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# Geological Report on Site Investigation and Construction of Corin Dam, Cotter River, A.C.T.

*by*

*E.J. Best*

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## SUMMARY

Corin Dam is an earth-cored rockfill structure; it is the largest of the three existing storages for Canberra's domestic water supply, having a capacity five times those of the Cotter and Bendora Dams combined. The dam has a maximum height of 245 feet, a crest length of 916 feet, and contains 1.82 million cubic yards of placed fill. The systematic site investigation commenced in 1961 with a preliminary evaluation of the suitability of the site, and during the following four years detailed feasibility and design investigations were carried out. Construction of the dam commenced in April, 1966, and the reservoir started to fill in April, 1968.

The country rock at the damsite consists of folded beds of quartzite, silicified sandstone and siltstone, and laminated siltstone. The regional strike of bedding is north-westerly, and bedding dips are generally steeper than 45°. This sequence is terminated 1,000 feet west of the river by the Cotter Fault, which is a major regional structure; older rocks, dominantly phyllite, crop out to the west of the fault zone. The sequence of rocks present at the damsite also crops out over much of the reservoir area. In addition, extensive areas of the river valley are underlain by quartz porphyry which has been intruded along the Cotter Fault zone.

The folding and jointing at the damsite are consistent with regional compression in a north-east/south-west direction which has produced tight, asymmetrical folds with well-developed joint sets. Six major faults were exposed in the damsite foundations, all with strikes between north and north-west, and westerly dips ranging from 40° to vertical.

Three distinct phases of site investigation - preliminary, feasibility and design - were carried out before construction commenced; each is described briefly to show the sequence of the investigation, the techniques used, and the relationship of the studies to the types of dam considered at each stage of the investigation. The various techniques used in the investigation are then described in some detail to show the basis for the geological evaluation of the site.

During construction of the dam, geological services were provided on a full-time basis, owing to the complex geological conditions revealed during the site investigations. The scope of these geological investigations is described briefly as a preface to the main body of the report, which deals in detail with the engineering geological aspects of the construction of the dam and appurtenant structures.

At the damsite, the distribution of the soil and scree overburden was plotted, using detailed survey spot heights, and related to the bedrock geology. The criteria adopted for the surface treatment of the core and filter zone foundations are discussed and related to the exposed bedrock geology: a plan of the actual foundation treatment reflects the geological structure in some detail.

The side channel spillway for the dam is excavated in the western abutment and is fully concrete-lined. Several design changes were necessary during construction because of adverse geological conditions. The factors influencing the location and design of the spillway structure are described, and a detailed account of the subsequent construction of the spillway is given.

The diversion tunnel, which is 1,308 feet long, was excavated in the eastern abutment. No major difficulties caused by adverse geological conditions were experienced during excavation, although some extra excavation was necessary at the inlet portal. The geology revealed during driving of the tunnel was logged and subsequently correlated with construction factors such as drive rate, amount of explosive used, overbreak and support. It is concluded that much of the tunnel was unnecessarily supported with steel sets.

Rockfill for the dam embankment was quarried from a prominent spur on the eastern side of the Cotter valley less than a mile upstream of the dam. The rockfill was derived from a sequence of quartzite and silicified sandstone, with some interbeds of siltstone, which was preserved within economic quarrying depths from the ground surface by a broad synclinal structure. The overall development of the quarry and the suitability of the material were largely determined by the geological succession and structure; these factors are considered in detail in the report. The sources of core material, filter material and concrete aggregate are also discussed in the section dealing with construction materials.

Only two paths of water leakage from the reservoir are considered at all possible, namely along the Cotter Fault and through the dam foundations (including abutments). Serious leakage along the Cotter Fault is considered unlikely; although the grout curtain was not extended across the fault zone, an additional groundwater level observation hole was drilled to monitor the gradient of the water table in this area. Leakage through the dam foundations was controlled by a comprehensive programme of curtain and blanket grouting.

During the grouting of the dam foundatons, 1,376 holes, totalling 49,529 feet, were drilled and grouted; the total consumption was 30,536 bags of cement. The pattern of grouting and the practical details of the techniques used are described, and the results are briefly summarised. The grout consumption during curtain is analysed in considerable detail, and correlations are established with depth of grouting, closure pattern, rock types and water test results. The results of these analyses are used to assess the effectiveness of curtain grouting. The grout consumption during blanket grouting is also analysed, and the high consumption in the foundations are correlated with geological features.

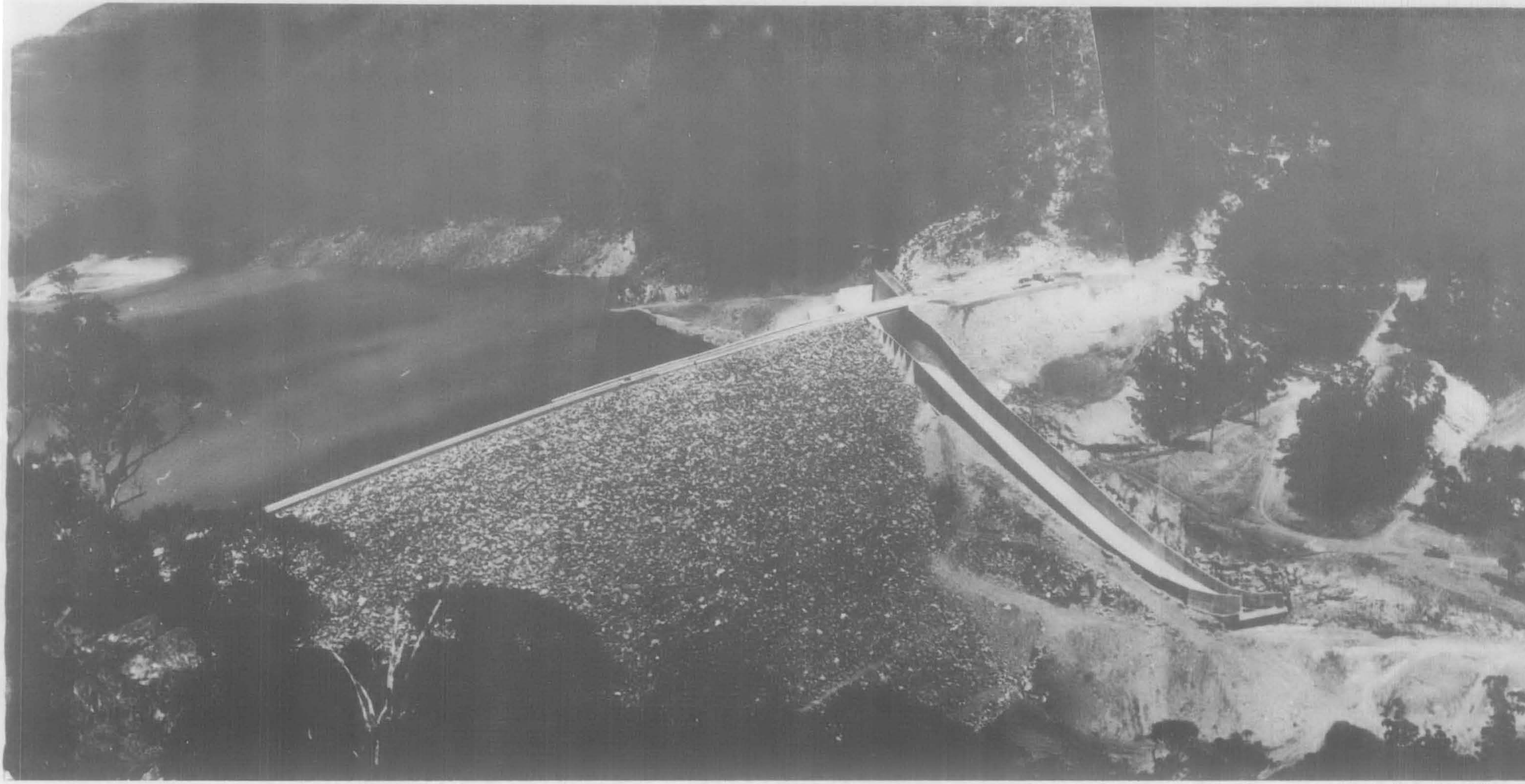

Fig.1 Corin Dam, Cotter River. A.C.T.

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## INTRODUCTION

Corin Dam is the third and largest dam constructed on the Cotter River to provide domestic water for a rapidly-growing Canberra. It is located at the confluence of Kangaroo Creek with the Cotter River; it is seven miles upstream from Bendora Dam and 37 miles by road south-west of Canberra (see Plate 1). The storage capacity of the reservoir behind Corin Dam is 16,600 million gallons, which is five times the combined capacity of the Bendora and Cotter reservoirs; the three storage areas together are adequate for a population of at least 200,000.

The dam is an earth-cored rockfill structure with a maximum height of 245 feet, a crest length of 916 feet, and a base width of 920 feet; it contains 1.82 million cubic yards of placed fill, of which 1.36 million cubic yards is rockfill. During construction of the dam, the Cotter River was diverted through a 1,308-foot long tunnel excavated in the east bank. The tunnel is fully concrete-lined, and has an internal diameter of 12½ feet; it also serves as the outlet for water, drawn from the reservoir by means of a valve tower and concrete-lined shaft. The spillway for the dam is a side-channel concrete structure, with a crest length of 210 feet, excavated in the west abutment. The overflow basin leads to a reinforced-concrete chute 40 feet wide, at the downstream end of which is a concrete ski-jump structure. The capacity of the spillway is 40,000 cubic feet of water per second.

The site for Corin Dam is one of six damsites on the Cotter River, selected in 1908 by surveyors during reconnaissance mapping of the water resources of the Australian Capital Territory. The most suitable of these sites was selected for the construction of the Cotter Dam, a concrete gravity structure 86 feet high, which provided Canberra's water supply from 1915 to 1961. Between 1945 and 1958, site investigations were carried out at the five remaining sites, designated A, B, C, D and E; sites A, B, C and D are located on a three-mile stretch of the river, while site E is a further seven miles upstream. Reconnaissance geological surveys were made at each site in 1945 (see Noakes, 1946a and b), and sites B, C and A were selected for more detailed investigation. Site C was finally selected for the construction of a double-curvature, thin, concrete arch dam 155 feet high; this was completed in 1961 and named Bendora Dam. During construction, however, the population of Canberra began increasing at an unprecedented rate (12% per annum), and it became evident that further water storage would be necessary by 1969. Preliminary geological investigations were therefore set under way in 1961 at Damsite E on the Cotter River, and at Googong Damsite on the Queanbeyan River (sites A, B and D were impracticable owing to the construction of Bendora Dam). Late in 1963, Damsite E was selected as the more suitable site, and a feasibility investigation to determine the type of dam most suited to the site commenced in January, 1964. This investigation continued until November 1964, when a decision was made to design and construct an earth-cored rockfill dam at the site. A design investigation was carried out during the following nine months to test specific aspects of the design layout, and to locate sufficient quantities of construction materials as close to the site as possible. All investigations were completed by August 1965, and details of design were finalised soon afterwards.

The contract for construction of the dam was let to Thiess Bros. Pty.Ltd. in March 1966, and work on excavating the tunnel portals commenced in April. Excavation of the tunnel, which was completed on 21st July 1966, was followed by concrete lining, and the Cotter River was diverted through the tunnel on 12th November 1966. The partially-constructed cofferdam was then extended across the valley, while the valley floor core zone foundations were cleaned off, grouted, and prepared for core placement. Construction of the core zone commenced in February 1967, and placement of the earth, filter and rockfill zones continued throughout the remainder of the year. Owing to the unusually dry winter, it was possible to place core material almost continuously throughout the year, with only a few brief interruptions caused by bad weather. Placement of the dam was completed in January, 1968, construction of the spillway was completed in March 1968, and the inlet end of the diversion tunnel was plugged in April 1968 to start filling the reservoir (5 months ahead of schedule).

This report deals mainly with the geological features and problems encountered during the excavation of foundations for the dam and associated structures; the curtain and consolidation grouting programmes are also considered in some detail. The techniques used and the results obtained during the site investigation are also included in this report so as to give a comprehensive picture of the site development. Much of the engineering geological information obtained at the site is given in a series of detailed plans and sections, which form a concise record of the foundation conditions exposed immediately before dam and concrete placement. This "as constructed" information may be useful in future years during maintenance operations, or if special problems arise.

All phases of the site investigation were directed by the Commonwealth Department of Works on behalf of the National Capital Development Commission. The Bureau of Mineral Resources provided all geological services during the investigation, and was responsible for the planning and supervision of the drilling programme; the Bureau also conducted most of the geophysical surveys at the site and carried out associated laboratory investigations. The Department of Works designed the dam, and was responsible for the supervision of all aspects of construction. A geologist was assigned by the Bureau of Mineral Resources, on a full-time basis, to provide engineering geological services throughout construction of the dam; he was on-site practically the whole time during the main constructional phase (May, 1966 to August, 1967).

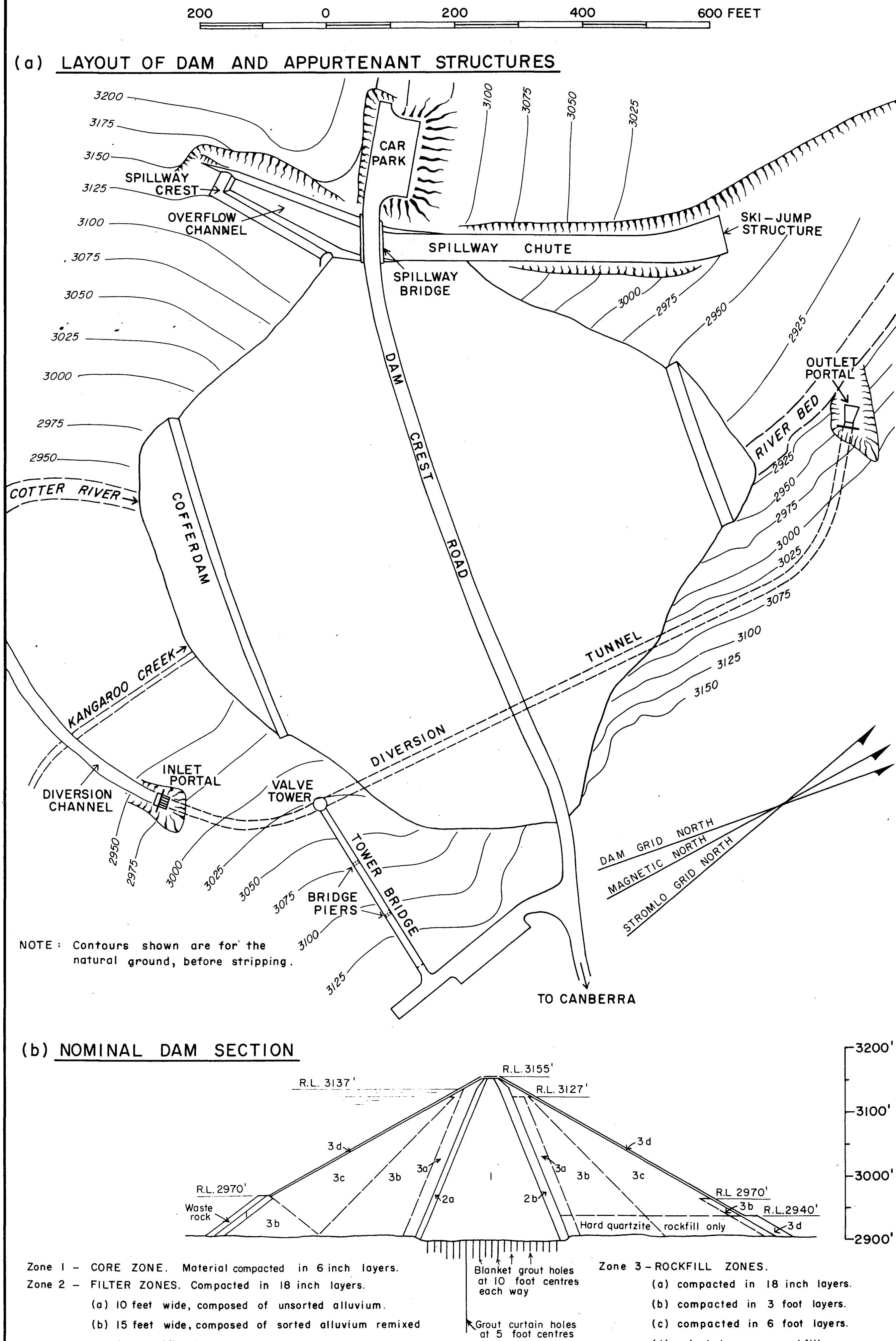
## PHYSIOGRAPHY

The Cotter River flows in a northerly direction through the mountainous country occupying the western half of the Australian Capital Territory. Its course and valley configuration has been determined mainly by the trend of the Cotter Fault, and the northerly strike of the folded country rock over which the river flows for much of its length.

The topography of the Cotter Valley changes quite markedly in the vicinity of Kangaroo Creek. Downstream from this tributary, the river flows in a narrow, youthful valley with overlapping spurs and steep river sides, whereas upstream from Kangaroo Creek, the valley floor is considerably wider and contains extensive alluvial flats (see Figs 4 and 5). This change in topography is explained on the basis of stratigraphy and structure. Upstream from Kangaroo Creek, extensive bodies of quartz

# CORIN DAM - GENERAL ARRANGEMENT

Fig. 2




*To accompany Record 1969/111*

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Fig. 3 :- Photograph of the dam during construction showing the zoning of construction materials - see Fig. 2 for descriptions of the various zones. The top of the core zone is at R.L. 3068 feet.
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porphyry and granite, exposed in the lower slopes of the valley, are intruded and faulted along the Cotter Fault and associated faults. The lower resistance to weathering and erosion of the granitic rocks has resulted in the development of a comparatively broad and straight valley floor for several miles upstream of Corin damsite. The valley sides are as steep as those downstream from the damsite, however, as the granitic rocks do not extend far up the valley sides. Towards the head of the catchment area, granite becomes the dominant rock type and the valley is very broad with gentle slopes (see Fig. 5).

These physiographical features have had a considerable bearing on the location and construction of Corin Dam. The original site selection was determined by a narrow constriction in the valley, coupled with a broadening of the valley floor immediately upstream which would provide a good reservoir volume. The development of the wide valley floor resulted in the accumulation of extensive alluvial flats close to the damsite; these have been utilised during construction to provide all the material for the filter zones in the dam. Also, the high degree of weathering in the granitic rocks exposed in the reservoir area has resulted in suitable impermeable core material being present close to the damsite.

## REGIONAL GEOLOGY

In the Corin Dam area, the bedrock exposed on the lower slopes of the Cotter valley consists of metasediments, while the upper slopes of both the eastern and western divides are composed dominantly of intrusive granitic rocks (the Gingera Granite to the west and the Murrumbidgee batholith to the east). Two distinct sequences, separated by the Cotter Fault, are recognizable in the metasediments. The oldest sequence (probably Upper Ordovician) is restricted to the western side of the Cotter Fault, and consists of sericite phyllite with interbeds of sheared quartzite; these beds have a northerly strike and an average dip of $60^{\circ}$ to the west. On the eastern (downthrown) side of the Cotter Fault, the rock types range from sandstone and quartzite to finely banded shale and slate; siltstone is an important component. Grade of metamorphism is low. Close to the damsite, the succession has been deformed into a series of north-west trending, plunging folds, and silicification is evident to varying degrees. The relationship of these younger rocks to those on the western side of the Cotter Fault has not been determined.

Several miles downstream from the damsite, the interbedded sandstone, siltstone and shale on the eastern side of the Cotter Fault is unconformably overlain by the Tidbinbilla Quartzite, believed to be of Silurian age. Several prominent quartzite outcrops are present in the Corin Dam area which were originally thought to be outliers of Tidbinbilla Quartzite. However, none of these outcrops have been proved to be of Tidbinbilla Quartzite; in fact, some are definitely part of the older folded sequence, and evidence from the others suggests that they also are part of the older sequence.

The Cotter Fault is the dominant structural feature of the area, and can be traced for more than 40 miles along the Cotter valley. It is considered to be a high-angle reverse fault, and the main movement appears to have occurred at about the time of the granite intrusion in late Silurian time. The throw of the fault has not been determined, but the fact that no Tidbinbilla Quartzite has yet been found on the western side of the fault suggests a large downthrow to the east.

Upstream of the damsite, granitic rocks also crop out in the floor of the valley. The main rock type is a quartz-feldspar porphyry, which is generally deeply-weathered. Every observed contact between porphyry and metasediments is faulted along north-north-west trending fault planes, and these faults are in turn truncated by the main Cotter Fault; the intrusion of the porphyry and the faulting which has brought it against the interbedded sandstones and siltstones must therefore have preceded the main movement on the Cotter Fault. It is the weathering and erosion of these porphyry bodies which has resulted in widening of the river valley in the reservoir area.

## GEOLOGY OF RESERVOIR AREA

The geology of the Corin reservoir area is shown on Plate 3. The surface trace of the Cotter Fault is entirely on the western side of the river, and is below top water level for a total of about $1\frac{1}{2}$ miles; therefore only a small part of the reservoir area is underlain by interbedded phyllite and quartzite.

For two miles upstream from the dam, the dominant rock type in the reservoir area is quartz porphyry. Outcrops are restricted to low-lying, scattered blocks and boulders, and the porphyry is generally deeply weathered. On the eastern side of the valley, the faulted contacts with the quartzite-siltstone sequence are generally below top water level, and on the western side of the valley the porphyry crops out only on the eastern side of the Cotter Fault; the porphyry is therefore restricted to a narrow zone up to 1,500 feet wide along the valley floor.

From 2 miles upstream of the dam to the head of the reservoir, quartzite and cleaved siltstone are the dominant rock types. Wide variations in the attitudes of bedding indicate considerable deformation of the succession, and the erratic course of the river suggests structural control - this contrasts strongly with the almost straight section of river where the porphyry is predominant. The Kangaroo Creek portion of the reservoir is also underlain by a folded succession of silicified sandstone and siltstone.

After filling of the reservoir, minor landslips will probably occur around the reservoir rim, particularly along the western side for one mile upstream of the dam (where the phyllite and the Cotter Fault zone crops out in the reservoir). In this area, the ground surface is steep, the bedrock consists of phyllite which has been loosened by gravity creep, and the overburden contains much loosely-compacted phyllite scree. There is no evidence to suggest that landslides involving displacement of large masses of bedrock will occur.


Fig. 4 :- Aerial view of the middle reaches of the Cotter valley, showing the change to a youthful river valley with overlapping spurs immediately downstream of Corin Dam. The Tidbinbilla Range is of probable Lower Silurian sediments which dip gently to the west - note the dip slope towards the Cotter River from the highest peak (Mt Tidbinbilla).

