arise. In any case the value of the construction stage work tends to be less direct, and therefore less evident, than that of the earlier work.

In the case of Tumut 2, no major geological problem arose during construction. There were however, numerous day-to-day problems which required a detailed understanding of the mechanical properties of local sections of rock. The systematic geological inspections and mapping contributed towards the early recognition and treatment of these problems, and in this way contributed to the rapid progress made during construction. The excavations were completed well ahead of the construction schedule. The geological reports and logs, together with photographic records, are considered to contain data of appreciable value to those engaged in the planning and construction of future underground projects.

One possible criticism of the construction stage work is that the scale of the geological logging (1:240, or 20 feet to 1 inch) was too large. Experience showed, however, that this scale was very suitable for recording most of the significant features. Even larger scales (1:120, or 10 feet to 1 inch, and 1:60, or 5 feet to 1 inch) would have been more suitable to illustrate the rock structure in certain critical areas. Such areas include the roof abutments, the portals of the draft tube and penstock tunnels, and other local areas where problems of overbreak, rock damage, or support, required very detailed study. The latter very large scales were commonly used by the engineering staff on design or construction drawings of such areas, and it would have been ideal if the geological detail could have been plotted directly onto these, or else transferred onto them without change of scale.

The form of presentation of the geological logs was the subject of much study. It is considered that if used together with photographs, and written descriptions, the logs depict adequately the significant geological features. The main advantage of the Authority's standardized form of log is that the same data must be recorded objectively on each sheet, and the possibility of important omissions is avoided. The special attention given to explanatory cross sections or plans, i.e. the "multiple viewpoint" approach, and to strip plans showing true strikes and dips, is considered to have added greatly to the value of the logs.

CONCLUSION

It has been attempted in the foregoing discussion to make a critical review of the geological work, and to assess its value in terms of results. In the author's opinion the work was very worthwhile. It is also considered that much of the value of the work would have been lost without the logical approach to geology adopted by the Snowy Mountains Authority. As a member of the Authority's Engineering Geology Branch, the author was throughout all stages able to remain in close contact with the job, and with the numerous engineering staff also engaged on it. He became in effect a member of a team, each of whom worked towards the same ultimate goal, namely, the successful construction of Tumut 2. In many instances their immediate interests conflicted. A particular site may have appeared ideal from the geological viewpoint, but have been quite unsuitable for topographic or other reasons. Conversely, engineering studies could have favoured a site which was obviously quite unsuitable geologically. Often, when it was not possible to find an arrangement to suit ideally both the engineering and geological conditions, it was necessary to work out a compromise. Thus it is evident that any approach without this close contact with the engineering staff would have led to misunderstanding and unnecessary work.

The author has been most fortunate to be able to work on Tumut 2 Project during site selection, design, and construction. Such continuity of work is considered to be very valuable, both to the project, in terms of results, and to the geologist, in terms of the experience he gains.

Throughout all these stages, the work has of necessity been carried out to the same exacting time schedules as the parallel engineering studies. In a few instances more time for the work may have given more satisfactory results from the geological point of view. Some aspects of the work reviewed in the discussion are notable examples. It is considered, however, that except in projects presenting extraordinary geological problems, working to time schedules is very much to the advantage of both geologist and employer. The geologist knows that slow work may produce geological information too late for its effective use. He therefore needs a planned programme of work, and efficient field and office methods. These requirements, together with the knowledge that the studies have a very real purpose, make the geologist's work in an engineering organisation both challenging and gratifying.

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APPENDIX I

DESCRIPTIONS OF MAIN ROCK TYPES

Happy Valley Granite

Hand specimens show the granite as a mottled light grey, granular, medium or medium to coarse grained rock, without any indications of foliation or preferred orientation of minerals. Grey to bluish grey glassy quartz, together with white, opaque flespars and black biotite flakes, are readily distinguished. The felspars are often euhedral and tend to be porphyritic. In some specimens taken near the contacts with Boomerang Creek Granitic Gneiss, dull greenish grey grains of altered cordierite are visible in small amounts.