News and Information Bureau photograph NDC 1113/22, taken in June, 1968.

SEISMICITY

Earth tremors have been recorded in the Canberra area, but there is no evidence to suggest that any have originated in the Cotter Valley. The only published data on seismic activity in and around the Australian Capital Territory are contained in Cleary, Doyle and Moye (1964). Between 1958 and 1962, 44 tremors were recorded, of which the largest were shocks of Richter magnitude 5 (epicentre 12 miles north of Jindabyne) and magnitude 4 (epicentre 14 miles south-east of Cooma). The nearest recorded epicentre to Corin Dam was about 27 miles south of the site, and about 9 miles east of the Cotter Fault; the shock had a Richter magnitude of 2. No seismic disturbances are known to have originated from the Cotter Fault.

No special aseismic design factors were incorporated in the design of the dam and spillway structures, as it was considered that the normal design factor of safety would be adequate for any earth tremors likely in the area. However an aseismic design factor was incorporated in the valve tower structure.

## DAMSITE GEOLOGY

The country rock at the damsite consists of folded beds of quartz sandstone, quartzite and laminated siltstone, with a north-westerly strike and dips mostly steeper than 45° (see Plate 4). This succession is terminated by the west-dipping Cotter Fault, 1,000 feet west of the river; the strata mapped at the damsite have been downfaulted by the Cotter Fault and brought into contact with the older, interbedded phyllite and quartzite to the west of the fault.

LITHOLOGY

The dominant rock type at the damsite is silicified quartz sandstone, beds of which occur throughout the geological succession. The sandstone beds range in grain size from very fine to coarse,* and the shape of the grains ranges from well-rounded to sub-angular. The sandstone consists mainly of quartz grains, with scattered grains of siltstone and shale. In hand specimen, the matrix of the sandstone appears to be of argillaceous material, but microscopic investigation has shown this to be the result of impaction of shale fragments and grains by the quartz grains. The amount of shaly material in the sandstones varies from insignificant amounts to 25% of the rock. In some sandstone beds, angular fragments of grey-green shale, up to 5 cm long, are commonly aligned sub-parallel to the bedding. In one part of the stratigraphical succession, the proportion of angular shale fragments is sufficiently high for the rock to be classified as a sedimentary breccia; generally, however, the shale fragments are scattered sparsely throughout the sandstone beds in which they occur.

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*For definitions of grain size, see Appendix 1.

Silicification is evident to varying degrees in most of the sandstone beds; some beds are so highly silicified that they are true quartzites in which the individual grains are not visible in a hand specimen.

Most beds of quartz sandstone are thick (6 inches to 20 feet) and uniform, the bedding planes being clean or thinly covered with silty material. Where sandstone beds occur interbedded with laminated siltstone, they are seldom more than 3 feet thick. Sandstone beds more than 3 feet thick are generally part of a sandstone sequence which contains a very few, thin interbeds of siltstone.

In the laminated siltstone, the laminae are from paper thin to 5 mm. thick, and show abundant cross bedding, graded bedding and slumping features; they are also highly contorted in areas of faulting and tight folding. In many places where it occurs as thin interbeds between thick beds of quartz sandstone, the laminated siltstone is cleaved and sheared. In areas where the interbeds of siltstone and sandstone have a similar range of thickness, however, shearing (if present) is generally restricted to a narrow zone (less than one inch) at the sandstone-siltstone contact.

A particularly characteristic sandstone sequence crops out extensively on the west bank of the damsite; it consists of well-bedded, silicified, fine to medium grained quartz sandstone, with many interbeds of sandstone that contain numerous irregular silty laminae. The silty laminae, which are dark grey and have a wavy appearance, are rhythmically interbedded with very thin beds of sandstone. The beds of uniform sandstone range in thickness from 4 inches to 30 inches, while the interbeds of sandstone containing silty laminae are more thinly bedded (2 inches to 30 inches thick).

Pyrite occurs scattered through rock of all types exposed at the damsite. It has four modes of occurrence: as single cubes, up to 2 mm. across, disseminated through the rock; as clusters, up to 10 mm. across, of pyrite crystals; as pods of disseminated pyrite up to 30 mm long by 10 mm. wide; and as irregular areas of disseminated pyrite associated with pods. The clusters of pyrite are restricted to sandstone beds, and microscopic examination reveals thin sinuous lines of pyrite running between grain contacts and linking the clusters. The pods of pyrite, however, occur mostly in the siltstone and shale.

Both sandstone and siltstone beds show a mild degree of metamorphism, probably due to folding and faulting. Microscopic examination shows that the quartz grains in the sandstones are strained, and muscovite in the siltstone beds exhibits "pressure shadows".

## STRUCTURE

### Folding

The foundation rocks at the damsite are tightly folded, the main folds consisting of two anticlines, one on each bank (see Fig.6) and one syncline on the west bank. Joint and bedding plane stereograms from exposures close to the dam axis show that the folds are asymmetrical, with axial planes dipping 75° to the north-east; they also show that the fold

Fig. 5 :- Aerial view of the upper Cotter valley showing Corin Dam and reservoir area 3 months after plugging of the diversion tunnel. Note the subdued topography and gentle slopes in the granite at the head of the catchment area.

News and Information Bureau photograph NDC 1113/19, taken in June, 1968.

axes plunge at 20° to the south-east (Best & Hill, 1962; Best, 1965). Detailed mapping during the feasibility investigation and during construction has revealed much minor folding on the limbs of the main folds, particularly on the west bank where several sets of tight minor folds are evident in the quartzite beds which contain silty laminae (see Plate 7). These folds are very tight, the crests and troughs being generally less than two feet wide. The quartzite beds are not significantly broken or jointed along the fold area, and individual quartzite beds could be clearly traced across the folds; it is inferred that much of the folding occurred while the rocks were still plastic.

The pattern of folding on the east bank has been obscured by displacements along major faults. It was deduced from the geological site investigation that the anticlinal axes on both sides of the 40° fault (see Plate 4) were the same fold, and that reversals of dip farther uphill were caused by a syncline. However, the attitude of bedding in the tunnel near the dam axis suggests that the dip reversal is due to displacement along the 60° fault exposed near top water level (Fault C in Plates 4 and 5). Mapping of the scattered bedrock exposures in the downstream rockfill zone has not revealed the northern extension of the anticlinal axis; this could be explained by assuming that the trace of the 60° fault continues to the north-west and displaces the fold axis.

On the upstream (hanging wall) side of the 40° fault (Fault A in Plates 4 and 5), the anticlinal axis can be correlated with a fold mapped in the foundations near the upstream face of the dam at R.L. 3040 feet. However, the fold axis of the latter plunges gently in the opposite direction to that of folds mapped in the core zone foundations. Bedding attitudes indicating north-west plunging fold axes have also been recorded on the west bank foundations, and at both the inlet and outlet portals. The area has therefore been subjected to gentle cross folding, which is superimposed on the tight folding previously described; this has been confirmed by observed cross folds in the well-bedded quartzite with silty laminae in the western foundations.

## Faulting

The foundation rock at the damsite has been highly stressed, and numerous displacements of adjacent blocks of rock have been noted. In the quartzite and sandstone, fault movement has taken place along joints and bedding planes with the consequent development of innumerable striated and slickensided surfaces. Where siltstone interbeds are present in the sandstone and quartzite, local stresses were relieved by shearing of the less competent silty interbeds.

Detailed mapping of the core zone foundations has revealed six persistent major faults. Four of the faults were located during the site investigation, and the presence of a suspected fifth fault, in the valley floor, was confirmed during construction. The sixth fault, which had not previously been located, was also exposed in the valley floor; although it has a considerable displacement, it has little surface expression and was of no engineering significance. Many other faults requiring treatment are present in the dam and spillway foundations, but the six faults referred to above are the most significant from the geological viewpoint; these faults are labelled A, B, C, D, E and F in Plates 4 and 5, and are described below in some detail.

Fault A is a zone of crushed rock, with much clay, which crops out in the eastern core zone foundations and dips obliquely upstream at 40° (see Fig.7); it was the most significant structural defect in the dam foundations and required extensive surface treatment. The fault zone ranges in thickness between 6 and 48 inches, and rock in the hanging wall of the fault is extensively sheared and weathered for several feet adjacent to the fault zone. The displacement along the fault is not known; if the anticlinal axes mapped on both sides of the fault are the same fold, the displacement would not be great.

Towards the valley floor, fault A terminates against a steeply-dipping, north-trending fault zone which is marked by a zone of highly cleaved, laminated siltstone, with some decomposition to clay; this is fault B, which dips at 70°-75° to the west and has a strike of 170° M (see Figs 7 and 9). The wedge of rock at the intersection of the two faults, i.e. on the hanging wall of fault A and the footwall of fault B, is so highly contorted and fractured that the sequence of interbedded sandstone and siltstone is unmappable, even at a scale of 10 feet to 1 inch. The fault zone (fault B) generally ranges in thickness between 4 and 10 feet. However, towards the downstream limit of the core zone, the fault zone gives way to a series of clay seams in jointed sandstone, with no apparent dominant plane of displacement (foreground of Fig.9a). The fault was penetrated at depth by drillholes D.D.7 and D.D.3. Hole D.D.3 shows considerable shearing in the footwall of the fault at a depth of 100 feet below the river bed. The amount and direction of movement on fault B is not known; however, the displacement must have been considerable, as it is not possible to correlate any of the strata to the west of the fault with the geological succession on the eastern side of the fault.

Fault C crops out in the core zone foundations high on the eastern bank (see Figs 7 and 8); it consists of up to 12 inches of finely crushed sandstone, which has weathered out to form a distinct notch in the bedrock surface. The footwall of the fault is silicified and contains many quartz veins and stringers. The fault dips at 60° to the south-west, and a projection of the fault plane down to the level of the diversion tunnel correlates closely with a fault of similar attitude mapped in the tunnel walls. Fault C displaces the 40° fault (A), but it has not been traced downstream in the rockfill zone foundations.

Fault D crops out on the west bank as a near-vertical zone of very crushed quartzite 24 inches wide (see Fig.10). Downstream from the dam axis, the fault bifurcates and continues along strike as two zones of crushed quartzite 9 to 12 inches thick; both branches of the fault were mapped in the spillway chute excavation. Upstream from the core zone, the main fault plane becomes less distinct and branches out as a series of thin seams of crushed rock. It was deduced from the site investigation data that the displacement along this fault is at least 250 feet, with downthrow to the west. However, mapping of the spillway ski-jump excavation has proved conclusively that the thickly-bedded quartzite in drillhole D.D.17 is the same sequence as the prominent quartzite rib exposed on the west bank, near the upstream face of the dam and in the spillway channel excavation. It is now evident that the total vertical displacement on fault D is about 400 feet with downthrow to the east; this is made up of about 200 feet displacement on each of the north-trending arms of the fault exposed in the spillway excavation.

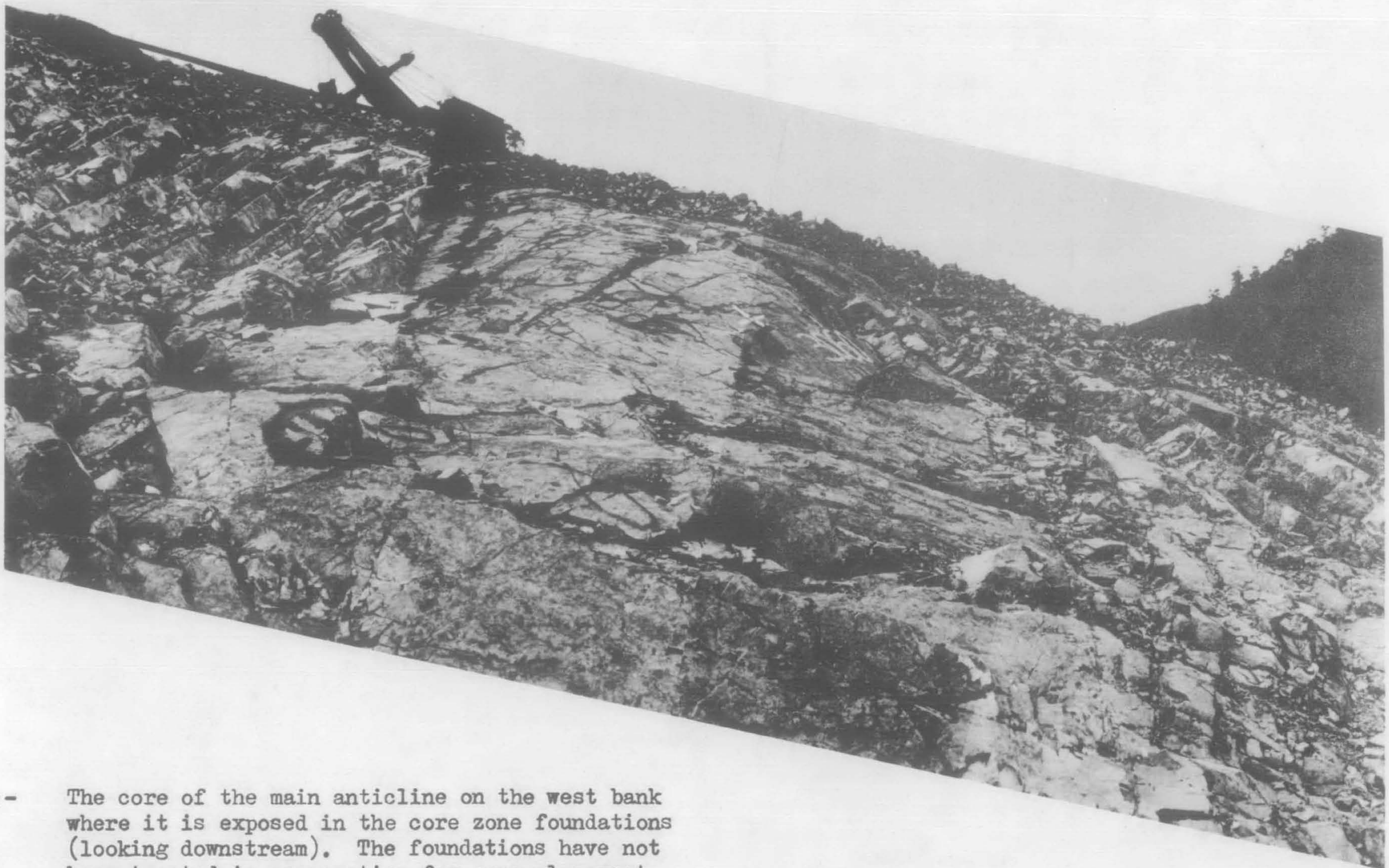

Fig. 6 :- The core of the main anticline on the west bank where it is exposed in the core zone foundations (looking downstream). The foundations have not been treated in preparation for core placement. The white cross on the rock marks the point 220 E, 1020 N on the dam grid.
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In the core zone foundation, contorted laminated siltstone crops out immediately downhill of fault D. The wedge of siltstone widens to the south, and contains a large isolated block of thickly-bedded quartzite (see Plate 4). Probably the contact of the siltstone with the regularly interbedded sequence immediately downhill is faulted, and the contorted siltstone and isolated block of quartzite are part of the fault zone associated with displacement on fault D.

Fault E is the actual trace of the displacement which was predicted as a result of the site investigation; it is, in fact, 30 feet west of its predicted position. The fault is near-vertical, and consists of a zone of crushed rock, ranging in width from 2 to 12 inches, which contains a few impersistent clay seams. Although the fault zone is rather narrow (see Fig.12), the displacement along it must be considerable, as the rock sequences on either side of the fault are quite dis-similar. To the east, a sequence of thickly-bedded silicified sandstone is overlain by interbedded siltstone and sandstone, and underlain by sandstone with interbeds of siltstone and sandstone with silty laminae; this suggests that the thickly bedded sandstone, which is about 90 feet thick, is the same sequence as the quartzite exposed at the upstream and downstream ends of the spillway excavation. If this is so, the vertical displacement on fault E is about 150 feet, with downthrow to the west.

The fault could not be traced across the upstream rockfill zone foundations, as siltstone is predominant on both sides of the fault in this area. However, the major fault exposed at the inlet portal is directly in line with the upstream extension of fault E. Downstream from the core zone, faulting was evident in part of the rockfill zone foundations and in the extreme downstream end of the spillway excavation; these faults are also in line with the trace of fault E. It appears likely, therefore, that fault E extends across the entire site.

Fault F contains very little crushed rock, and in many places along its surface trace it has the appearance of a major joint. However, the dis-similar rock sequence on either side of the fault again indicate considerable displacement. Fresh, well-bedded quartzite with silty laminae is exposed between fault F and fault B, and is overlain by thickly bedded quartzite with siltstone interbeds; the succession is therefore downthrown to the west. To the north, the fault trace is displaced an unknown amount by fault B.

All of the major faults at the damsite trend north to north-west and generally have steep dips; they are therefore part of the system of faults, mapped by Oldershaw during his investigation of the Cotter Fault zone, which are truncated by the Cotter Fault (Oldershaw, 1965). The interpreted displacements are shown in the interpretive cross section, Plate 18. The foundation treatment of the fault zones is discussed under "Engineering Geology".

## Jointing

In the sandstone and quartzite beds, jointing is invariably moderately to well developed. Most joints are straight or only slightly curved, and the more prominent joints persist for some distance at the surface, generally at least 25 feet. Striations and slickensides are well developed on many of the joint faces, and iron-staining of the joint surfaces is very common.

Joints are particularly well-developed in the well-bedded quartzite containing silty laminae. Bedding plane joints are dominant and persistent, and are generally spaced less than 12 inches apart near the bedrock surface. Other sets of joints are less regularly developed and few of the joints in them are very persistent; however, the joints are as closely-spaced as the bedding plane joints.

In the laminated siltstone, bedding plane joints form the dominant joint set. Where siltstone occurs interbedded with sandstone, the joints are closely spaced (less than 5 inches). In thick sequences of laminated siltstone, however, the joint spacing ranges from 5 inches to 3 feet, except where cleavage is well-developed.

In the sandstone and quartzite, dominant joints are open (see Fig.14) and iron-stained to considerable depths (250 feet in drillhole D.D.5). Joint places in the siltstone, however, are generally tight and clean, except in areas of major faulting.

During the preliminary investigation, joint pattern stereograms were constructed from joint measurements at prominent outcrops; these showed that a definite joint pattern exists which is consistent with the compressive forces which caused tight folding. A set of joints parallel to the Cotter Fault was also recorded from outcrops high on the western foundations. The stereograms were also used to predict that no adversely-dipping sets of joints would be present in the western wall of the spillway channel excavation (which would be up to 80 feet deep). Although this prediction was substantiated during construction, one of the joint sets identified in the stereogram analysis was much more persistent than anticipated, and subsequently caused overbreak and instability in the much shallower eastern wall of the excavation (for details and remedial treatment, see "Engineering Geology - Spillway").

## STRATIGRAPHY

During the feasibility investigation, the stratigraphical sequences mapped in exposures at the damsite were plotted and augmented by information from the diamond drill core logs; interpretive correlations were then made to produce a composite stratigraphic column (Fig. 3 of Best, 1965). Even with the considerable additional information obtained during construction, it has still not been possible to construct a reliable composite stratigraphical column, owing to the considerable faulting and folding revealed at the site. The stratigraphy of the west bank and valley floor has been determined with some degree of certainty, and it is now evident that some of the correlations in Best (1965) are incorrect. The considerable displacement along fault B has precluded the correlation of any beds on the east bank with those of the west bank, and the paucity of continuous bedrock exposures on the east bank has caused a lack of detailed knowledge of the stratigraphy and geological structure to the east of the river. Figure 13 summarises what is known of the damsite stratigraphy, and Plates 17 and 18 show the interpreted structure and stratigraphy of the site.

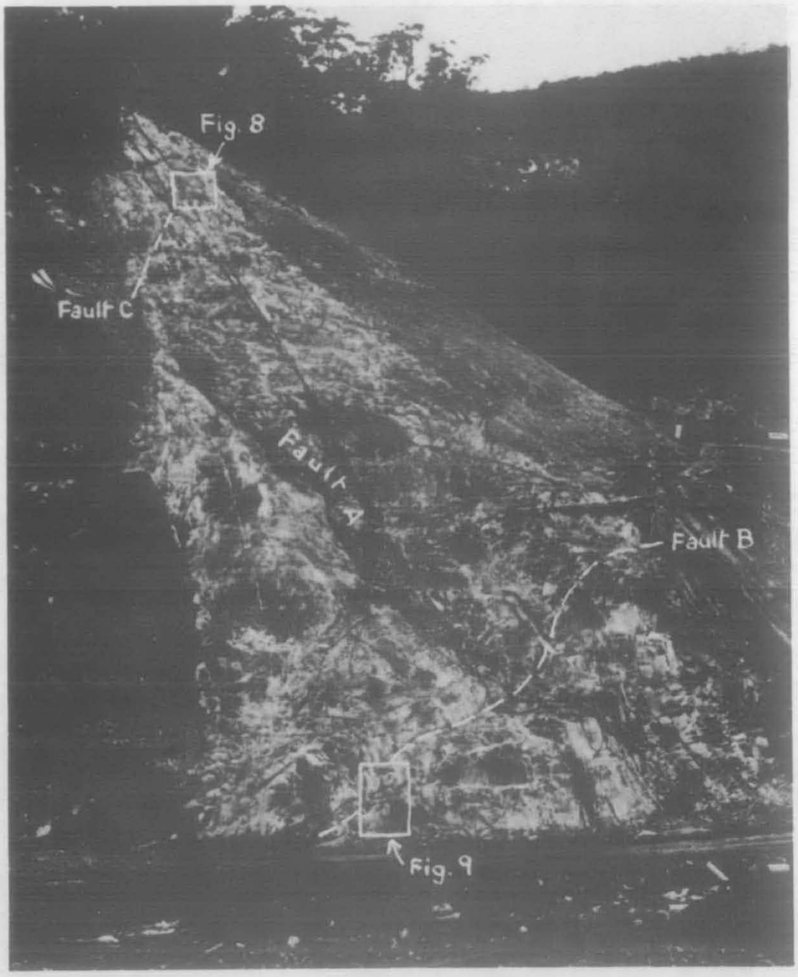


**Fig. 7 :-** The eastern core zone foundations, after removal of overburden by sluicing, showing the prominent surface trace of Fault A, and the positions of Faults B and C.

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**Fig. 8 :-** Fault C exposed in the eastern core zone foundations. The fault is bounded on both sides by thickly-bedded sandstone. The location of this photograph is shown in **Fig. 7**
M 437/30-35 17/8/66

The key to the re-interpretation of the stratigraphy of the west bank and valley floor was the exposure of well-bedded quartzite with silty laminae at the downstream end of the spillway excavation. This immediately suggested that the thickly-bedded quartzite exposed in the spillway downstream of fault D is the same sequence as that exposed in the spillway channel excavation, and not a separate, older sequence as was suggested in Best (1965). Detailed mapping of the succession at the lower boundary of the thickly-bedded quartzite in the two exposures along the spillway line confirmed that the sequences are the same, even though the sedimentary breccia is not well-developed in the downstream exposure. Using this correlation, it was evident that the geological succession downhill of fault D is a repetition of the succession overlying the well-bedded quartzite with silty laminae; the downthrow on fault D is therefore to the east, and not to the west as was suggested in Best (1965).

The thickly-bedded quartzite in the valley floor core zone foundations (i.e. between faults E and F) is now thought to be another repetition of the sequence exposed in the spillway channel excavation. Unfortunately the exposure at the downstream end of the spillway, which led to the re-evaluation of the stratigraphy, was not excavated until several weeks after core had been placed on the valley floor foundations; therefore it was not possible to make a direct comparison of the valley floor quartzite with either of the exposures in the spillway. However, the detailed core zone foundation mapping shows that the sequence is about 90 feet thick, and that it is overlain by interbedded sandstone and siltstone. Although a bed of sedimentary breccia was not noted at the bottom of the sequence, it could well be present - the sedimentary breccia at the downstream end of the spillway was not located until a specific search was made for it.

The rock sequence exposed in the core zone foundations between faults E and B is also part of the stratigraphic succession exposed on the west bank, although the true succession appears to have been obscured by subsidiary faulting. Downstream from the grout curtain line, fresh quartzite with silty laminae is exposed; this is terminated a few feet upstream of the curtain line by a shear zone and a bed of highly cleaved siltstone. Farther upstream, a sequence of thickly bedded quartzite with interbeds of siltstone crops out, while adjacent to fault B is a large faulted block of thickly-bedded quartzite, which abuts against the interbedded quartzite and siltstone (see Plate 7).

To the east of fault B, no reliable stratigraphic correlations are possible between the rock sequences mapped in various areas. In fact, some of the sequences represented in Fig.13 may be incomplete or incorrect due to displacements along unidentified faults; two possible faults are indicated in Fig.13, along with two definite faults intersected in D.D.11. The direction of displacement along fault B is unknown, so it is not possible to determine whether the known stratigraphy of the west bank overlies or underlies the succession exposed on the east bank. Mapping upstream of the dam on the west bank indicates that the laminated siltstone at the top of the succession is at least 400 feet thick. Further, the quartzite bluff which crops out 200 feet west of Diadem Trigonometrical station at R.L.3400 feet (see Plate 2) is part of the succession of quartzite with silty laminae which crops out extensively on the west bank of the damsite. It therefore appears likely that the west bank succession is younger than the rock sequences exposed on the east bank; i.e., fault B is a normal fault with downthrow to the west.

WEATHERING

There is no clearly-defined, uniform weathered zone at the damsite, as the degree of weathering depends upon the abundance of open joints and broken zones. In the absence of close jointing, the sandstone and quartzite are fresh or only slightly weathered at the surface. Where they are cut by persistent open joints however, a weathered zone up to 12 inches wide, is developed adjacent to the joint faces. If the joint spacing is close enough, the entire rock is weathered to a considerable depth. Weathering along joints has been noted at depth in all drill holes at the damsite; the deep weathering is probably related to the low water table at the site.

The laminated siltstone is also generally fresh, or almost so, at the surface, even in areas where it is cleaved. The joints are much tighter than in the sandstone, and so weathering along joint planes is seldom extensively developed. Where the siltstone has been sheared, however, weathering is pronounced and commonly has resulted in the development of clay; clay is particularly common in thin interbeds of siltstone bounded by competent sandstone beds. Sheared siltstone is generally weathered well below ground surface, e.g., the sheared siltstone in fault zone B, where intersected by D.D.3 at a vertical depth of 80 feet below ground surface, is very weathered, although nearby rock is fresh.

**Fig.** 9 (a) :- Fault B (dipping 70° to the west) as exposed in the eastern core zone foundations after removal of overburden by sluicing. This view is taken from the downstream boundary of the core zone (see **Fig.** 7).
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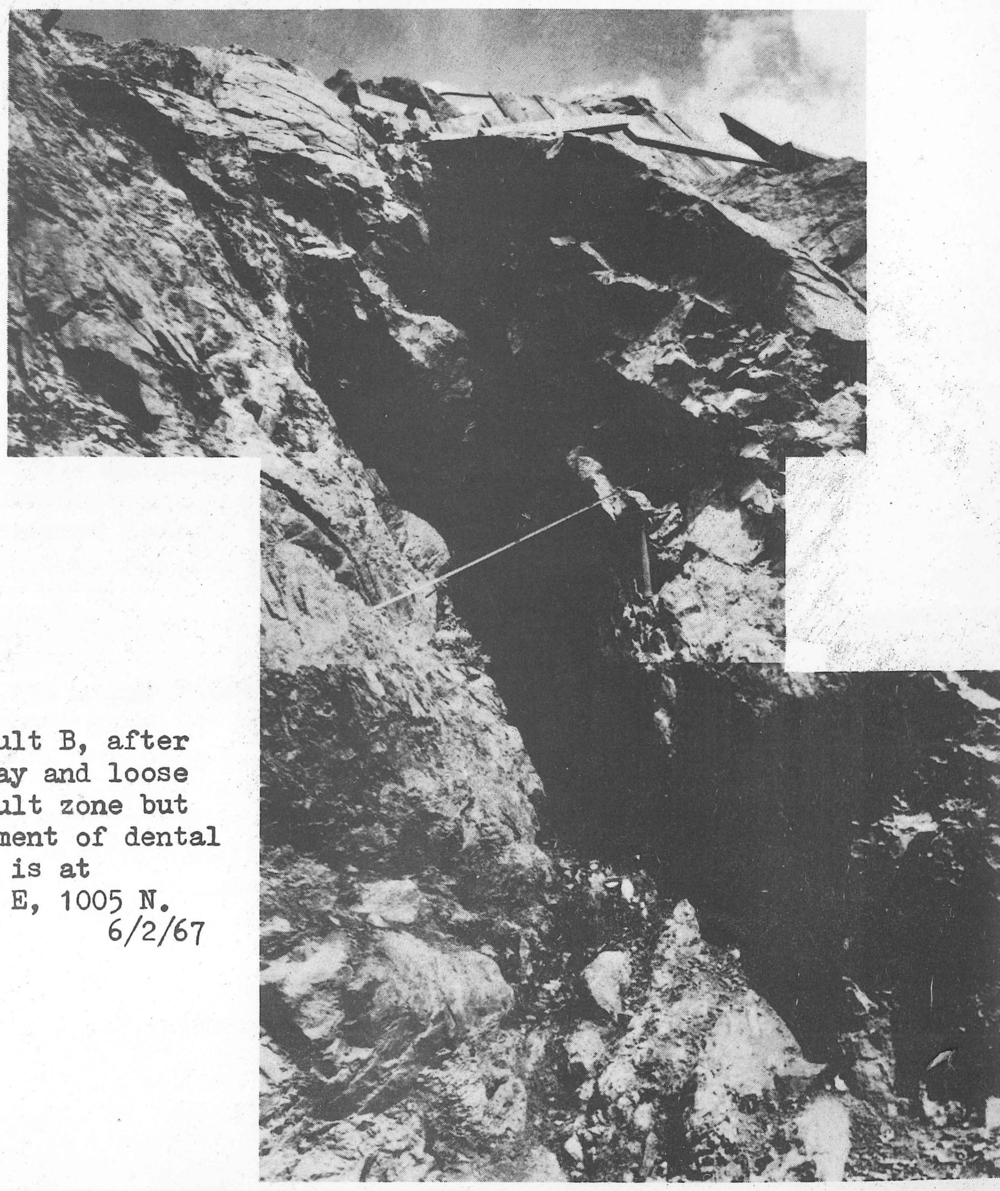

**Fig.** 9 (b) :- Fault B, after excavation of clay and loose rock from the fault zone but before the placement of dental concrete. Hammer is at co-ordinates 652 E, 1005 N.
M 484/35-37 6/2/67

## GEOLOGICAL SERVICES BEFORE CONSTRUCTION

The systematic geological investigation of the damsite commenced in April, 1961, and continued, with one major break of 21 months, until August, 1965. Three distinct phases of site investigation - preliminary, feasibility and design - were carried out during this period; they are described briefly to show the sequence of the investigation, the techniques used, and their relationship to the types of dam considered at each stage of the site investigation.

### HISTORY OF INVESTIGATIONS

The preliminary investigation consisted of a general appraisal of the engineering geology of the site, and this was used as a basis for comparison with an alternative damsite. Geological information was obtained by mapping outcrops and bedrock exposures in hand-dug pits and trenches. This was followed by a programme of diamond drilling and water pressure testing of specific geological targets. A seismic refraction survey was carried out at the damsite to supplement information from drilling and surface mapping (see Wiebenga, Polak and Kirton, 1962), and reconnaissance geological surveys were made of possible sources of construction materials. It was concluded that the site was unsuitable for the construction of an arch dam, though it appeared to be well-suited for a concrete gravity or rockfill dam (Best and Hill, 1962).

After the decision had been made to construct a dam at Corin damsite, a detailed feasibility investigation was carried out to determine the type of dam most suited to the site. Initially, three types of dam were considered; a concrete buttress dam, a multi-arch dam, and a rock-fill dam. The feasibility investigation, consisting of additional diamond drilling, water pressure testing, geological mapping and geophysical investigations, was directed firstly to obtaining general information common to the three types of dam, e.g. distribution of rock types, degree of weathering and jointing, leakage, and so on. Later, special requirements for each type of dam were investigated (such as spillway line for a rockfill dam and buttress foundations for a multi-arch dam). A provisional cost-estimate of the three types of dam indicated that the multi-arch and rockfill designs were of comparable cost, and cheaper than a buttress dam. At this stage, an earthfill dam was also considered because test pitting indicated a considerable volume of suitable construction material less than a mile from the damsite; this type of dam was later rejected on economic and technical grounds.

One of the major phases of the feasibility investigation was the sluicing of large areas of the proposed foundations with high-pressure water jets. Much valuable geological information was obtained by this technique, and three major fault zones were exposed; one of these is critical, as it affects a large area of the foundations close to the dam axis. Insufficient time was available to determine the exact location, attitude, and nature of the fault at depth, and this was a major factor in the decision of November, 1964, to design and construct a rockfill dam.

The design investigation was initiated as soon as the type of dam had been decided, and continued until August, 1965. By January, 1965, it was decided that the spillway should be located on the west bank and the diversion tunnel on the east bank. The search for construction materials proved more difficult than anticipated and several possible areas for rockfill were drilled before a suitable quarry site was located. By July, 1965, the spillway site, tunnel portals and valve tower shaft had been investigated by geological mapping, diamond and water pressure testing; the main fault zone at the damsite had been intersected at depth by drilling; an adequate supply of rockfill material had been found; and systematic augering and pitting in the Cotter valley upstream of the damsite had located sufficient impermeable core material (Best, 1965).

All aspects of the site investigation are described in detail in two reports, which are accompanied by all relevant geological maps, sections, and drill logs; the preliminary investigation is described in Best and Hill (1962) and the feasibility and design investigations are recorded in Best (1965). As the present report deals with the overall picture of the engineering geology of the site revealed during investigation and construction, the main features and techniques of the geological investigations before construction are briefly described below.

## GEOLOGICAL MAPPING

### Outcrop mapping

The first stage of the preliminary investigation consisted of mapping all outcrops in an area measuring 1500 feet by 1200 feet surrounding the damsite. Geological observations were plotted at a scale of 50 feet : 1 inch by plane table tacheometry, supplemented by Abney level, tape and compass traverses.

During the feasibility investigation, outcrop mapping was extended over an area of 3,500 feet by 4,500 feet on the east side of the river valley, as there are several possible leakage paths from the reservoir in this area (see Plate 2). The most obvious leakage path, however, is along the Cotter Fault zone immediately to the west of the dam; this area was therefore mapped in as much detail as possible, and a separate report on possible leakage of water along the Cotter Fault was prepared (Oldershaw, 1965). At the same time, all outcrops in the reservoir area were mapped. The outcrop mapping during the feasibility investigation was plotted at various scales, depending on the detail of geology exposed, but all information was compiled and reduced to a scale of 1 inch : 400 feet (Plate 3).

During the design investigation, the only additional outcrop mapping necessary in the damsite area was at the tunnel outlet portal; an area 300 feet by 200 feet was mapped by plane table tacheometry at a scale of 1 inch : 20 feet. However, a considerable amount of mapping was carried out at the prospective quarries for rockfill. Several areas were mapped by tape, compass and Abney level, at a scale of 1 inch : 40 feet, before a suitable quarry site was located (see Best, 1965).


**Fig.** 10 :- Fault D exposed in the western core zone foundations, looking downstream. Folded, well-bedded quartzite crops out uphill, while cleaved, laminated siltstone is exposed immediately downhill of the fault. Co-ordinates of P are 360 E, 1000 N.
G 9390 - 9392 18/10/66

**Fig.** 11 :- Typical core zone foundations in the steeply-dipping, interbedded sandstone and siltstone on the lower slopes of the west bank.
G 9393-9394 18/10/66

Pitting and trenching

Because of the paucity of outcrops at the damsite, it was necessary to expose bedrock between outcrops so that the lithology, stratigraphy and geological structure could be determined. The outcrop mapping of the preliminary investigation was therefore supplemented by 1100 feet of hand-dug trenches and eight hand-dug pits; 200 feet of the trenching failed to expose bedrock because the overburden was too deep.

Bulldozed trenches were used extensively during the feasibility and design investigations to provide continuous bedrock exposures. Their use at the damsite was restricted to the proposed spillway area, where several trenches totalling 500 feet in length were excavated; and to the gully on the east bank at the upstream margin of the proposed dam (total length of trenches was 90 feet). However, trenching was a particularly useful technique during the mapping of possible sources of rockfill material, and also during the investigation for possible leakage paths between Kangaroo Creek and White Sands Creek. About 7,000 linear feet of bulldozed trenches were excavated specifically to provide geological information during the investigation. Useful geological data were also obtained from exposures in the many roads, and access tracks bulldozed through the area under investigation.

Pitting was used extensively in the search for suitable sources of core and filter material. All river flats and likely sources of core material for several miles upstream of the site were investigated by a grid pattern of pits excavated by a mechanical backhoe. The grid spacing was generally 200 feet, and the pits in the prospective core borrow areas were 10 feet deep; pits in the alluvial deposits were generally only 7 or 8 feet deep, as large boulders prevented the backhoe from penetrating to its full range. In the most promising of the core borrow areas, penetration to greater depths was achieved by bulldozing trenches as deep as possible, and then excavating backhoe pits in the bottom of the trenches. Hand augering in some of the pits was also carried out.

Sluicing

The preliminary investigation indicated that the geological structure at the damsite is complex. It was therefore decided that the foundations of the damsite should be stripped of soil and scree by bulldozers, and the exposed rock washed by high-pressure water jets. (For details of the equipment used see Best, 1965.) As effective sluicing of the foundations did not start until late in the feasibility investigation, it was necessary to select areas of the foundations for this treatment. Even though the programme of sluicing was less extensive than originally planned, the results obtained were of great value.

About 40,000 square feet of foundations were sluiced and geologically mapped at 1 inch : 10 feet; the information obtained greatly facilitated the interpretation of structure and rock types at the site. Sluicing also provided a representative sample of the foundations for inspection by engineers, consultants, and prospective tenderers, thus providing a sound basis for the evaluation of the foundation treatment considered necessary. The location, during the feasibility investigation, of a faulted area on the east bank was invaluable, as this was an important factor in deciding the type of dam to be constructed. Had the fault not been discovered until construction had commenced, additional investigation and treatment, with consequent costly delays, would have been inevitable.

Sluicing was also used during the design investigation to expose bedrock at the proposed inlet portal. An area 120 feet by 60 feet was sluiced and geologically mapped at 1 inch : 10 feet; faults and folds were revealed which could not have been reliably assessed by any other investigation technique.

## DIAMOND DRILLING

Forty eight diamond drill holes, totalling 7,151 feet in length, were drilled by the Snowy Mountains Hydro-Electric Authority, under the direction of geologists from the Bureau. Fourteen of the holes, totalling 2,604 feet, were drilled to test prospective sources of rockfill material; three holes, totalling 869 feet, were drilled to evaluate leakage through the dam abutments; and the remaining 31 holes, totalling 3,678 feet, were drilled to test foundations for the dam, spillway, tunnel portals, and valve tower shaft. The footages drilled during the preliminary, feasibility and design investigations were 801; 2,994 and 3,356 feet respectively. The depths, coordinates, elevations, inclinations, azimuths and objectives for all holes are tabulated in Appendix 2.

All holes were drilled with NMLC bottom discharge bits and stationary split inner tube core barrels, and core loss was restricted to the washing away of some clay in a few of the fault zones. Geological logs, at a scale of 1 inch : 10 feet, of all drill holes are included in the two reports of the site investigation (Best and Hill, 1962, and Best, 1965).

## WATER PRESSURE TESTING

All holes drilled during the preliminary investigation were water pressures tested as drilling progressed. Pressure water for the tests was supplied by the drills' circulation pumps which had a maximum capacity of 15 gallons per minute (g.p.m.). As the rate of water leakage from the test sections frequently exceeded 2 g.p.m. per foot of hole, it was necessary to test the drill holes in sections of 10 feet or less; even so, many of the tests were inconclusive as a sufficient range of gauge pressures could not be obtained.

During the feasibility investigation, water pressure testing techniques were modified to enable reliable results to be obtained in high leakage zones. Flush-coupled EX casing replaced N-rods in the supply line, thus considerably reducing friction losses at high flows.

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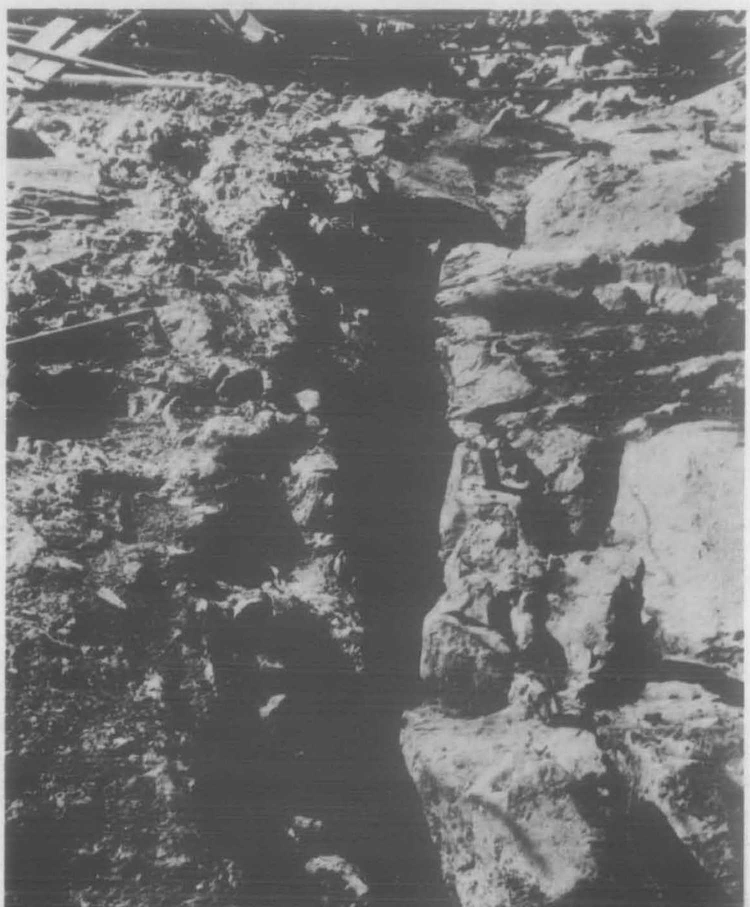

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Fig. 12 :- Views of Fault E exposed in the valley floor core zone foundations. The top view shows the fault after sluicing of the bedrock surface; the bottom view shows the extent of excavation for concrete dental treatment. Approximate co-ordinates of the centres of both views are 540 E, 985 N.

A centrifugal pump with a capacity of 140 g.p.m. was used to provide the pressure water for the first four holes, but even this was not entirely satisfactory. A system of gravity feed was therefore used in all later holes; this consisted of a 2,500-gallon water tank high up on each hillside which supplied water to all drill sites through a 3-inch diameter pipeline. The tanks were kept full of water by reciprocating pumps operating from the river, and the elevation of the tanks above the river (400 and 440 feet) was such that the gravitational head of the water was sufficient to produce an adequate range of gauge pressures for water pressure testing. This arrangement worked very well indeed, and is recommended for investigations where high leakage rates are likely to be obtained.

The effective pressures of water in the test sections were calculated from the recorded gauge pressures by allowing for the water column in the supply line, back pressure due to groundwater, and pressure losses in the supply line and packer. This was the first project investigated by the Bureau at which corrections were made for friction losses caused by the packers; it was therefore necessary to calibrate the packers used to determine the friction losses at a wide range of water flows (4 g.p.m. to 80 g.p.m.). Calibration tests were also carried out on N-drill rods with streamflow couplings, as correction graphs were not available for this equipment (which is often used for the supply line on other projects). Practical details and the results of the calibration tests are given in Appendix 3; it is evident from these results that for a given flow, the friction loss for an N-size mechanical packer is slightly higher than for an N-size hydraulic packer, but considerably lower than for a B-size hydraulic packer. It was considered inadvisable to use N-hydraulic packers at Corin damsite, because the closely-jointed rock could easily jam the packer in the hole. Most of the water pressure testing was therefore carried out using N-mechanical packers with 20 feet of perforated N-rod as an injector tube.

The corrected results were presented as histograms showing water losses at a pre-determined range of pressures plotted against depth. However, a more useful presentation of the results was obtained by calculating joint permeabilities, using a formula derived by the Snowy Mountains Authority, and plotting the results against depth. Although the derivation of the formula assumes isotropic rock and laminar flow (conditions which are seldom obtained), it is considered that these values are more useful for comparing results from different drill holes and damsites than either the histogram method used above or the calculated Lugion values.

Full field results of the water pressure tests, their corrections, and the water loss and joint permeability histograms are given in the site investigation reports.