Microscopic examination shows the granite to be entirely crystalline, and fairly equigranular, with grains usually ranging from 1 to 3 mm. in diameter. The grain boundaries may be simple or complicated, but there is always satisfactory interlocking of the grains.

Boomerang Creek Granitic Gneiss

Hand specimens are dark grey, granular, and medium grained. A few specimens show poorly developed layering or banding. Grey, glassy quartz, semi-opaque whitish-grey felspar, black biotite, and silvery muscovite can be readily recognized. The felspars often occur as subhedral to euhedral porphyroblasts. Dull, greenish-grey almost rectangular grains of altered cordierite can usually also be seen. Most specimens have a dull appearance, apparently due to lack of contrast between the mineral grains and also to the presence of appreciable amounts of altered cordierite. In some specimens the altered cordierite individuals are associated with biotite and muscovite, forming aggregates which give the rock a blotchy appearance.

Microscopic examination shows the gneiss to be entirely crystalline, and equigranular to somewhat porphyritic. The grain boundaries are in most cases relatively simple, but a few are intricate and complex.

Porphyry

In hand specimen the porphyry is a dark brownish grey porphyritic rock. Lath-shaped phenocrysts 1 to 5 mm. long of white felspar, and greenish-black aggregates of ferromagnesian minerals are set in a microcrystalline ground mass. Quartz phenocrysts occur in some of the wider dykes elsewhere in the project area, but have not been observed in the dykes which pass through the power station. A preferred orientation of the phenocrysts and ferromagnesian mineral aggregates is often observed.

Under the microscope the porphyry is entirely crystalline, and shows a distinctly prophyritic texture. A distinct preferred orientation of the groundmass minerals is evident, and most of the phenocrysts have their long axis approximately parallel. The felspar phenocrysts are of partly altered plagioclase, and the ferromagnesian mineral aggregates contain biotite, chlorite and epidote. The ground mass consists mainly of intergrown crystals of quartz and felspar, and also contains smaller quantities of biotite, muscovite and epidote.

APPENDIX II

PHYSICAL PROPERTIES OF ROCKS

The following tables show compressive and tensile properties of the two main rock types at Tumut 2, determined from diamond drill cores which were mainly 1-5/8 or 2-1/4 inches in diameter, and from 2 to 3 times their diameters in length. For each group of specimens tested the average result is given, together with the range. Most samples tested for compressive strength broke across the rock fabric, forming a typical "Cone fracture". These are the ones listed under (a). Several specimens which gave anomalous results and were mostly found to have broken along previously undetected joints are listed under (b). The results of the tensile tests in which all specimens appear to have broken along joints, are listed in (c).

(a) Compressive Properties, Samples Broken Across Rock Fabric

Rock Type and number of Samples		Modulus of Elasticity p.s.i. x 10^6		Compressive Strength p.s.i.		Location of Samples and Remarks
		Average	Range	Average	Range	
Happy Valley Granite	1	11.3	-	17,200	-	Stress Site 1, Machine Hall Upstream Wall
	5	8.8	4.4	18,000	5,300	Stress Site 2, Machine Hall Upstream Wall
	4	5.9	2.9	20,200	3,300	Stress Site 3, Machine Hall Upstream Wall
	2	9.1	0.6	17,400	-	Stress Site 2, Exploratory Tunnel, compressive strength of one sample only
	2	8.1	2.4	14,900	-	Stress site 3, Exploratory tunnel; compressive strength of one sample only
	3	5.9	1.6	20,300	800	Stress Site 5, Exploratory Tunnel
	1	7.4	-	24,200	-	Stress Site 9, Exploratory Tunnel
Boomerang Creek Granitic Gneiss	3	10.9	6.9	19,700	10,000	Stress Site 1, Exploratory Tunnel
	1	12.8	-	15,000	-	Stress Site 4, Machine Hall downstream wall
	3	10.0	1.2	23,600	4,100	Stress Site 5, Machine Hall downstream wall
	1	9.8	-	-	-	Stress Site 5, Machine Hall downstream wall, compressive strength not determined
	2	6.3	0.0	21,800	200	Stress Site 6, Machine Hall downstream wall.