## WATER LEVEL OBSERVATIONS

Two of the diamond drill holes (D.D.5 and D.D.11) were cased for their entire depth with perforated casing so that groundwater level measurements could be made regularly during investigation, construction, and filling of the reservoir. Water level measurements were also possible in D.D.10 during the investigation, but the hole became blocked as soon as site preparation for construction commenced.

Water levels recorded in drill holes D.D.5, D.D.10, and D.D.11, between April 1964 and August 1969 are tabulated in Appendix 4, together with the corresponding rainfall figures at the site. The water table configuration revealed by these observations is discussed in the section "Leakage from the site".

An additional water table observation hole was drilled and cased during construction of the dam; details of the planning of this hole are given in the section on "Leakage from the site".

## GEOPHYSICAL INVESTIGATIONS

Geophysical methods, both field and laboratory, were used extensively in the course of the preliminary and feasibility investigations. Seismic refraction traversing was the technique most suited to the field investigations, and traverses totalling 15,150 feet were carried out by the engineering geophysics group of the Bureau; a geophone spacing of 50 feet was used throughout, and most traverses consisted of 24-geophone spreads. In addition, a geophysical survey of prospective spillway locations was carried out by a private geophysical company, using a portable 12-channel seismograph; 8 traverses totalling 5,000 feet were surveyed, using geophones at spacings of 20 feet. The seismic refraction survey was also supplemented by 1200 feet of resistivity traversing and 5,450 feet of magnetic traverses.

Laboratory geophysical investigations were carried out on 78 specimens of drill core from the damsite. Static and dynamic tests were conducted, and some samples were tested both wet and dry. Elastic moduli were determined from these tests and also from the field investigations. This programme for determining the physical properties of the foundation rocks was carried out to provide design information, should a concrete dam be built at the site; the results are therefore not included in this report.

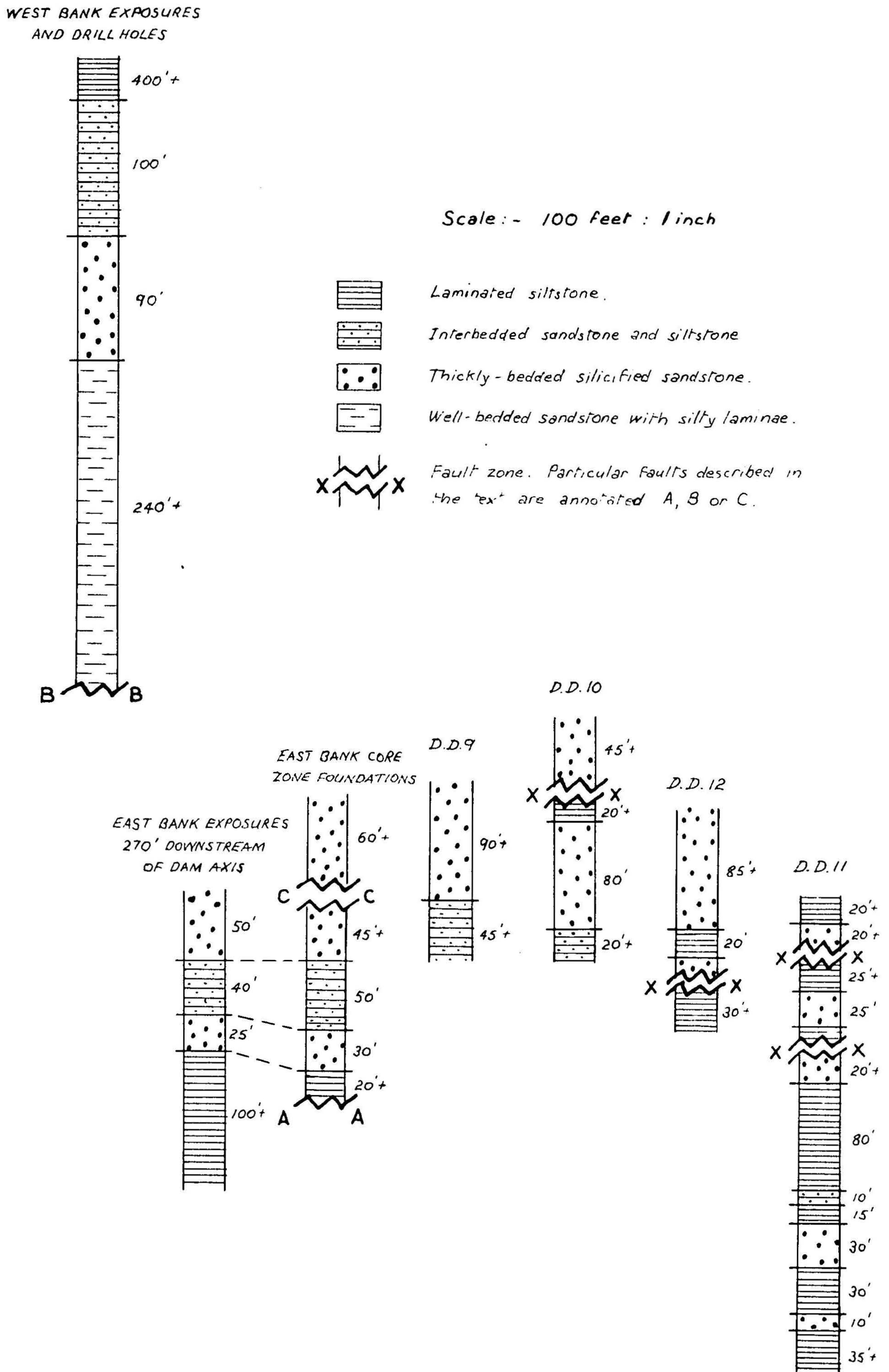
Details of the techniques and results of the geophysical surveys are given in Wiebenga, Polak and Kirton (1962), Polak and Kevi, (1966), and Starkey (1964); the geological implications of the results are discussed in the site investigation reports and in Best and Hill (1967).

## PETROGRAPHY

Five specimens of drill core from the preliminary investigation were selected for microscopic investigation, in thin section, to determine the nature of sedimentation and the origin of the pyrite in the specimens; the petrographic descriptions are given in Appendix 2 of Best and Hill (1962).

CORIN DAM **FIG. 13**

## COMPOSITE STRATIGRAPHICAL SEQUENCES AT DAMSITE




To accompany Record 1969/111 I.55/A16/517

During the investigation of the Cotter Fault zone by Oldershaw in 1964, representative samples of rock types exposed in and adjacent to the fault zone (phyllite, siltstone, silicified sandstone, and quartz porphyry) were collected and examined in thin section. The petrological characteristics of the rocks are described in the text of Oldershaw (1965), but separate descriptions of the thin sections are not given.

Petrographic methods were used to determine the amount of pyrite in the quartzite beds of the rockfill quarry - this information was required to assess the suitability of the rock for use as concrete aggregate. Specimens of drill core were cut with a diamond saw and examined in reflected light under a microscope. The pyrite was identifiable without polishing the cut surface, and a point counter was used to determine the percentage of pyrite in the core samples. The results are given in the section on "Concrete Aggregate".

## GEOLOGICAL SERVICES DURING CONSTRUCTION

### DETAILED FOUNDATION MAPPING

#### Damsite

Foundations for the core and filter zones of the dam were cleaned down with compressed air and water jets at an early stage of the construction - this was necessary so that surface leaks during blanket grouting could be readily detected and treated. Geological mapping was therefore carried out as soon as possible, before the rock surface became obscured by cement and grout. At this early stage there was little survey control on the foundations, and it was necessary to use plane table tacheometry extensively to provide control for the geological observations; all of the eastern foundations and about half of the western foundations were mapped by this method, using 340 and 300 control stations respectively. Before the geological mapping was completed, however, the positions of all blanket grout holes were painted on the bedrock surface by the contractor. These holes are on a 10-foot grid, and by plotting their positions on sheets of graph paper it was possible to sketch the geology directly onto the field sheets at a scale of 1 inch : 10 feet. This method was used for mapping the remainder of the western foundations and the valley floor foundations.

A geological map of the core and filter zone foundations was compiled from all field sheets at a scale of 1 inch : 10 feet. (1:120) The information contained on this map provided the basis for the evaluation of any problems which arose during grouting and foundation treatment. As construction progressed the map was amended as necessary in areas where foundation excavation and treatment substantially modified the surface trace of geological features recorded on the map. The final map of the core and filter zone foundations is reproduced at a scale of 1 inch : 20 feet in Plate 7. (1:240)

Detailed geological mapping of the rockfill zone foundations was not possible, as it was not necessary for the contractor to remove the rock rubble from the bedrock surface. Geological features exposed in the rockfill zone foundations were mapped by plane table tacheometry at a scale of 1 inch : 40 feet; this information is included in Plate 4. However, many of the geological observations were isolated in rubble-covered areas, and may therefore not have been typical for the general area in which they were located. The lack of continuous exposure in the rockfill zones has also considerably hindered the interpretation of the stratigraphy and geological structure at the site.

### Spillway

The walls and floor of the excavation for the spillway were geologically mapped at a scale of 1 inch : 10 feet, except for the western wall of the overflow channel excavation, which was mapped at 1 inch : 20 feet. The walls of the excavation were mapped in elevation, and control for the profiles was obtained by plane table tacheometry. The western wall of the overflow channel is 50 feet high, and although it was mapped in three stages, it could not be mapped accurately enough to justify a map scale of 1 inch : 10 feet. The floor of the spillway excavation and the foundations for the crest and buttressed sections were mapped after final cleaning for placement of concrete. The geology was sketched onto field sheets, on which were plotted the concrete construction joints, and there was sufficient control for the sketching, in the form of anchor bars, for plane tabling to be unnecessary. The geology of the floor and walls of the spillway excavation is plotted in Plates 9 and 10.

### Tunnel

The diversion tunnel was geologically mapped at 1 inch : 10 feet as excavation progressed. It was difficult to trace the continuity of geological features from one wall to the other, because the use of steel sets and timber lagging obscured the roof exposures; this situation was aggravated by the fact that the tunnel alignment is at an acute angle to the strike of many of the geological features. Geological logs of the tunnel mapping are given in Appendix 6.

Geological mapping was carried out at the inlet portal to supplement the information recorded from the sluicing and diamond drilling; Plate 12 is a map of the inlet portal as excavated. However, no mapping was carried out at the outlet portal because of the complexity of the exposed geology.

## GROUTING INVESTIGATIONS

Numerous consultations between engineers and the site geologist were held to discuss the progress of grouting, hole spacing and orientation, likely causes of high grout consumption, effectiveness of grouting, and the location of uplift gauges and grout caps. A description of the grouting programme, its relationship to the site geology, and analyses of grout consumption are given in some detail in a later chapter of this report.

SLOPE STABILITY

Geological advice was frequently requested on the stability of steep slopes excavated in bedrock. Slope stability was particularly important in the spillway excavation, where conditions after excavation were often at variance with the anticipated conditions; details of the particular problems and their treatment are given in the section on engineering geology of the spillway. The quarry and the tunnel portals were other areas where slope stability was a matter for consideration during construction.

FOUNDATION TREATMENT

As the programme of blanket grouting progressed, doubts arose as to the efficiency of the grouting in the top two or three feet of the bedrock surface. To ensure that water could not penetrate to the contact between the core and the foundation, a rigorous programme of foundation treatment was carried out. Geological advice was requested on many occasions where the attitude, nature and extent of particular geological features had a considerable bearing on the type and extent of foundation treatment in a particular area. The criteria for the various methods of foundation treatment were assessed early in the construction period during discussions and field inspections with the site engineers and supervisors. The foundation treatment is discussed in detail under "Engineering Geology - Damsite", and Plate 8 shows the treatment carried out before core placement.

DEVELOPMENT OF ROCKFILL QUARRY

Before any rockfill could be obtained from the quarry, the area had to be stripped of soil and scree, and the laminated siltstone overlying the suitable rockfill material had to be removed by ripping and blasting. As the strata exposed at the quarry are considerably folded and faulted, the lower limit of the siltstone could only be determined by frequent inspections of the stripped area. During the stripping, a fault was revealed which considerably reduced the economic area for quarrying; geological mapping, supplemented by percussion drilling, was necessary to evaluate revised limits for the quarry.

During the quarrying operations, regular inspections were made to check the exposed geology with the anticipated conditions, and supplementary geological mapping was carried out. This enabled assessments to be made, as required, of areas where a particular grade of rockfill could be obtained or developed.

The development of the quarry is considered in detail in a later chapter.

# ENGINEERING GEOLOGY

## DAMSITE

The cleaning down and preparation of foundations for the dam did not reveal any unexpected geological features, and foundation conditions were very much as predicted from the site investigation. Construction of the dam therefore went ahead without any special foundation investigations or treatment being necessary.

### Overburden

A total of 150,600 cubic yards of overburden, consisting of topsoil, slopewash, scree and alluvium, was removed from the dam foundation area during construction. The distribution of the overburden is shown in Plate 6, which was compiled from detailed surveys before and after cleaning of the foundations; in this Plate, naturally-occurring pockets of thick overburden are distinguished from the mounds of overburden produced by bulldozing of the abutments during the investigation. Of the naturally-occurring pockets, A1 and A3 were delineated during the site investigation and were allowed for in the contract estimate of quantities. A2 represents the accumulation of scree at the foot of the prominent quartzite outcrop on the west bank. The quartzite is abruptly terminated by a fault, and it is evident from the bedrock contours that the Cotter River has eroded the siltstone to the south-east of the fault zone, forming a steep slope along the margin of the quartzite. The notch so formed has subsequently been filled with scree from the quartzite outcrop to a maximum vertical thickness of 23 feet. The depression at A4 was also formed by differential erosion along a fault zone with thickly-bedded quartzite on the uphill side of the fault; the maximum vertical thickness of overburden in this area was 17 feet.

### Foundation Treatment

#### Rockfill zones

The rockfill-zone foundations were treated mainly by removing the overburden down to bedrock; mechanical scrapers, bulldozers and front-end loaders were used. Although the resultant bedrock surface was generally littered with loose rock after the operation (because of the heavy equipment used), in most areas it was obvious that no further treatment was necessary, and rockfill was placed on this surface. In the valley floor, however, there were small depressions and gullies in the bedrock surface of the downstream rockfill zone and these were inspected by hand-dug trenches to ensure that no persistently-wide clay seams were left untreated before placement of rockfill. Only one seam was considered sufficiently persistent to require treatment, and this was excavated by hand and backfilled with filter material. The extension of the 40° fault zone into the downstream rockfill zone high up on the east bank was similarly treated.

Core and filter zones

After the removal of overburden from the damsite by bulldozers, the bedrock surface below the core and filter zones was cleaned down by compressed air and water jets. At this stage, much of the loose rock at the surface was removed, either by the air and water jets or by pick and shovel. Next, the foundations were consolidated by blanket grouting, with holes at 10-foot centres. When the blanket grouting in the valley floor was completed, the final foundation treatment was carried out, and the core and filter material placed soon after. The blanket grouting and final foundation treatment then progressed steadily up both abutments ahead of core placement.

The design criteria for the core zone and filter zone foundations were that they should be non-erodible and impermeable. The main purpose of the blanket grouting was to render the top few feet of the bedrock impermeable, and thus prevent water from penetrating to the interface between the bedrock and the earth core. It was anticipated that grouting would not be completely effective in the closely-jointed rock forming the foundations and two diamond drill holes were drilled into grouted rock to evaluate the effectiveness of grouting. Unfortunately, the drill core obtained was badly disturbed (owing to poor drilling techniques and equipment) and only a few joints showed any evidence of grout penetration. Excavation of rock for final foundation treatment also indicated that many joints and cracks near the surface were not grouted. The final foundation treatment was therefore carried out with three objectives:

(1) to complete the impermeable barrier partly-formed by the blanket grouting;

(2) to remove seams of erodible or compressible material, or to cap them with concrete or mortar; and

(3) to treat near-vertical or overhanging rock surfaces so that the slope of the contact between earthfill and foundation does not exceed 2 : 1 (vertical to horizontal)

Criteria for foundation treatment were established during field inspections and consultations between the site engineers and geologist. Several factors (such as width, persistence, orientation and attitude of seams with respect to the dam layout and to the bedrock surface) had to be considered when evaluating the necessary treatment for each defect in the foundations; it was therefore impossible to apply rigid criteria for the treatment. However, examples of typical foundation treatment which illustrate the general criteria used are given in the following table:

| FOUNDATION DEFECT | TREATMENT |
|---|---|
| **Fissures** | |
| Open joints and fissures | Remove any loose blocks of rock. Apply cement slurry, mortar slurry or P.A.M.* to open fissures in final foundation surface. |
| **Clay Seams** | |
| Seams up to 2" wide | Cover with P.A.M.* or mortar slurry. |
| Seams 2" to 4" wide | Excavate seam and backfill with P.A.M.* or mortar slurry. |
| Seams more than 4" wide | Excavate to depth equal to width of seam and backfill with concrete. |
| **Weathering** | |
| Pockets of highly weathered rock. | Excavate until suitable foundation rock is exposed. |
| **Steep Bedrock Slopes** | |
| Bedrock slopes with a gradient of 2 vertical: 1 horizontal or steeper. | Slopes flattened to 2: 1 by trimming, backfilling with concrete, or a combination of both. |
| **Ungrouted rock at surface** | |
| Some areas occurred along grout curtain line where geological features indicated that top 2 or 3 feet of bedrock may not be effectively grouted. | Excavate trench 3 feet into bedrock along curtian line and backfill with concrete (Grout cap); jackhammers used for excavation. |

*P.A.M. is pneumatically - applied mortar.

Plate 8 is a plan of the core zone foundation, showing details of the final foundation treatmen+ It was impracticable to represent all the areas where cement slurry was used, but all individual areas where mortar slurry, P.A.M. and concrete were used are shown on the plan. The quantities of the various types of treatment used were as follows:

| | |
|---|---|
| Concrete for grout caps | 157 cubic yards |
| Concrete for dental treatment | 682 cubic yards |
| Pneumatically-applied mortar | 161 cubic yards |
| Mortar slurry | 45 cubic yards |
| Cement slurry | 9½ cubic yards |

Fig. 14 :- Open bedding plane joints in thickly-bedded quartzite exposed in the valley floor core zone foundations (co-ords 540 E, 1025 N ). These joints were backfilled with fine aggregate concrete during foundation treatment.
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Fig. 15 :- Dental concrete foundation treatment after the excavation and concrete backfilling of Fault A. This view looks upstream to the intersection of Fault A by Fault B. Small areas of concrete dental treatment of steep rock faces can be seen downhill of the trace of Fault B.
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The figure for the groutcap concrete includes the continuous grout cap along the curtain line extension below the spillway crest; of the total, about 50 cubic yards of concrete was used for grout caps in the core zone foundations.

The most extensive single feature requiring foundation treatment was, as anticipated in the design report, the 40° fault (Fault A of Plate 4) on the east bank. Plate 8 shows clearly the extent of the concrete dental treatment for this fault zone (see also Fig.15), and a considerable amount of hand excavation was necessary before satisfactory rock for concrete placement was exposed. The shattering and weathering of rock in the hanging wall of the fault is also reflected in extensive dental concrete used upstream of the surface trace of the main fault. All other major fault zones are narrow, and required little excavation and backfilling with concrete.

The long, narrow areas of treatment with P.A.M. and concrete evident in Plate 8 reflect the surface trace of bedding planes, as most of the clay seams and broken zones were caused by movement along bedding planes during folding of the rock sequence.

## SPILLWAY

### General

The site for the spillway at Corin Dam was dictated by the surface topography and bedrock configuration; topographic considerations eliminated the possibility of locating the spillway on the east bank, and the bedrock surface on the west bank restricted the position of the structure, within very narrow limits, to a rib of rock close to the downstream face of the dam. As the plan position of the spillway was thus virtually fixed, details of the design were largely based on laboratory model tests. These tests showed that a side channel arrangement was advisable to prevent scour of the upstream face of the dam during high spillway discharges. The need for a horizontal curve in the chute was also indicated by model tests, as the scour basin in the river bed was dangerously close to the outlet portal and toe of the dam with a straight chute layout. Because of this curve, the downstream end of the chute, including the ski-jump structure, has a super-elevated floor to assist in throwing the water away from the toe of the dam.

The entire spillway was excavated in bedrock. The vertical depth of the excavation below original bedrock surface ranged from a maximum of 80 feet in the overflow channel area to a minimum of 3 feet at the north-eastern corner of the ski-jump structure. The rock exposed in the floor of the excavation was everywhere more than adequate to support the concrete structure, and no special foundation treatment was necessary. However, problems were encountered in the walls of the excavation where adversely-dipping or closely-spaced joints were exposed, and changes in design were necessary to overcome these defects in the rock.

The geology of the spillway excavation is shown in Plates 9 and 10. The upstream end of the overflow channel is excavated in thickly-bedded quartzite and the underlying closely-jointed, well-bedded quartzite; apart from a few minor folds, the beds generally dip steeply to the south. At the upstream end of the chute, the spillway line crosses the main anticline exposed in the west bank of the damsite, and the well-bedded quartzite dips at 45° to 60° to the north-east on the northern limb of the fold. At chainage 520 feet, a major fault exposed in the excavation has downthrown the geological succession some 300 feet to the north-east; this has repeated the succession exposed in the overflow channel, and the chute structure downstream from the fault is founded on the thickly-bedded quartzite. The ski-jump structure is founded on the uppermost beds of the underlying sequence of well-bedded quartzite.

The engineering geological problems which arose during construction of the spillway are considered below.

## Spillway Crest

The spillway crest structure is founded on thickly-bedded quartzite, except at the upstream end where interbedded sandstone and siltstone was exposed (see Plate 11). After excavation to the founding level of the crest structure, the bearing strength of the rock was everywhere more than adequate. However, subsequent excavation for the overflow channel revealed the presence of a set of persistent joints which seriously reduced the stability of the crest foundations. The joint set dips at 30° to the north (i.e. towards the channel excavation and downstream), and many individual joints are open or lined with a veneer of orange-brown plastic clay; some joints persist for more than 20 feet at a constant attitude. The instability of the rock was evident from the considerable overbreak which occurred during excavation, particularly along the lip of the channel cut (see Fig.16), and it was obvious that preventive measures would be necessary to stabilise the crest foundations. This stability problem also had implications for the proposed design of the crest structure, which included the installation of a row of post-tensioned cables along the spillway crest (to counteract the overturning moment during high discharges). It was originally intended that the holes for these cables would be vertical, but as added vertical stresses would have increased the sliding forces along the potential failure planes, it was decided to instal the cables at an angle of 65°, or less, in a downhill direction. In consequence, the crest structure design had to be altered because the cable tensioning plates could not be left exposed for tensioning the cables after the crest had been constructed, as was originally intended.

The stability problem was overcome by extensive rock-bolting, and by constructing the concrete crest in three separate stages (see Plate 11). After excavation of the channel, the crest foundations were cleaned off with air jets and the area was geologically mapped at 1 inch : 10 feet. The mapping included the measurement of 200 joints, which were subsequently analysed by a contoured stereogram (see Fig.17); the purpose of the joint study was to evaluate the proposed orientations for the rock bolts and tension cables in relation to the attitudes of the main joint sets. On the basis of the geological information obtained, two rows of grouted rock bolts, nominally 5 feet apart and with a longitudinal spacing of 5 feet, were planned along the crest. The bolts are

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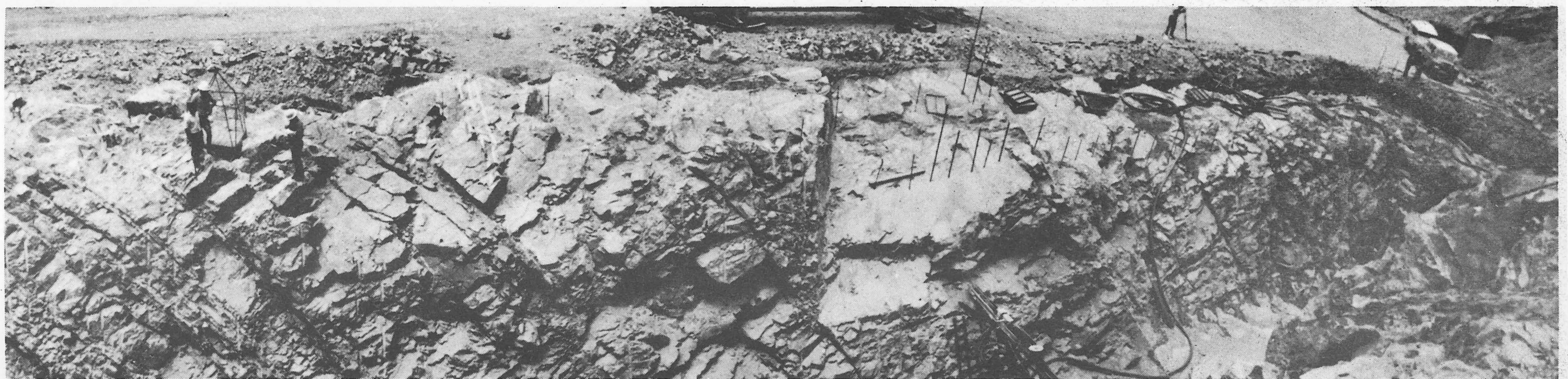

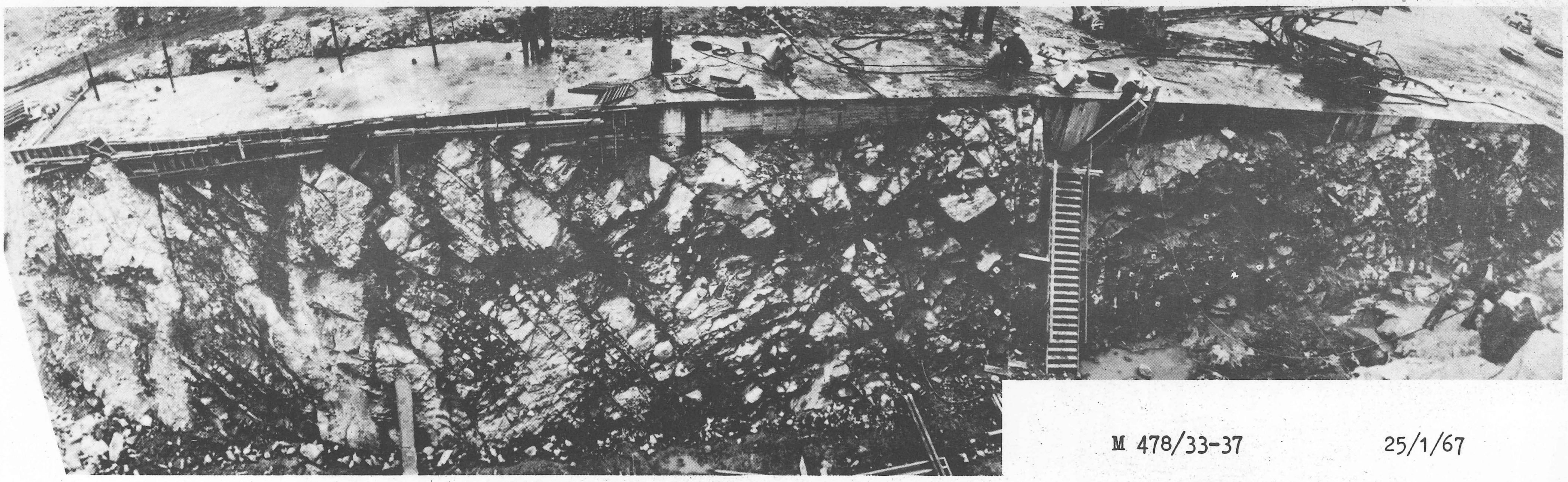

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**Fig. 16 :-** Views of the spillway crest foundations, before and after first stage concreting, showing the persistent, adversely-dipping joint set which caused overbreak along the lip of the excavation.

generally oriented normal to the dominant joint set, i.e. at 60° to the horizontal in a direction 174°M, and all bolts are 20 feet long. The length was determined by the desire to increase the coefficient of friction on all joints of the dominant set which are exposed in the wall of the channel (see section in Plate 11). In practice, the location and orientation of each bolt was determined by examination of the bedrock surface and interpretation of likely conditions to be encountered during drilling; the orientation of some holes was changed to avoid adverse geological features. To ensure good conditions for the bearing plates of these bolts, a concrete slab was poured on the crest foundations, and the bolts were installed from the surface of this slab, which is at R.L. 3125 feet. Fig. 16 shows the spillway crest before and after emplacement of the slab, and Plate 11 shows the position and orientations of the 46 grouted rock bolts which were installed in the crest. Bolts were not considered necessary in the upstream and downstream ends of the crest area, as the adverse joint set is not well-developed.

The orientation finally adopted for the 26 post-tensioned cables was virtually dictated by attitude of the adverse joint set, combined with the position of the bedrock surface downhill from the crest area. On the one hand, the joint set dictated that the cables should be angled at 65° or less from the horizontal in a downhill direction; on the other hand, the holes had to be as steep as possible, so that the anchorage zones are not too close to the bedrock surface. It was therefore decided that the collars of the cable holes should be located as originally planned, but that the orientation of the holes should be changed from vertical to 65° in a direction normal to the line of holes and downhill; inspection of the joint stereogram showed that this orientation is not near-parallel to any joint sets (see Fig.17). The cables are 35 feet long, have an anchorage zone 12½ feet long, and were tensioned to 50 tons before final grouting of the cable holes. A concrete anchor block was constructed along the crest before installation of the cables, and after tensioning, testing and grouting, the third and final stage of the concrete crest was placed (see section in Plate 11).

As an added precaution, the stability of the crest foundations was further increased by the installation of 135 grouted rock bolts in the wall of the channel excavation below the crest. The bolts are all 12 or 14 feet long, and replace a similar number of proposed 15-foot long anchor bars in the original design. However, whereas the anchor bars would have been installed on a regular pattern, the rock bolts were located according to the rock conditions exposed in the wall. The programme of rock bolting in the wall is shown in the elevation in Plate 11, and the attitude of these bolts in relation to jointing and bedding is illustrated in the section through the crest.

Overflow channel

Where the channel is excavated in the well-bedded quartzite succession, doubts were originally held regarding the stability of the western wall of the excavation, which is generally between 40 and 60 feet high. The rock is closely-jointed (1" to 24" spacing), many joints having a thin veneer of clay, and the rock was obviously loosened during the excavation. Rock bolting was attempted, but with little success, probably due to a combination of the attitude of bedding (which was nearly parallel to the bolt holes) and the use of expansion shell rock bolts. These bolts have a limited range of expansion to form the anchorage, and it is likely that under the prevailing geological conditions, the bolt holes tended to be enlarged during drilling; consequently the bolt anchor would not grip the rock.

As there were no unfavourably-oriented joint sets or fault planes which could cause major instability of the rock face (which has a batter of 4 vertical: 1 horizontal), it was considered that removal of the loose blocks of rock before concrete placement would reveal sound foundations for the concrete walls. The use of pre-splitting techniques during excavation was of considerable significance, as it reduced the overbreak and replacement concrete to minimal amounts. The application of P.A.M., with or without mesh, was considered for temporary support and protection against spalling and minor rock falls; however such measures were deemed unnecessary when the contractor decided to change his programme and place the concrete walls soon after completing the excavation (it was originally intended that the rock face be left exposed throughout the oncoming winter).

No problems were encountered during the construction of the concrete wall, and no rock falls occurred while the rock face was left unsupported.

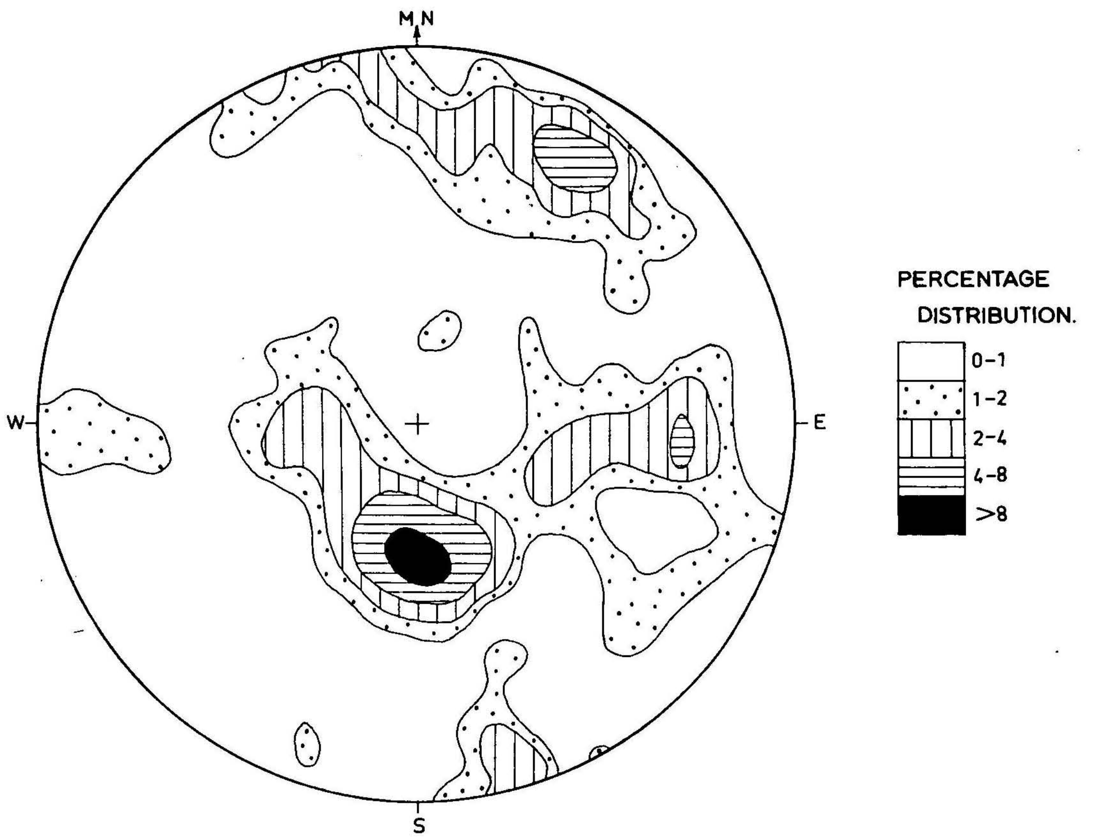
## Spillway chute

The well-bedded and closely-jointed quartzite is also exposed in the west wall of the chute excavation downstream to the fault at chainage 520 feet. However, as the bedding dips in towards the excavation between chainages 400 and 520 feet (see Plate 9) the rock slope was unstable at the 4:1 batter. The situation was further aggravated by closer jointing and deeper weathering than in other areas - the result of tight folding and the major fault in the area (see Figs 18 and 19). The original design for the western wall of the spillway chute required a rock slope against which the concrete was placed, as in the overflow channel. However, the stability of the rock slope was so doubtful that alternative designs for the chute were considered. The merits of a monolithic structure with self-supporting vertical walls were compared with a modification of the original design, whereby inadequate rock in the wall would be replaced by concrete; the former solution was found to be the safer and cheaper, and was the design finally adopted. The rock slopes behind the western wall of the chute were subsequently inspected, and any unstable rock which could fall and endanger the chute wall was removed.

## Ski-jump structure

The main problem with the ski-jump structure was to make a realistic assessment of the extent of scour in the river valley during high spillway discharges, and the likelihood of erosion undermining the

(a) Contoured stereogram of poles to 200 measured joint planes.




(b) Stereographic plot of dominant joint sets, together with adopted orientations of crest rock bolts and post-tensioned cables.

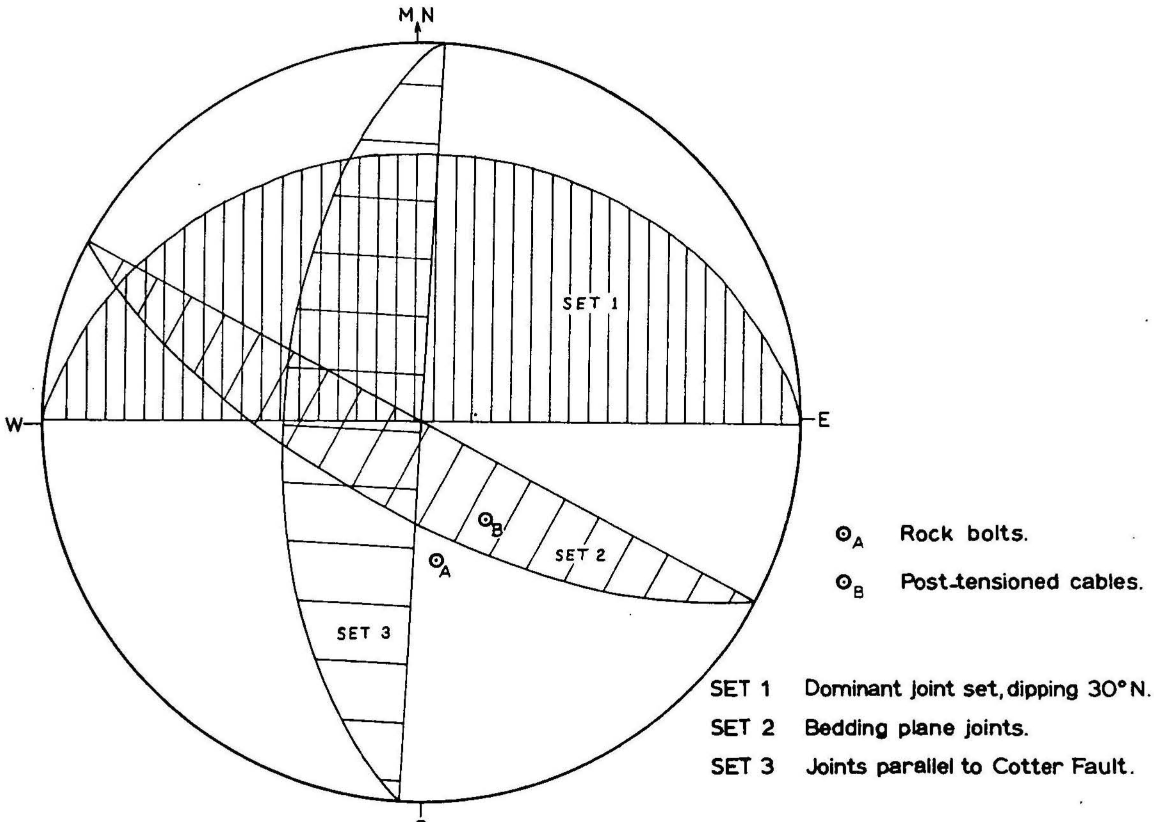



Fig. 17 :- Joint stereograms of spillway crest foundations.

All the above data are projected on to the lower hemisphere, using an equal area net.

*To accompany Record 1969/111* I 55/A16/516

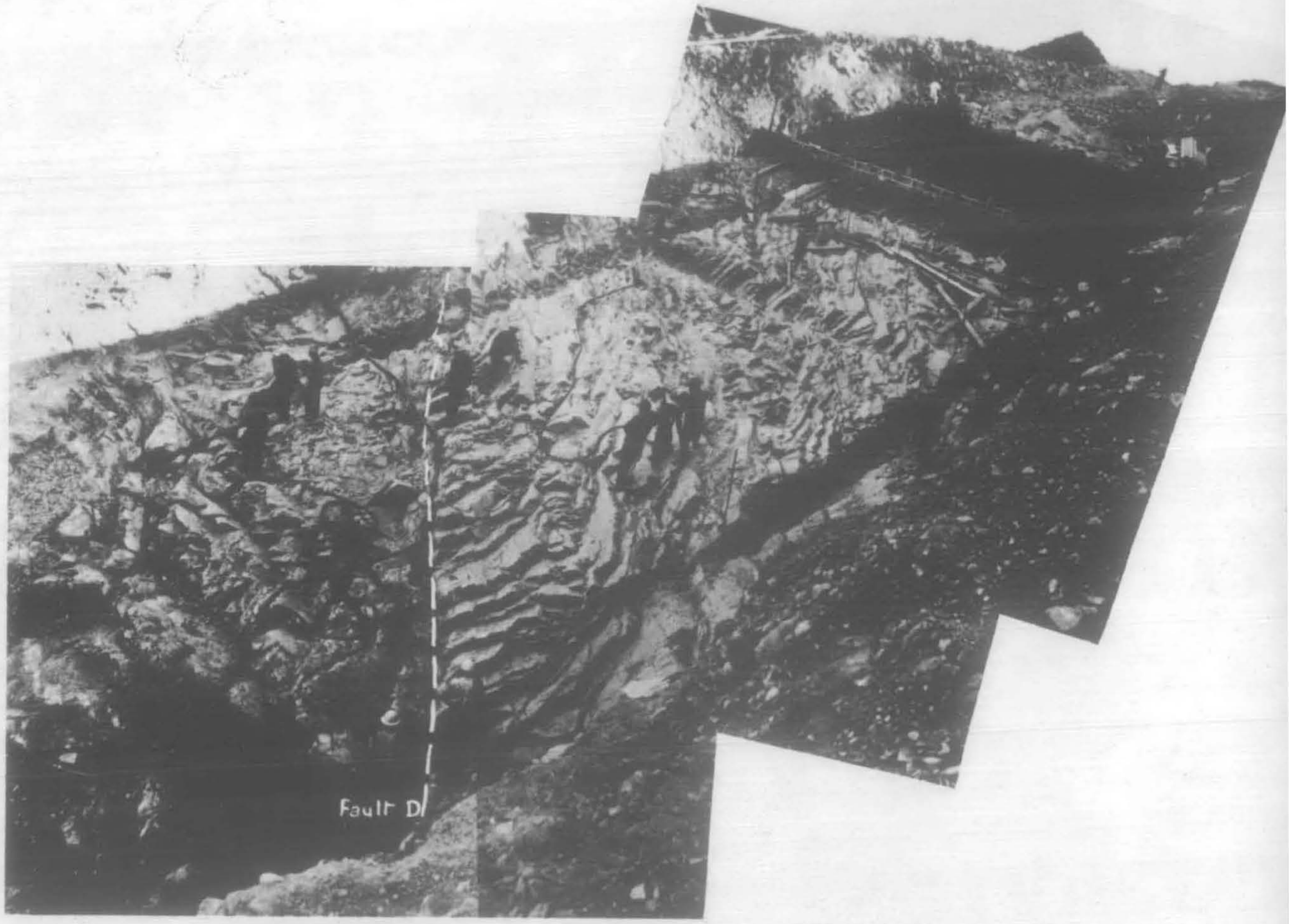



Fig. 18 :- Foundations of the spillway chute between chainages 460 and 540 feet. The thickly-bedded quartzite at the downhill end of the cleaned area has been downthrown against well-bedded quartzite along Fault D.
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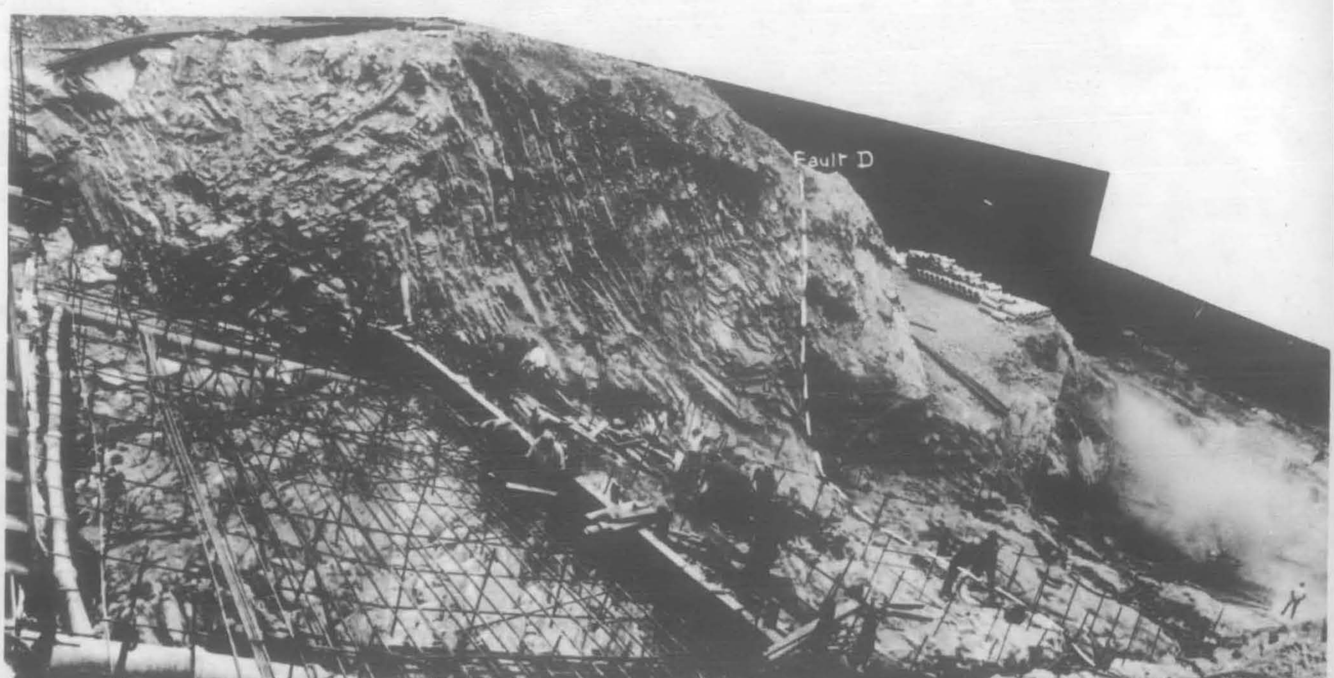


Fig. 19 :- The western wall of the spillway chute excavation between chainages 460 and 540 feet, showing adverse bedding dips, tight folding and closely-spaced jointing in the well-bedded quartzite. The instability of this face under load was a major factor in the decision to adopt a free-standing design for the spillway chute training walls.
M 489/17-22 31/3/67

spillway structure during very high discharges. Results from the model tests showed that a scour basin up to 80 feet deep could develop in the valley floor during high discharges, and continued running of the model at high flows resulted in progressive erosion of the sides of the scour basin by eddy currents. The fractured bedrock was represented in the model by gravel of various grain sizes, corresponding to the spacing of fractures in natural rock, but of course it is impossible to represent the interlock of jointed bedrock in such a model. It is considered that this natural interlock imparts a considerable factor of safety to the actual spillway, as compared with the results of model testing, and it is highly unlikely that even the maximum flood discharge would enlarge the natural stilling basin to the extent of undermining the ski-jump structure. However, because of the impossibility of obtaining reliable data for evaluating the problem, it was deemed prudent to construct a concrete cut-off wall, 25 feet deep, below the ski-jump, and to grout the rock for up to 40 feet below the bottom of the cut-off. Construction of a concrete cut-off was indicated on structural grounds alone, and the additional cost for grouting and making a deeper cut-off was considered a small price to pay for insuring the stability of the ski-jump structure.