(b) Compressive Properties, Samples Mostly Broken Along Joints

Rock Type and number of Samples		Modulus of Elasticity p.s.i. x 10^6		Compressive Strength p.s.i.		Location of Samples and remarks
		Average	Range	Average	Range	
Happy Valley Granite	1	6.1	-	9,000	-	Stress Site 1, Tumut 2 Power Station; Not recorded whether broke along joint or not. Probably broke along joint.
	2	6.8	1.2	4,100	600	Stress Site 7, Exploratory Tunnel; samples broke along joints dipping 55° and 65° to the plane normal to core axis.
Boomerang Creek Granitic Gneiss	1	9.8	-	9,800	-	Stress Site 1, Exploratory Tunnel; sample broke along joint or altered zone near joint.
	1	6.9	-	12,400	-	Stress Site 8, Exploratory Tunnel; sample broke along joint dip 55° to the plane normal to core axis.
	1	-	-	3,300	-	Stress Site 4, Machine Hall downstream wall; probably broke along joint.

(c) Tensile Properties

Rock Type and number of Samples		Modulus of Elasticity p.s.i. x 10^6		Compressive Strength p.s.i.		Location of Samples and Remarks
		Average	Range	Average	Range	
Happy Valley Granite	1	-	-	300	-	Stress Site 3, Exploratory Tunnel. Type of failure not noted.
	2	3.6	1.1	800	300	Stress Site 5, Exploratory Tunnel. One recorded as breaking along joint dip 30° to plane normal to core axis. The other broke to "plane fracture" dip 10°.
	1	7.9	-	1,000	-	Stress Site 13, Exploratory Tunnel. Recorded as breaking along "plane fracture", dip 10°.
Boomerang Creek Granitic Gneiss	1	6.9	-	1,400	-	Stress Site 1. Exploratory Tunnel. Recorded as breaking partly along joint dip 65° to plane normal to core axis.

APPENDIX III

ROCK BOLT SUPPORT AT THE POWER STATION

Experimental work carried out by the Snowy Mountains Authority to determine the functions of rock bolts in hard, jointed rock has been described by T.A. Lang (1957). He also describes their successful use as support in the construction of Tumut 1 Power Station. The laboratory experiments were made by using model or actual rock bolts in a variety of materials. These included a series of model joint blocks cut in plastic, and loose, angular particles of crushed rock. The work indicated that if the ratio of bolt length to bolt spacing was greater than 2, then the effects of individual bolts interacted, and the originally loose materials formed an integral structural member, which was capable of carrying considerable loads, and behaved quasi-elastically. The construction of Tumut 1 Power Station generally confirmed the results of the laboratory work, and indicated the desirability for rock bolting over large rock areas to be in pre-determined patterns.

Based on the Tumut 1 experience, patterns of rock bolts, in which the bolt length/spacing ratio was 2:1 or greater, were used throughout the large excavations at Tumut 2. Slight variations were made in the patterns, and additional bolts were often installed, where the need for such measures was indicated by the local geological situation.

The bolts used in the greater part of the Machine Hall were 0.75-inch diameter medium tensile steel bars, mostly 12 and 14 feet long. These were furnished with expansion shell type anchors, and 6-inch square steel plates for bearing against the rock surface. When installed the bolts were stressed to a nominal load of 12,000 lbs.

Two methods were used to grout in place the downwards sloping bolts in the vertical walls. In the first method the bolt was inserted in the hole, and grout poured in around it. The other more satisfactory method finally adopted was to fill the hole with grout before insertion of the bolt. Upwards sloping bolts to be grouted were fitted with a plastic de-aeration tube which extended from the anchor end and passed through a hole in the bearing plate. The collar of the bolt hole was sealed with quick-setting cement at the time of installation of the bolt. Grout was pumped up a plastic tube which passed through another hole in the bearing plate, and extended several inches above it.

During the later stages of construction, bolts consisting of 1 inch diameter mild steel bars with hollow cores were used in places requiring upwards sloping grouted bolts. The hollow cores served the purpose of the plastic de-aeration tubes, which were no longer used. In the smaller tunnels in the power station area, rock bolts 10 feet and 8 feet long were used.