An associated problem was the assessment of the necessary surface treatment of the bedrock exposed immediately downstream of the ski-jump. At low flows the water is not thrown very far from the ski-jump, and some protection of the exposed rock is necessary to prevent progressive erosion. The contract specifications called for blanket grouting of the rock, but in view of the low grout consumption in similar rock during blanket grouting of the core zone foundations, it was considered unlikely that grouting would significantly consolidate the rock below the spillway. It was decided that surface treatment with P.A.M., reinforced by steel mesh securely anchored to bedrock, would be adequate, and this was the only form of treatment carried out. An area of about 2,300 square feet in the floor of the excavation was treated in this way, and the application of P.A.M. was extended for 6 feet above the floor in the western wall.

DIVERSION TUNNEL.

## Inlet Portal

The geology of the inlet portal excavation is shown in Plate 12. The original bedrock geology was exposed by sluicing and was mapped during the design investigation; this showed that the dominant geological features, from the excavation viewpoint, are the anticlinal structure and the fault zone along the axis of the anticline. It was recommended in the design report that the portal face be established in rock on the north-eastern side of the fault, thus avoiding the main fault zone and adversely-dipping beds in the rock face.

The design position of the portal face is shown in the elevations of Plate 12, and excavations of the tunnel was initially carried out to this line. However, as the excavation was deepened towards invert level, the fault zone was exposed in the face; the fractured rock associated with the fault caused spalling behind the design line to such an extent that a near-vertical face, which was obviously unstable, was formed. It therefore became necessary to batter back the portal face by drilling a row of pre-splitting holes, at a batter of 2 vertical: 1 horizontal, in such a position as to intersect the unstable face at tunnel roof level - this was to reduce extra excavation to a minimum. The new face was supported, as excavation proceeded, by grouted rock bolts, 10 feet long and at nominal 5 foot centres, supplemented by wire mesh. Fig 20 is a composite photograph of the portal face 4 months after excavation, and shows the closely-jointed rock in the core of the anticline.

It was originally intended that the portal face be lined with pneumatically applied mortar as final treatment; however, it was decided that the grouted rock belts and steel mesh would provide adequate permanent support.

## Outlet Portal

The design investigation showed that the bedrock in the outlet portal area consists of a succession of highly contorted and faulted siltstone, with some sandstone interbeds. A horizontal diamond drill hole 160 feet long was drilled along the line of the tunnel from the river bank to test the rock at tunnel level, and to determine the optimum position for the portal face. Closely-fractured siltstone, with a prominent sandstone interbed about 10 feet thick was encountered in the general area of the portal face. The sandstone bed was seen to have a true dip of 65°, and from correlation with surface mapping, the most likely strike direction was parallel to the portal face with dip direction to the north-west. It was therefore recommended that, from the geological viewpoint, the portal face should be established at the upper contact of the sandstone bed. However, because of the steep slope of the east bank in the portal area, this would have resulted in an unduly large excavation; the final design position of the portal face was located in the siltstone overlying the sandstone bed. The portal was successfully established in the design position, due largely to a prominent set of joints which are almost parallel to the portal face. The sandstone bed was exposed in the tunnel 13 feet from the portal face (see Appendix 6- sheet 12 of the tunnel logs).

**Fig. 20 :-** Composite photograph of the inlet portal excavation showing the rock bolts and wire mesh supporting the portal face.

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Grouted rock bolts were used to support the portal face during tunnelling, and were later supplemented by pneumatically-applied mortar for permanent support.

## Tunnel

### General

As constructed, the tunnel is 1,308 feet long from portal face to portal face, and is fully concrete lined to a circular section 12½ feet in diameter. The tunnel was used for diversion of the river during construction of the dam; it also serves as the channel for water drawn from the reservoir through the intake tower. The tower, which has six inlet valves at various levels, is connected to the tunnel by a concrete lined shaft, 17 feet in diameter, located 235 feet from the inlet portal.

Excavation of the tunnel from the outlet portal commenced on 24th May, 1966, and from the inlet portal on 1st June, 1966. Much of the tunnel was supported by steel sets, which were placed as excavation proceeded, and 240 shifts were worked before breakthrough was achieved on 21st Julu, 1966. In the sections supported with steel sets, the tunnel was excavated to a horsehsoe-shaped section, 16 feet high and 16 feet across, except between chainages 228 and 300 feet where the dimensions were 19 feet by 19 feet; in the lengths of tunnel not supported by steel sets, the tunnel was excavated to a circular section about 14½ feet in diameter. The wedge cut method of excavation was used, with a general advance of 5 feet for each round, and in average conditions about 40 holes were required for each 5-foot advance. AN60 explosive was used for all blasting operations, and rounds were fired with 1 to 10 millisecond delay detonators. After firing, the spoil was mucked out with a front-end loader.

The contract specifications were written so that support of the tunnel during excavation was entirely the responsibility of the contractor, unless it was considered that the support installed by the contractor was dangerously inadequate. This situation did not arise, as the support adopted was extremely conservative -- steel sets were used to support all but 230 feet of the tunnel. The sets were fabricated from 6" by 5" I - sections of rolled steel, and were installed at a general spacing of 5 feet, decreasing to 4 or 3 feet towards the portals; a total of 229 sets were installed. The sections of tunnel where sets were not used were left virtually unsupported, as a total of only 37 rock bolts were installed in 230 feet of tunnel. Wedge and shell anchored rock bolts were used, and were generally 8 feet long; all bolts were used to pin isolated blocks of rock, and as they were for temporary support only, they were left ungrouted.

The valve tower shaft was excavated immediately after the tunnel breakthrough. The shaft was excavated to a nominal diameter of 20 feet, except for the top 20 feet which was excavated to a diameter of 27 feet -this was to allow for the thickening of the shaft lining necessary for a seismic design of the valve tower. The top 20 feet was pre split with 3-inch-diameter holes at 2½-foot spacing, and required only 2 circular steel sets for support. The rest of the shaft was supported by steel sets at 5-foot vertical centres.

Geology

Geological mapping of the walls of the tunnel was carried out periodically during excavation. The roof of the tunnel was also mapped in as much detail as possible; however, the timber lagging and cribbing used in the sections of tunnel supported by steel sets obscured most of the geological features in these areas. The geological data have been plotted,at a scale of 1 inch . 10 feet, on tunnel logs which also show graphically details of excavation and support (see Appendix 6). A more compact record of tunnel excavation is given in Plate 13.

The conditions encountered during tunnelling were substantially as anticipated in the design report. The rock is generally fresh, and laminated siltstone is the dominant rock type; about half of the tunnel length is excavated in thick sequences of siltstone. Interbedded siltstone and sandstone was encountered in the tunnel for a total length of 405 feet (30% of the total tunnel length), and the remaining 270 feet of tunnel is excavated in thickly-bedded silicified sandstone. The strike of bedding is near-parallel to the direction of the tunnel for much of the tunnel length, particularly downstream from chainage 350 feet (see Plate 13); the tunnel is therefore excavated in a part of the stratigraphical succession which is small compared with the length of the tunnel. Assuming there is no repetition of strata by faulting, the first 300 feet of the tunnel (from the inlet portal) peneteates 180 feet of the stratigraphical succession, while the remaining 1,000 feet of tunnel penetrates only 170 feet.

Jointing is moderately, to well, developed in the rock penetrated by the tunnel, and the percentage breakage along joints for the excavation generally ranges between 50 and 90. Joints in the laminated siltstone are commonly tight and fresh, while in the sandstone major joints are generally weathered and iron-stained; some are lined with a thin veneer of clay. The dominant joint set throughout the tunnel is parallel to the bedding, though other joint sets are locally well-developed (see tunnel logs). The predominance of tightly-jointed siltstone along the tunnel line is reflected in the absence of significant inflows of water during tunnel excavation. The scattered seeps which did occur were invariably in sandstones beds.

Only one wide fault zone was intersected by the tunnel; it crops out in the inlet portal area, and was penetrated by the tunnel 40 feet from the portal. At tunnel level, the fault consists of a zone 10 feet wide, of highly cleaved siltstone, which caused some overbreak in the roof. All other faults intersected by the tunnel consist of zones of crushed rock and clay less than 18 inches wide. There are 9 faults with zones of crushed rock and clay between 6 and 18 inches wide, and most of these are part of the dominant fault system at the damsite with northerly to north-westerly strikes; consequently the faults were intersected by the tunnel at an acute angle, and were exposed for some distance along the tunnel line (up to 70 feet). However, rock immediately adjacent to the fault zones is generally not severely affected by the faulting; it is only where two or more faults occur close together that tunnelling conditions deteriorated significantly. A number of seams of crushed rock and clay less than 4 inches wide were exposed in the tunnel; they were generally caused by local adjustment of blocks of rock and occur predominantly in the interbedded sandstone and siltstone sequences. Such seams generally occur along bedding planes or on prominent joint planes.

Two of the major faults exposed on the east bank of the dam foundations can be correlated with faults mapped in the tunnel. The 60° fault which crops out high on the eastern foundations (fault C in Plate 5) correlates with a 6-inch zone of crushed rock and clay which was exposed in the tunnel between chainages 545 and 572 feet; the rock on both sides of the fault at tunnel level is fresh laminated siltstone. The 40° fault on the east bank (fault A) is correlated with a prominent contact between sandstone and siltstone which was exposed between chainages 480 and 510 feet. Although a significant fault was not evident, the contact cuts across the bedding, dips at 35° to 40° to the south-west, and is on the projected strike line of fault A at tunnel level.

Extrapolation of the 65° fault on the east bank (fault B) southwards along strike indicates that the fault should have been intersected by the tunnel upstream from the shaft junction. No faults were mapped in this section of the tunnel, however, and it is assumed that the fault revealed in the tunnel between chainages 250 and 400 feet has displaced fault B. No other faults intersected by the tunnel can be correlated with surface mapping.

## Engineering geology

The engineering geology of the tunnel is summarised in Plate 13, which shows graphically the geological conditions encountered and their relationship to engineering factors, such as drive rate, support, amount of explosives used, and overbreak.

The drive rate does not show any correlation with geological conditions, and it is evident that there were no serious delays caused by adverse rock conditions. Tunnelling was generally faster in unsupported sections, as expected, and the slower-than-average progress indicated between chainage 227 and 327 feet was due to the larger tunnel diameter for 72 feet of tunnel near the shaft junction.

The plot of the amount of explosives used per foot of tunnel also shows no significant correlation with geology. Apart from the higher charges necessary in the larger diameter section near the shaft junction, only two correlations can be made; the low charges required between chainages 856 and 871 feet were the result of penetration of a faulted block of siltstone, and the higher charges used between chainages 1163 and 1210 feet were probably necessary because of excavation in thickly-bedded sandstone.

It was not possible to make any reliable estimate of overbreak during tunnel excavation because the steel sets and timber lagging obscured most of the roof. However, it has been possible to calculate overbreak, using the records of concrete placement in the arch section of the tunnel. For each concrete pour of the tunnel arch, the volume of rock excavated outside the final 12½-foot diameter lined tunnel was calculated by adding the estimated volume of timber lagging left behind the steel sets to the volume of concrete placed (4 cubic feet of timber per foot of tunnel was allowed for lagging). The overbreak was then calculated by subtracting from this figure the theoretical volume of rock between the tunnel arch form and the peripheral holes of the drilling pattern.

The calculated values of overbreak are shown in Plate 13 (the units are cubic yards per foot of tunnel). High values were obtained between chainages 314 and 572 feet, where the tunnel intersects a number of faults striking nearly parallel to the tunnel alignment, and also in two sections towards the outlet end of the tunnel. From chainage 1021 to 1054 feet, a prominent joint set dipping 45° north-east was noted on the tunnel log as causing considerable overbreak, and this is reflected in the calculated value. The other tunnel section with high overbreak, between chainage 1163 and 1210 feet is excavated in openly-jointed sandstone which is affected by several minor faults.

The support used during excavation of the tunnel has been the subject of considerable discussion between the contractor and the client. A total of 229 steel sets were used for support, although the contract specifications made provision for payment for only 50 sets. The contractor subsequently entered a claim for payments to cover the cost of extra excavation and concrete placement required by the use of steel sets throughout most of the tunnel; the claim was based on his assessment that the support was necessary because of bad tunnelling conditions. The following considerations based on the geological log of the tunnel excavation, are relevant to the question of optimum support requirements.

1. The contrast between the sections of the tunnel supported with steel sets at spacings of 5 feet or less and the 230 feet of tunnel not supported by steel sets should be noticeable, particularly as very few rock bolts were necessary in the unsupported areas. This contrast is lacking in all of the geological features noted on the log sheets (such as rock type, rock classification, breakage along joints, joint systems).

2. The lack of contrast in the assessment of rock classification is brought out in Plate 13 by showing the plot of rock classification which would require the support actually used (dashed line). This plot shows that the rock condition as noted on the tunnel logs generally shows better quality rock than is indicated by the support actually used. In fact, the only areas where the assessed rock condition is significantly lower than the condition indicated by actual support occur in the sections of the tunnel not supported by steel sets; this indicates that the geologist's assessment of rock condition is generally conservative. On the basis of the actual rock classification and thegeological logs, only about 250 feet of tunnel required support with steel sets.

3. The contrast between the sections of tunnel supported by steel sets and the virtually unsupported sections should be indicated by substantially lower explosive charges being required to blast each foot of tunnel in rock so openly-jointed and faulted that steel sets are required. This contrast is not at all evident in the explosive charge plot in Plate 13.

It is concluded that the geological conditions in many sections of the tunnel supported by steel sets did not differ appreciably from conditions in areas which were either totally unsupported or supported by rock bolts at very wide spacings. As there were no stability problems in the sections where steel sets were not used, it is apparent that steel sets were not necessary in other sections of the tunnel with similar geological conditions.

## CONSTRUCTION MATERIALS

### Rockfill quarry

The quarry for rockfill is situated just under a mile upstream from the dam, on the eastern side of the Cotter valley (see Plate 3 and Figure 4). It is located on a spur between two small creeks, 800 feet apart, and the quartzite sequence, which provided the bulk of the rockfill material, originally cropped out as a prominent cliff 140 feet high. The area was topographically well-suited for the development of a deep quarry, and geological investigations proved that adequate quantities of suitable rockfill material could be obtained from the one site.

The quarry was worked by means of five benches, each nominally 40 feet high. Explosive was placed in holes drilled from the surface, or the bench above that being worked, at an inclination of 4 vertical: 1 horizontal, to a depth of 40 feet. Holes were spaced 12 to 18 feet apart, and three hole sizes were used - 6 inch, 4½ inch and 3 inch diameter. The average rate of drilling, based on the whole operation, was as follows:

| | | | |
|---|---|---|---|
| Crawlmaster drill | - | 11.9 | feet per hour of drilling |
| Joy - Rand drill | - | 30.6 | " " " |
| Joy drill | - | 21.9 | " " " |

Figures on bit wear are not available.

Ammonium nitrate, with Quilox and diesoleum, was the explosive used, and A.N. 60 gelignite and Cordtex was the priming charge. The average explosive factor - pounds of explosive per ton of rock broken - for the whole quarry operation was 0.75. The rock breakage factor is similar to results obtained in recent Snowy Mountains' quarry operations of comparable magnitude. So little secondary breaking of rock was needed that none was recorded.

### Geology

The stratigraphic succession of the quarry area is the same as that which was exposed in the spillway overflow channel area. The oldest sequence exposed consists of well-bedded quartzite at least 120 feet thick, which is overlain by 90 feet of thickly-bedded quartzite; a sedimentary breccia bed between the two sequences confirms the correlation with the succession at the damsite. A sequence of interbedded sandstone and siltstone overlies the thickly bedded quartzite, and grades to dominantly laminated siltstone at the top of the exposed succession (see Fig.21).

The quarry is located on a broad syncline, with a gently dipping west limb and a moderate to steeply dipping east limb. Dips on the west limb are regular, but on the east limb, considerable tight folding and associated faulting has taken place. Exposures during quarrying operations have confirmed the structural interpretations made during the design investigation; the fold axes plunge at about $10^{\circ}$ to the south, and the axial planes dip at $50^{\circ}$ to the east.

Three major faults are present in the quarry area, only one of which was located during the design investigation. The known fault was located by a bulldozed trench along the western margin of the quarry area, where the sequence of quartzite is abruptly terminated by a faulted contact with quartz porphyry; the fault dips at $50^{\circ} - 70^{\circ}$ to the east. As the quartz porphyry exposed in the trench was seen to be sheared and deeply weathering, the contact marks the western limit of quarrying.

The most significant fault from the quarrying viewpoint crops out along the northern margin of the quarry. The presence of a fault in this area was strongly suspected during the investigation, because of the abrupt termination of the cliff outcrop; as this coincided with a prominent gully, it was assumed that the surface trace of the fault followed the gully. However, the fault was exposed in the proposed quarry area during stripping of overburden; it dips at $45^{\circ} - 50^{\circ}$ to the north, and has a normal displacement of about 150 feet. The attitude and displacement of this fault had an important bearing on the development of the quarry.

The third major fault was exposed along the southern margin of the quarry at a late stage of the quarrying operations. It dips at $45^{\circ} - 50^{\circ}$ to the north, and has a normal displacement. Only the well-bedded quartzite sequence is exposed to the south of the fault plane, so it is only possible to calculate a minimum amount of displacement, which is 140 feet. The fault was of no engineering significance, as very little rock was quarried from the footwall of the fault.

Geological plans of the quarry area, both before and after quarrying, are shown in Plate 14; these are accompanied by two geological sections which show the distribution of rock types and the geological structure. The information in this Plate is necessarily generalised, because of the impossibility of establishing control for detailed mapping during quarrying operations. In particular, the faults which are associated with the tight folds on the east limb of the main anticline are not represented on the geological plans or sections.

Development of quarry:

The first stage in the development of the quarry was the removal of all overburden and the laminated siltstone preserved in the cores of the synclines. It was estimated from the investigation that excavation to depths of up to 80 feet would be necessary in places before suitable rockfill material was exposed; this was verified during the stripping which continued until sandstone beds were encountered at depth. Most of the siltstone could be removed by ripping, but blasting was necessary in the deeper pockets where the siltstone was fresh and tightly-jointed. The total volume of overburden and waste rock removed from the quarry was 136,000 cubic yards.

During the stripping of overburden in the northern part of the quarry area, the fault plane at the northern end of the quartzite cliff was exposed. To the north of the fault trace, weathered siltstone was exposed, and it was evident that the source of suitable rockfill material would be at a considerable depth over much of this area. The attitude and throw of the fault was readily determined where it displaced the western limb of the main syncline. However, on the eastern limb, the


Fig. 21 :- Panoramic view of the quarry looking east from the ridge of quartz porphyry left along the western quarry boundary. The structure and geological succession are indicated by annotations - see Plate 14 for the reference. This panorama was taken at a late stage of the quarry development, and working of the fifth bench is well-advanced.

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trace of the fault was obscured by subsidiary folding and faulting, and drilling was necessary to locate the top of the interbedded sandstone and siltstone sequence in the region of the inferred easterly extension of the fault. Trial tests with a percussion drill proved that sandstone could be differentiated from siltstone by a much slower penetration rate, and nine percussion holes were subsequently drilled to depths of up to 80 feet. From the information obtained by drilling and geological mapping, the northern limit of the quarry area was revised as shown in Plate 14.

The revision of the quarry limits reduced the surface area to such a degree that insufficient rockfill material was available above the proposed quarry floor level of 3150 feet; the relative merits of extending the quarry boundary to the south-east and lowering the quarry floor were therefore considered. The presence of rockfill material to the east had been adequately proved during the investigation. However, to the south-east there were no outcrops or other positive geological information, although extension to the south of the geological structure exposed at the quarry indicated that suitable rockfill material would be present. On the other hand, excellent rockfill material was known to be present in the quarry area below the 3150-foot level. Calculation of the volume of solid rock required for the rockfill zone of the dam, assuming a bulking factor of 1.27,* indicated that lowering the floor of the quarry 20 feet to R.L. 3130 feet would provide sufficient rockfill material within the revised quarry limits; this was therefore the solution finally adopted. Five benches, each nominally 40 feet high, were worked.

Full-scale quarrying operations started in November, 1966, when the first and second benches were developed to provide rockfill for the cofferdam. All rockfill was derived from interbedded sandstone and siltstone, and the grading was close to the minimum acceptable limits - some loads of rock were in fact rejected because of excess fines. After river diversion, the downstream rockfill zone foundations were prepared, and as 100% quartzite rockfill was required for the bottom 30 feet of the dam (see Fig. 2), a quarry face was developed in the thickly bedded quartzite. This was the beginning of working of the third bench at the quarry, and after this, there were no problems in obtaining suitably-graded rockfill. Whenever poor quality material was encountered, mixing with loads of rock from the thickly-bedded quartzite upgraded the rockfill to the desired specification.

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* The bulking factor is the ratio placed volume: solid volume, where placed volume is the final volume of the rockfill after it has been placed and compacted in the dam, and the solid volume is the original *in situ* volume of the rock before blasting. Sometimes the bulking factor is calculated as a percentage, i.e.

$$\frac{\text{placed volume} - \text{solid volume}}{\text{solid volume}} \times 100\%$$

The bulking factor for rockfill dams generally ranges from 1.18 to 1.36, and it was considered that the median of this range would be a conservative estimate for the Corin Dam quarry.

The calculation of the quarry floor level was based on an assumed bulking factor and assumed batters for the walls of the excavation. Checks on the validity of the assumptions, as quarrying progressed, were possible when the dam and the quarry were surveyed in order to calculate progress payments to the contractor. Calculations of volumes were based on survey levels on a 20-foot grid covering the dam and the quarry, and the survey information was fed into a computer programme to determine the volume of rock quarried and the volume of rock placed at the time of the survey. At one stage during construction, it appeared that there was not going to be enough rock in the quarry. Development of the benches had reached the stage where it was impracticable to further lower the quarry floor, and tentative plans were made to quarry rock outside the accepted limits. However, the bulking factors determined from the calculated volumes between successive surveys varied so widely as to be inexplicable by variations in the quality of quarried rock. An error was subsequently found in the computer programme for calculating the volume of placed rockfill, and when this had been corrected it was evident that sufficient rock was available within the planned quarry limits. In fact, the final overall bulking factor was 1.34 and final batters of the quarry excavation were a little steeper than allowed for; this has resulted in about 40,000 cubic yards of available rock being left in the fifth bench of the quarry.

Suitability of material

Except for some marginal rock at the top of the interbedded sandstone and siltstone sequence, the rock from the quarry has been more than adequate in fulfilling the requirements of suitable rockfill material. The proportion of siltstone in the upper beds of the interbedded sequence was sufficiently high for excess fines to be formed, but mixing with good quality rock reduced the volume of rejected material to an insignificant amount.

As expected, the quarry produced rock of adequate durability and grading for use as rockfill. However, the engineering properties of the rockfill, and the economical operation and development of the quarry, have been considerably enhanced by particular geological features of the quarry area. These features include:-

(1) different ranges in spacing of bedding planes in the three main rock sequences. This has greatly facilitated the grading of the dam into the four main rockfill zones (see Figs. 2 and 3). In the interbedded sandstone and siltstone, the spacing of bedding planes ranges from two inches to 5 feet, but mostly less than 3 feet; it therefore provided rockfill for zones 3a and 3b. The main quartzite bed has bedding planes which are so widely-spaced that the size of rock fragments was determined mainly by joints and amount of explosives used; it provided the bulk of material for zones 3c and 3d. The well-bedded quartzite has bedding plane spacings between 2 inches and 2 feet; it provided rockfill for zones 3a and, to a lesser extent, 3b.

(2) well-jointed rock caused by folding and faulting. Even at depth, joints are well-developed and this has greatly facilitated breakage of the rock during blasting. In the main quartzite bed, the joint spacing is generally large, and fracturing across the texture of the rock was necessary to provide convenient-sized rockfill. However, in rockfill from the interbedded sequence and the well-bedded quartzite, many of the rock fragments are bounded entirely by joint planes.

(3) the lack of a dominant joint set or other prominent plane of parting in the quarried rock, which has resulted in irregular, roughly equi-dimensional rock fragments. Apart from increasing the bulking factor, and thereby reducing the volume of quarried rock, this factor has also increased the interlock of the rockfill fragments and the overall strength of the rockfill zone.

(4) the folding of the geological succession. The benches of the quarry were developed in such a way that different rock sequences of the geological succession were exposed in the same face; this enabled working of the most suitable rock sequence in the quarry to provide rockfill for any particular zone in the dam. In the earlier stages of quarry development, this technique enabled marginal quality rock to be uograded by mixing with good quality rock from a different section of the same face. The effect of folding is well-illustrated by the sections in Plate 14. If the quarry had been worked as a series of north-south faces, it can be seen from section XX' that at any one time, a very limited section of the geological succession would have been exposed in any one face. For a series of east-west faces, however, section YY' shows that a large portion of the geological succession would be exposed in each face. While it was not practicable to work straight east-west faces at the quarry, the benches were developed simultaneously to the east and north (east and south in the fifth bench) so that the same effect was obtained.

On the other hand, the presence of pyrite (see Section on concrete aggregate) which is apparently oxidising in the rockfill has produced highly acid waters that are emerging from below the dam (see last two paragraphs of section on leakage from the reservoir). The emerging waters are highly aggresive; their source and affect are currently (March, 1969) being studied.

## Quantities.

The total volume of solid rock quarried for rockfill was 1,032,600 cubic yards, and the total volume of stripping (overburden and laminated siltstone) was 136,000 cubic yards.

The final volume of the rockfill zones in the dam is 1,359,400 cubic yards; this is made up of 166,500 cubic yards in zone 3a (12.2% of total), 572,300 cubic yards in zone 3b (41.9%), 610,800 cubic yards in zone 3c (45.2%), and 9,800 cubic yards in zone 3d (0.7%).

## Core material *

The core borrow area is situated on the western side of the Cotter valley, $\frac{3}{4}$ mile upstream from the damsite (see Plate 3 and Fig. 4). It is located on a broad spur between two creeks, and is entirely below top water level of the reservoir. The bedrock consists of deeply-weathered quartz porphyry, which is overlain by up to 20 feet of slopewash material.

The borrow area was delineated during the investigation by systematic pitting and augering. Laboratory tests proved that both the slopewash material and the weathered porphyry would be suitable core material; the tests also indicated that the slopewash material would be better than the weathered porphyry, and it was intended that the porphyry would not be used in the dam unless there was insufficient slopewash. However, difficulty was experienced by the contractor in adequately compacting the slopewash material in the dam, and after using some of the porphyry, he indicated that recompaction and the associated delays would be reduced to a minimum by continued use of porphyry in the core. As the physical properties of the weathered porphyry are adequate for the design requirements, it was ahreed that porphyry could be used; 70% of the completed earth core is composed of weathered porphyry.

The total placed volume of core material is 342,200 cubic yards.

## Filter material *

All material for filter zones in the dam was obtained from river flats within a mile upstream of the dam. The upstream filter zone is composed of unsorted alluvium from these flats, but as the grading for the downstream zone is more critical, sorting and remixing of the alluvium was necessary. However, as the natural grading of the alluvium was generally close to the specified grading, there was very little waste material from the processing.

The total placed volume of filter material is 116,400 cubic yards.

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* Geological services in the investigation of core and filter material were generally restricted to advice about possible sites of suitable material. During construction, quality control and placement supervision was in the hands of Department of Works' officers.

## Concrete aggregate

It was hoped that material suitable for coarse concrete aggregate could be obtained from the quartzite beds at the rockfill quarry. However, visual inspection of drill core obtained in the course of the quarry investigation revealed the widespread occurrence of scattered pyrite in rocks in all parts of the geological succession. It was considered inadvisable to counteract the deleterious properties of pyrite by using sulphate-resistant cement; so petrographic tests were therefore undertaken to determine the amount of pyrite in the rock.

Drill hole D.D.35 was selected for testing, and cores at about 20-foot intervals were submitted for examination in reflected light. The cores were cut with a diamond saw, and the percentages of pyrite (readily identifiable in reflected light without polishing) in the exposed faces were determined using a point-counter. The following results were obtained.

| Depth | Rock type | Percentage of pyrite |
|---|---|---|
| 40' 6" | Interbedded sandstone and siltstone | approx. 1.0 |
| 67' 0" | Thickly bedded quartzite | 1.42 |
| 88' 0" | Thickly bedded quartzite | 0.50 |
| 109' 8" | Thickly bedded quartzite | 1.50 |
| 129' 9" | Thickly bedded quartzite | 1.26 |
| 144' 4" | Thickly bedded quartzite | 0.50 |
| 164' 6" | Well-bedded quartzite | 0.42 |
| 194' 6" | Well bedded quartzite | 0.08 |
| 210' 0" | Well-bedded quartzite | 0.05 |

The results show that the coarse grained, thickly-bedded quartzite, which would provide the best concrete aggregate, has an average pyrite content of 1.2% by volume. Although standard references on concrete technology do not give a figure for the maximum allowable content of pyrite, it was evident that far too much pyrite is present in the rockfill quartzite for it to be suitable for use as concrete aggregate. The well-bedded quartzite judging from the samples tested, has a lower pyrite content, but it was impossible to obtain suitable rock at the early stage of construction that concrete was required. The contractor therefore elected to obtain coarse concrete aggregate from the Mugga quarry at Canberra.

A source of fine aggregate (particle sizes less than 3/16 inch) was located in the large river flat immediately upstream of the dam. Treatment would have been necessary, however, and the contractor preferred to use washed sand from the Murrumbidgee River at Point Hut Crossing, 20 miles away.

LEAKAGE FROM THE RESERVOIR.

Three general areas of possible leakage were considered and evaluated during the investigation; they were -

(1) from the reservoir rim away from the damsite,
(2) along the Cotter Fault zone, and
(3) through the dam foundations.

Reservoir rim.

The only area in which leakage could conceivably take place is the Kangaroo Creek - White Sands Creek divide, within 3,500 feet of the damsite. In this area, a maximum theoretical head of 200 feet could act on a leakage path only 2,000 feet long. However, for leakage to occur, the rock must be openly jointed or fractured to a depth of at least 400 feet below the top of the ridge. Detailed geological mapping and several seismic traverses were carried out, and supplementary trenching was undertaken to expose bedrock in low velocity zones. It was concluded that serious leakage in this area would not occur. Observation drillhole DD11, 470 feet east-north-east of the eastern end of the dam crest, has provided information on the effect of the filling reseevoir on the groundwater levels in the western part of the divide - see Appendix 4 and the sub-section on leakage through the dam foundations.

Cotter Fault zone

The Cotter Fault crops out extensively in the reservoir area, the nearest exposure to the dam being only 900 feet from the spillway channel; it crops out again in the river a further 1,100 feet downstream. If continuous open jointing extends to a depth of up to 300 feet, it is possible that the full head of the reservoir will act on a leakage path 2,000 feet long.

The trace of the Cotter Fault zone was mapped in detail during the feasibility investigation, and an evaluation of the possibility of leakage was made (Oldershaw, 1965); in addition, a drill hole (D.D.6) was drilled through the main fault zone. It was found that the Cotter Fault truncates a system of parallel faults, trending north-northwest, which have considerable vertical displacements (the fault traversing the west abutment and spillway is one of this system). In the area upstream of the dam, faulted blocks of quartzite and quartz porphyry were mapped, and at least two faulted blocks of quartzite are present adjacent to the Cotter Fault in the downstream area. From the results of water pressure testing in drillholes D.D.6 and P-2, and from observations of sections of the fault zone exposed during construction of haul roads, it is apparent that the fault zone itself consists of closely sheared, tight rock. A leakage problem only exists if the faulted blocks of quartzite and porphyry are present as an interconnected series of openly-jointed blocks of rock. It was recognised that the reservoir might leak along the Cotter Fault zone, but unacceptable leakage was considered to be so unlikely that the grout curtain was not extended into the phyllite on the western side of the fault zone. As a precaution, an additional water table observation hole was drilled and cased during construction and systematic water level measurements were started as soon as the reservoir began to fill.

The hole, C.F.1, is about 50 feet east of the Cotter Fault and is 600 feet north-north-west of D.D.5 (see Plate 4). It was anticipated that any significance leakage through the quartzite adjacent to the Cotter Fault would be reflected in the gradient of the water table between D.D.5 and the new hole as the reservoir filled; remedial grouting can be carried out, if necessary, once the reservoir is fully operational.

Water levels in the observation holes are recorded in Appendix 4, together with levels measured in investigation holes. Holes D.D.5 and C.F.1 in March, 1969, had hydraulic gradients from the reservoir (which at that time was at R.L. 3099 and had been within two feet of that level for more than three months) of 1:7.4 and 1:7 2. Despite the fairly steep gradient indicated by the observation holes springs have appeared a short distance east of the fault, and 1100 feet downstream from the reservoir. Their proximity to the fault zone suggests that the Cotter Fault and/or associated faults provide the leakage path. The springs were first noticed in November, 1968; flow has not been gauged but it appears to be steady at no more than one or two cusecs. Provided rate of flow does not increase substantially no remedial action is necessary.

Dam foundations

All of the exploration holes drilled at the damsite were water pressure tested to assess the amount and extent of grouting necessary to reduce leakage to within acceptable limits. Moderate to high water losses were recorded during testing, and special techniques and modifications were necessary to obtain reliable results. The corrected water pressure test results were used to calculate joint permeabilities, which were then plotted on a section across the damsite. (Plate 17 of Best, 1965). The results showed a pronounced difference in permeability between openly-jointed and tight rock; it was recommended that the grout curtain be extended 30 feet into tight rock as shown in the section.

Joint permeabilities calculated from tests in the openly-jointed rocks were high (many of the calculated values were greater than 1,000 feet per year); it was therefore concluded that grout consumption would generally be moderate to high. The interpreted depth to tight rock was greater on the west bank than on the east bank (90 to 170 feet compared with 40 to 80 feet). The contrasting extent of open jointing on the two sides of the river is illustrated by the groundwater levels recorded, during 1964 amd 1965, in drillholes D.D.5, D.D.10, and D.D.11 (see Appendix 4). On the west bank, the groundwater level ranged between 20 and 50 feet above the river in D.D.5 (which is 800 feet from the river); on the east bank, the groundwater level varied between 100 and 125 feet above river level in D.D.10 (400 feet from the river) and between 90 and 115 feet above river level in D.D.11 (800 feet east of the river). It is therefore apparent that the west bank foundations are generally more permeable than those of the east bank.

Leakage of water through the foundations was controlled by blanket grouting and curtain grouting (the results of grouting are discussed in the next section). The main purpose of blanket grouting was to prevent water leaking through the foundations to the base of the earth core. In spite of a close spacing of blanket grout holes, it was felt that the blanket of grouted rock was by no means 100% effective, and rigorous foundation treatment was carried out to make up for any deficiencies. The purpose of the grout curtain is to limit the leakage of water between the reservoir and the valley immediately downstream of the dam to a negligible amount. The curtain below the earth core was generally extended to a depth of 150 feet by the primary and secondary pattern of holes, though over much of the west bank the grout holes were extended down to 175 feet. The tertiary grout holes generally penetrated to a depth of 100 feet, and the quaternary ~~pattern~~ to 50 feet. The curtain therefore is generally deeper than appeared necessary from the water pressure test results, particularly on the east bank. There has been a general change in philosophy on grouting over the past 3 or 4 years, and it is accepted that the site is over-grouted on the basis of present day criteria.

The recent change in grouting criteria is illustrated by the grout curtain extension below the spillway crest, which was grouted at a late stage of the construction period. The spillway crest is founded on openly-jointed quartzite in which the joints are open down to a depth of about 200 feet. The spillway crest is also on the shortest possible leakage path from the reservoir, and the grout curtain below the crest is the only barrier to leakage, which could incidentally damage the concrete spillway structure by uplift. In this critical situation, rough calculations were made of the likely leakage rate, assuming that openly-jointed rock with a joint permeability of 10,000 feet per year is present to a depth of 250 feet. Assuming a perfect grout curtain only 100 feet deep, the leakage rate was calculated as 0.25 cubic feet per second (cusec); for a 50-foot deep curtain, the leakage rate would be 6.4 cusecs. The maximum capacity of the drainage system below the spillway, by comparison, is 8 cusecs. It was therefore decided that primary curtain holes would be 175 feet deep, secondary holes 140 feet deep, tertiary holes 90 feet and quaternary holes 35 feet deep. Thus, although the grout takes in the foundations of the spillway crest were by far the largest recorded for any section of the curtain line (see Plate 15), the density of grout holes below the spillway crest is slightly lower than elsewhere in the grout curtain.

Despite the generally rigorous treatment of the dam foundations water began to emerge from a spoil dump below the dam *(as springs which flow into the outlet channel 40-250 feet downstream of the outlet portal)in June, 1968, when the reservoir depth was only 80-90 feet. The initial flow was only a fraction of a cusec but increased as the reservoir filled. At the time of writing (March, 1969), when

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* The spoil dump was not included in the design of the dam. It was placed against the downstream toe of the dam as a convenient means of disposing of waste material and, at the same time, landscaping the area. The dump is up to 15 feet thick and has a soil-covered, level, surface about 300 feet by 250 feet; it consists of both rock and soil.

the reservoir level is at about R.L. 3099, the flow is very roughly one cusec and does not appear to have altered greatly in the last few months. A feature of this water is its high acidity; the pH has ranged between 3.1 and and 4.0, and is now 3.8. The spoil dump has masked the source of the water but the volume is too great and the rate of flow too consistent for rainwater or shallow groundwater. Water also drains into the outlet tunnel, both upstream and downstream of the grout curtain; the very diverse chemistry of water emerging into the tunnel at various points indicates a variety of sources. However, none of it is as acid as that in the spoil dump spring; it therefore seems likely that the acidity of the spoil dump spring is mainly due to the oxidation of pyrite in the rockfill of the dam. Investigations are continuing.

A separate report will be issued on the water emerging downstream of the dam, from the outlet tunnel, the spoil dump and the Cotter Fault zone.

Observation hole D.D.11 showed a hydraulic gradient from the reservoir of 1:12.4 in March, 1969, indicating moderate permeability between the reservoir and the hole; some reservoir water is therefore probably leaking through the right abutment.

## GROUTING

### GENERAL GROUTING PLAN

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

(i) "A" hole curtain grouting, consisting of high pressure grouting in deep holes, to reduce seepage under the dam.

(ii) "B" hole blanket (or consolidation) grouting, consisting of low pressure grouting of shallow holes, to minimise leakage of water through the foundations of the core zone; blanket grouting also strengthens the foundations by cementing open joints and cracks.

### Curtain grouting

The grout curtain consists of a single line of holes under the impermeable core, upstream of the dam axis, with wing extensions on both abutments; on the east bank, the curtain extends for 120 feet horizontally beyond the dam crest, while on the west bank it extends under the spillway crest (see plan in Plate 15). On the east bank the grout curtain was supplemented by grouting from the tunnel.

The nominal layout of the curtain grout holes consisted of 150-foot deep holes at 20-foot centres (slope distance), with intermediate holes 100 feet deep (also at 20-foot centres); this pattern was then bisected with 50-foot holes at 10-foot centres, giving a nominal spacing between holes of 5 feet. The nominal orientation of the holes below the embankment was at right angles to the average slope of abutments. Holes at the foot of each abutment were progressively steepened as the bedrock surface slope flattened, and holes in the valley floor were vertical. In practice, the angle of grout holes towards the top of the west abutment was changed to vertical to provide better intersections with steeply-dipping joint planes. The western extension of the grout curtain consists of vertical holes as far as the northern end of the spillway crest, where the orientation of the grout holes was changed to coincide with the plane of the crest stressing cables (i.e. at 65° in a downhill direction at right angles to the crest along the eastern crest, and vertical along the southern crest). The curtain extension ends as a fan of grout holes in the western wall of the spillway excavation. Beneath the spillway crest, the nominal spacing of grout holes was reduced to 4½ feet so that the cable holes could be used for grouting.

The spacing, depths, positions and orientations of the holes actually drilled and grouted are shown in Plate 15. The total number of holes drilled to construct the grout curtain below the dam was 292 and the total length grouted was 25.757 feet.

Blanket grouting.

Blanket grouting was carried out over the entire foundations for the core and downstream filter zone. Holes were planned nominally on a 10-foot grid with one axis parallel to the line of curtain holes and the other axis parallel to the dam grid north-south line. The holes were 25 feet deep in the area from 30 feet upstream of the curtain line to 50-feet downstream; beyond these limits the holes were 15 feet deep. Generally, the blanket holes,were drilled normal to the average slope of the abutments. However, if such an orientation was nearly parallel to a set of prominent joints, the direction of the holes was changed to obtain more intersections with the open joints, and thereby increase the effectiveness of grouting. All holes to the south-west of the main anticline on the west bank were drilled at 70° in a north-easterly direction to ensure that all of the steeply-dipping bedding plane joints were intersected by several grout holes. A number of holes in the valley floor and on the east abutment were also re-oriented to intersect a greater number of bedding plane joints.

The number of blanket grout holes in the embankment area was 1,034, plus 36 holes below the spillway retaining wall footings; the total length grouted was 23,098 feet.

## PRACTICAL DETAILS OF GROUTING

### Drilling

All blanket holes were drilled with Atlas-Copco percussion drills, attached directly to the stand-pipes of the holes; 1⅞" diameter bits were used throughout. Percussion drilling was also used, wherever possible, for the curtain holes. However, the east abutment was too steep and irregular for percussion drilling equipment, and curtain holes between eastings 635 and 998 (dam grid) were diamond drilled using EX bits (1⅜" diameter). Percussion holes along the curtain line were drilled with 2½" diameter bits, except for the eastern wing extension (where 2" bits were used) and the spillway crest;

along the crest, the holes for the stressing cables were 3" in diameter, while the intervening holes were either 2" or $2\frac{1}{2}$" across, depending upon the rig used to drill the holes.

Washing and water tests

Before water testing and grouting, all drilled stages were washed by water circulation to remove drill cuttings, rock flows, clay and other loose material from joints in the rock adjacent to the hole. Washing was continued until the discharge water was clear. No attempt was made to wash from one hole to another.

All stages were water tested for a period of 10 minutes at a pressure based on the depth of the stage (1 pound per square inch per foot depth, calculated for the bottom of the stage being tested ); exceptions to this rule were the second stages of the 15-foot deep blanket holes, which were tested at 25 p.s.i. The main purpose of water testing was to obtain an indication, by comparison with other stages previously grouted, of the quantity of grout likely to be used, and the grout mix best suited to the conditions. In shallow stages, water tests also revealed surface leaks which could be caulked before grouting, to minimise leakage of grout.

Pattern of grouting

The closure method was used for both the blanket and curtain grouting. For the blanket, primary holes were drilled and grouted at 20-foot centres on the grid layout (defined in an earlier section). Intermediate holes in the centre of each parallelogram formed by four primary holes were then grouted, followed by closure holes to complete the nominal 10-foot-grid spacing. Where intermediate holes took less than 0.25 cubic feet (cu.ft)* of cement per foot of hole, the closure holes were omitted. On the other hand, where closure holes took more than 1 cubic foot of cement per foot of hole, additional holes were grouted; this was seldom necessary. Downstage grouting in two stages (0 to 10 feet and below 10 feet) was used in most blanket holes.

Primary holes for the grout curtain were drilled and grouted at 40-foot centres (36 feet along the spillway crest), after which secondary holes were drilled, midway between the primary holes, and then grouted. The tertiary holes were drilled and grouted midway between the combined primary and secondary pattern, thus reducing the spacing between grouted holes to 10 feet. Quaternary closing holes were generally necessary, the criterion being that if the last hole grouted had a stage with a consumption greater than 0.25 cubic feet per foot of hole, a closing hole was required. The downstage method of grouting was used throughout, except along the eastern wing extension which was upstage grouted, using packers. Almost all stages of the curtain grouting were 25 feet long.

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* 1 cubic foot of cement is equivalent to 1 bag.

Injection pressure:

The gauge pressures used throughout the blanket grouting were 10 p.s.i. for the first stage and 25 p.s.i. for the second stage.

The gauge pressures for curtain grouting were based on 1 p.s.i. per foot depth to the bottom of the stage being grouted. Exceptions to this rule were that stages at a depth greater than 150 feet were grouted at 150 p.s.i.; further, some holes in the valley floor were limited to a maximum of 100 p.s.i., owing to surface leaks and connections. The curtain holes drilled from the tunnel formed a radial pattern and were 40 feet long; they were grouted in two 20 - foot stages. The gauge pressure for the first stage of these holes was 50 p.s.i. and for the second stage 100 p.s.i.

During grouting, full pressures were maintained continuously wherever possible; however, it was sometimes necessary to reduce pressure to enable leaks to be caulked successfully.

Method of grout injection

Grout was injected by the continuous circulation method, using 2-inch ram, double-acting, reciprocating pumps, powered by compressed air. The mixers (80 gallons capacity) and agitators (40 gallons) were provided with rotating paddles for mixing, and the amount of water mixed with the cement was measured with water meters. Pressures were measured by gauges at the pump and grout hole manifold, and the volumes of grout remaining in the tanks on completion of a hole were measured by dip-stick.

Most stages were grouted from the standpipe. However, in a number of curtain holes, grout leaks occurred at the surface during grouting of the first stage. In such cases, and in other holes where it was considered advisable to protect the near-surface rock from high pressures (e.g. the spillway crest), a packer was placed at the bottom of the first stage during grouting of the second and subsequent stages. The use of a packer was also necessary in some of the blanket grout holes to eliminate excessive surface leakage. Pneumatic packers were generally used; a mechanical packer was used on a few occasions.

Leakage of grout from surface cracks was controlled by caulking with oakum, lead wool, wooden wedges and cement fondu. When it was suspected during blanket grouting that grout was travelling over a wider area, or to a greater depth, than was necessary, intermittent grouting was adopted: grouting was stopped when 100 cubic feet of cement had been injected, and after allowing the grout to set, grouting of the stage continued. In several holes, intermittent grouting had no effect, and it was necessary to thicken the grout by the addition of sand.

Grouting of each stage continued until the hole accepted grout at a rate of less than 1 cubic foot of cement in 20 minutes at pressures of less than 50 p.s.i., in 15 minutes at pressures between 50 and 100 p.s.i., or in 10 minutes at pressures greater than 100 p.s.i. The valve at the top of the standpipe was closed on completion of grouting to retain grout in the hole and connected joint systems until pressure dropped or until the grout attained its initial set. Washing of grout out of the hole was permitted 4 hours after grouting when further stages had to be grouted.

In order to avoid connections with ungrouted stages, secondary curtain holes were drilled to follow at least two stages behind primary holes; similarly, tertiary holes were drilled and grouted at least two stages behind secondary holes. The criterion generally used for depth of grouting was that if the planned final stage of a hole took more than 1 cubic foot of cement per foot of hole, a further stage would be drilled and grouted. After several initial deep holes, this criterion was modified for the spillway crest curtain on the advice of Mr.J. Barry Cooke, consultant to the Commonwealth Department of Works; holes were restricted to a maximum depth of 140 feet, regardless of high grout consumption in the lower stages.

A concrete grout cap, 3 feet wide and 3 feet deep, was constructed along the curtain line where it was considered that leakage paths across the curtain line existed near the surface; Plate 8 shows the locations of the grout caps in the core zone foundations. A continuous grout cap was also constructed along the western extension of the grout curtain.

## Composition of grout

The cement used for grouting was Southern Portland Type 'A' Superfine Cement. The water: cement ratio (by volume) used for initial grouting was 6:1, and this was reduced to 4:1 if the grout 'take' exceeded 1 cubic foot of cement in 5 minutes. If the high rate of consumption continued, the mix was progressively thickened to 1:1; this was generally sufficient to reduce the rate of grout consumption. Where high grout takes were continually obtained, even after intermittent grouting, a grout composed of 1 cubic foot of cement, 2 cubic feet of sand and 8 gallons of water was used; this was necessary in only 5 blanket grout holes.

# RESULTS OF GROUTING

## Cement consumption

The total cement consumption during grouting at Corin Dam was 30,536 cubic feet; this was divided evenly between curtain and blanket grouting (15,284 cubic feet and 15,252 cubic feet respectively).

The results of curtain grouting are plotted on a section along the curtain line (Plate 15), which shows the stages of grouting, together with the cement consumption in cubic feet for each stage. To show the results graphically, the stages are classified, by variations in line thickness, into 3 ranges of cement consumption. One obvious feature of the plotted results is the general low consumption for the 0 to 25 foot stages and, to a lesser extent, for the 25 to 50 foot stages; this is a reflection of the effect of grout travel from adjacent blanket holes, which were purposely grouted before the curtain holes. The only other discernible pattern is the correlation of high consumption areas in the valley floor and below the spillway crest with the thickly-bedded, openly-jointed quartzite sequence. The total length of stages for curtain grouting (including the spillway cut-off) was 26,431 feet, made up of 306 holes; the average cement consumption was therefore 0.57 cubic foot per foot of hole.

In the blanket grouting, 15,252 cubic feet of cement was injected into 1,070 holes with a total length of 23,098 feet; this gives an average consumption of 0.68 cubic feet per foot of hole. However, over a third of the total cement consumption was injected into 26 holes which had individual takes of more than 100 cubic feet. It is likely that much of the grout injected into these high take holes leaked beyond the general limits of the grout blanket, and a more realistic figure of average consumption is obtained by eliminating these results. The modified figures show that 9,732 cubic feet of cement was injected into 22,528 feet of hole, giving an average consumption of 0.43 cubic feet per foot of hole.

The detailed results of blanket grouting are not given in this report, as the bare figures are rather meaningless. However, a plan showing the cement consumption for all blanket grout holes has been used as a basis for a statistical classification of the foundation rock according to the amount of cement injected; the results are illustrated in Plate 16 and are discussed in "Analysis of Grouting - Blanket Holes".

Uplift

Five uplift gauges were installed in the core zone foundations, two on the east abutment and three on the west abutment; all gauges were installed in areas where uplift was thought most likely to occur. Displacement was indicated on one gauge only, which was located in a faulted area towards the top of the eastern foundations where a joint system parallel to the foundation surface was present. During grouting of the top stage of the nearest blanket grout hole at 10 p.s.i., a displacement of 0.035 inches was recorded.

During grouting of the second stage, no uplift was indicated at 25 p.s.i., but a further displacement of 0.007 inches was recorded when the pressure was increased to 50 p.s.i. In the few areas of the east bank where similar bedrock conditions were apparent, the pressure for first-stage blanket grouting was reduced to 5 p.s.i.

On the western foundations, extensive areas occur where bedding plane joints in the well-bedded quartzite are nearly parallel to the bedrock surface. Two uplift gauges were installed in such areas, and pressures in adjacent blanket grout holes were deliberately increased by up to 30 p.s.i. above normal grouting pressures; however, no uplift was recorded. Normal grouting pressures were therefore used in these areas of slabby foundations, except in a few holes where conditions were considered to be worse than in the areas tested by uplift gauges; first-stage pressures were reduced to 5 p.s.i. in these holes.

During the curtain grouting of tertiary holes at the foot of the west abutment, some unexpectedly high grout takes were recorded which were probably caused by uplift. Most of the high-take stages were below 100 feet, and were therefore grouted at pressures of 100 p.s.i. or greater. Packers were placed at depths ranging between 25 and 40 feet, and as the stages were part of the tertiary pattern, the full grouting pressure was probably exerted at packer depth. This factor was aggravated in at least three of the stages where a faulty gauge pressure, reading 40 p.s.i. too low, was used. Surface leaks were recorded during grouting of some of these high-take stages below 100 feet, and it is difficult to envisage leakage paths to the surface from such depths which had not previously been grouted by upper stages. Uplift of previously grouted rock seems the most likely explanation for these high takes and surface leaks. In this area, the quaternary holes (generally 50 feet deep) were extended to a depth of up to 150 feet to seal any leakage paths left ungrouted or opened up by the tertiary grouting. There was no evidence of uplift during grouting of any other sections of the grout curtain.

## ANALYSIS OF GROUT CONSUMPTION

In the following analyses, the term 'unit take' is used extensively; this is defined as the cement consumption, in cubic feet, per linear foot of grout hole, and allows direct comparison of grout takes in stages of differing lengths.

### Curtain grouting

#### Correlation with depth and closure pattern

Under normal conditions, it would be expected that the unit takes would decrease with depth below the bedrock surface for each pattern; further, the unit takes should decrease for each successive phase of pattern closure. The following table shows the variation of unit takes with depth and the closure pattern for the entire grout curtain.

| Depth of grouting | Unit takes (cu. ft/lin ft) for closure phases Primary | Secondary | Tertiary | Quaternary |
|---|---|---|---|---|
| 0' - 50' | 0.80 | 0.48 | 0.20 | 0.09 |
| 50' - 100' | 1.53 | 0.63 | 0.31 | - |
| 100 - 150 | 1.40 | 0.65 | - | - |
| Full depth | 1.24 | 0.59 | 0.25 | 0.09 |

It can be seen that the grout consumption progressively decreases with each closure phase for all three of the arbitrary depth ranges, and also for the full depth of the grout holes, However, a significant decrease in grout consumption with increasing depth is not apparent in any of the four phases of grouting. There are two factors which obscure the actual decrease in grout take with depth in the tabulation given above. The main factor is the interference of the blanket grouting with thethe grout consumption figures for the upper stages of the curtain holes. Blanket grouting to a depth of 25 feet was always completed adjacent to the curtain line before curtain grouting, and this partially grouted the rock around the first stage of all curtain holes; in many cases, the second stages were probably partially grouted also. This explains the low unit takes for the 0 to 50-foot zones, compared with deeper zones of the same grouting phases.

The other factor affecting the relative unit takes is the increase in pressure of grout injection with depth. In the 50 to 100 foot zones, pressures of 75 and 100 p.s.i. were used, while in the 100 to 150 foot zones injection pressures were 125 or 150 p.s.i. Bearing this in mind, it is evident that for a standard injection pressure, the average unit take between 100 and 150 feet would be considerably lower than between 50 and 100 feet for the primary holes; it is also evident that for the secondary holes, the deeper stages would have a lower average unit take than the 50 to 100 foot stages, instead of the slightly higher average unit take noted in the table.

Correlation with rock types.

The unit takes tabulated in the previous section are the averages for the entire grout curtain. Because of the irregular distribution of rock types and the complex geological structure intersected by the grout curtain, the figures are not typical for any particular geological sequence. An analysis of the grout consumption was therefore undertaken to determine the variation of unit takes with rock type. This was done by plotting the geological structure and rock type boundaries onto a print of the cross section showing curtain grout takes (Plate 15). From this, areas representing the five main rock types were selected, and then analysed by summing the grout takes and the footage of grout holes in each division. Areas adjacent to major faulting and around the diversion tunnel were excluded from the analysis so that the calculated unit takes reflect the overall jointing of the various rock types. The results of the analysis are given in the following table.

| Rock type and area | Total grout take (cu.ft.) | Total linear footage | Unit take (cu.ft/lin.ft). |
|---|---|---|---|
| Thickly bedded quartzite | | | |
| Spillway crest | 4,581 | 4,195 | 1.09 |
| Thickly bedded sandstone | | | |
| East bank | 1,945 | 2,640 | 0.74 |
| Interbedded sandstone and siltstone | | | |
| Spillway crest | 997 | 1,970 | 0.51 |
| Lower slopes west bank | 819 | 2,425 | 0.34 |
| Valley floor | 568 | 1,200 | 0.47 |
| Laminated siltstone | | | |
| Lower slopes west bank | 44 | 250 | 0.18 |
| Well-bedded quartzite | | | |
| West bank | 835 | 5,980 | 0.14 |

It was not considered necessary to separate primary, secondary, tertiary and quaternary takes in this analysis, as all four phases are roughly evenly represented in each area considered. The figures in the unit take column are therefore indicative of the relative abundance and openness of joints in the various rock types and areas analysed.

The results are, in general, as expected from geological considerations. The hard fresh, thickly-bedded quartzite is openly-jointed to depths greater than 200 feet below the surface, and has the highest unit take. The thickly-bedded sandstone on the east bank is also well-jointed, but the rock is not extensively silicified and is generally more weathered along joints than the quartzite; the unit take is therefore lower than for the quartzite. The laminated siltstone generally has fresh, tight joints and has a low unit take, while the unit takes in the interbedded sandstone and siltstone are intermediate between the unit takes for the sandstone and the laminated siltstone. The well-bedded quartzite with silty laminae has the lowest unit take of all, and this is the only unexpected feature of the analysis. The sequence is closely-jointed parallel to bedding and several other less persistent joint sets are present. The results of water pressure tests carried out during the feasibility investigation were contradictory in that D.D.2 showed low joint permeabilities, while D.D.4 showed very high rates of water leakage (so high that in some test sections adequate back pressures could not be developed). In view of the nature of the exposures in the sluiced areas, it was inferred that this rock sequence would accept moderate to large quantities of cement during grouting. Low grout takes were recorded during blanket grouting, and at first it was assumed to be a near-surface feature caused by weathering and clay infilling along joints. However, the low grout takes persisted throughout the curtain grouting also. It can only be assumed that clay linings are prevalent along the joints, that the water pressure tests in D.D.4 were erroneous in some unknown way, and that the clay linings were washed away during diamond drilling of D.D.2 and D.D.4.

Unit take evaluation.

The systematic evaluation of the results of curtain grouting is generally a difficult and seldom-attempted process, particularly in geologically-complex foundations; usually the qualitative reduction in grout takes during closure is taken as an indication of effective grouting, and an arbitrary cut-off is used to determine the final closure pattern. However, a standard method for systematically summarising and comparing grout curtain unit takes has been proposed, and the analysis can be extended to determine whether the point of diminishing returns has been reached during closure grouting (Grant, 1964). The method appears to be very sound, and I considered that an evaluation of Corin Dam grouting results by this method would be useful, in spite of the complex geological structure of the foundations. Briefly, Grant considered that comparisons of unit takes are valid when the values are plotted for uniform split spacings and similar geological environments. Under these conditions, a plot of unit take against hole spacing will produce uniform grouting curves; when plotted on log-log scale, the curves are, in fact, comparative void reduction curves. Providing that grouting procedures are uniform, the log-log plot should be a straight line, showing unit takes decreasing with closure, until the point of diminishing returns (i.e. grout saturation) is reached, when a change in gradient will be evident for closer hole spacings.

For analysis of the Corin Dam curtain grout takes, the curtain was again divided into areas of similar rock types. In each area, the total grout take and total linear footage of grout holes were calculated separately for primary, secondary, tertiary and quaternary holes. From these figures, the unit takes for the four phases of grouting were calculated for each rock type, and these were plotted on log-log scale ahainst hole spacing. The results, illustrated in Fig.22, show that 4 of the 5 rock type divisions are grouted to saturation. In these 4 cases, the theoretical point of saturation occurs at a hole spacing between the tertiary and quaternary spacings, which is as it should be for economical grouting. The plot for the laminated siltstone does not conform to the general pattern for two reasons: firstly, very few curtain grout holes penetrated the laminated siltstone (total footage for all phases was only 250 feet); secondly, the grout hole stages which did penetrate siltstone were all within 50 feet of the bedrock and were therefore affected to an unknown degree by blanket grouting.

A further indication of the likely efficiency of a grout curtain can be obtained by calculating the reduction in unit void from the ratio of unit takes in primary and closure holes. In his paper, Grant (1964) concluded from a detailed study of curtain grouting at 7 sites that a computed reduction in unit void of 90% or more of the original for the rock mass treated results in an effective grout curtain. The following table shows the calculated values for the main rock type divisions, and for the curtain as a whole, at Corin Dam.

| Rock type | Primary unit take | Terminal unit take | Indicated terminal unit* void | Indicated reduction in unit* Void |
|---|---|---|---|---|
| Thickly-bedded quartzite | 2.01 | 0.11 | 5.5 | 94.5 |
| Thickly-bedded sandstone | 1.41 | 0.05 | 3.5 | 96.5 |
| Interbedded sandstone and siltstone | 0.90 | 0.10 | 11.1 | 88.9 |
| Well-bedded quartzite | 0.58 | 0.08 | 13.8 | 86.2 |
| Laminated siltstone | | (insufficient grout holes) | | |
| Entire curtain | 1.24 | 0.09 | 7.3 | 92.7 |

* expressed as the percentage of original unit void.

The results (Fig.22 and Table above) together show that, by Grant's criteria, an effective grout curtain has been constructed at Corin Dam.

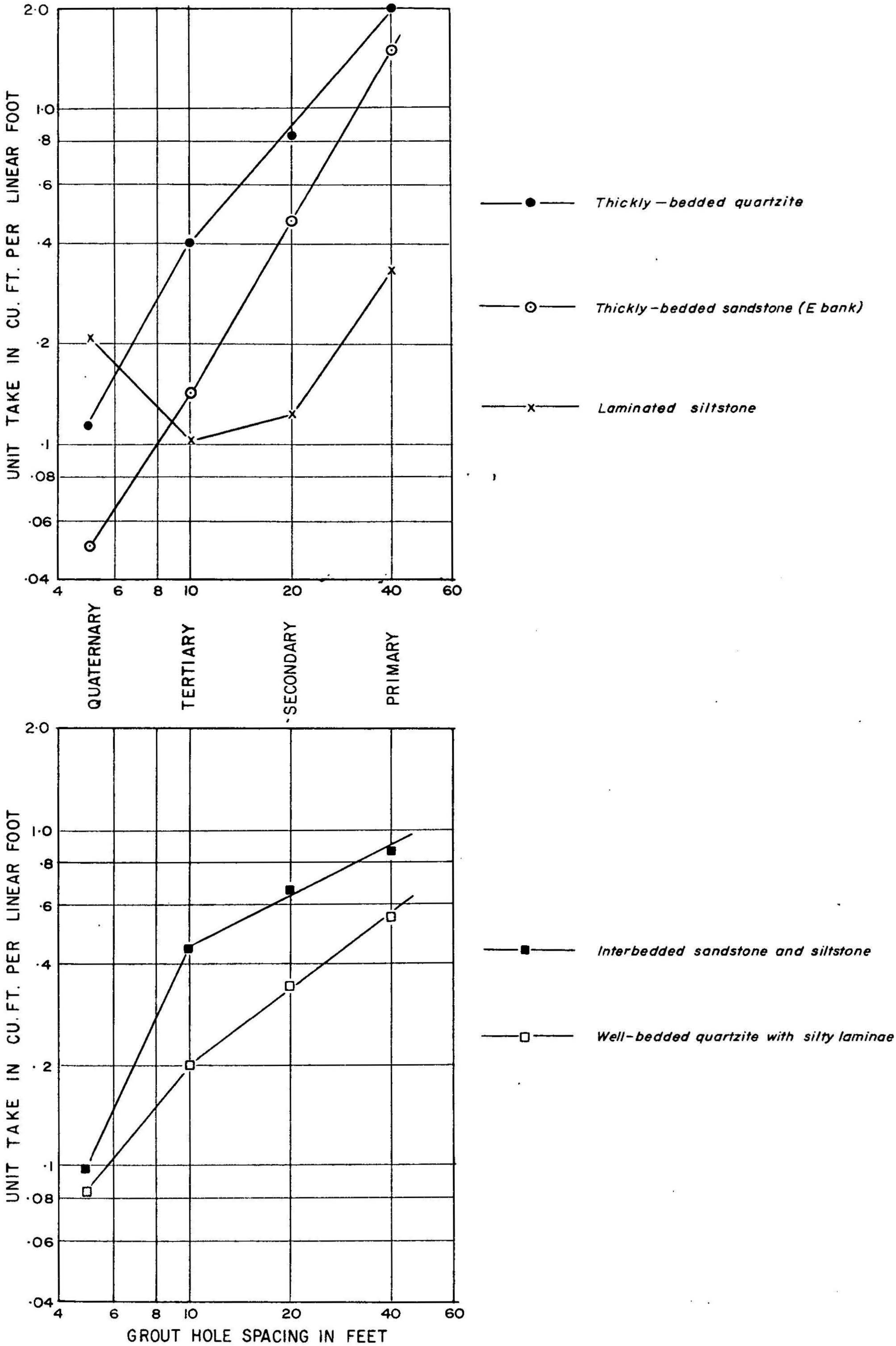


Fig. 22 Correlation of unit takes during curtain grouting with hole spacing for different foundation rock types

## Closure ratios

One of the measures of the completeness with which fissures have been filled during curtain grouting is the closure ratio; this is defined as the ratio between the average unit take of closure holes and non-closure holes. At Corin Dam, a number of proposed curtain holes of the quaternary pattern were not drilled or grouted, as grout takes in the adjacent holes were not high enough to warrant closer grouting (no stages had a unit take greater than 0.25 cu.ft./1 in. ft). Therefore, in addition to the quaternary holes actually grouted, some of the tertiary holes are closure holes also.

To obtain an indication of the ranges of closure ratios, the grout curtain was arbitrarily divided into 12 sections for analysis. The areas are defined in the following table, and the unit takes in closure and non-closure holes are given, together with the calculated closure ratios.

| Section of curtain | Unit take in cu/ft/1in.ft. Closure holes | Non-closure holes | Closure ratio |
|---|---|---|---|
| Spillway crest | 0.11 | 0.98 | 0.11 |
| Spillway buttress | 0.09 | 0.37 | 0.24 |
| 135 - 200 E | 0.09 | 0.31 | 0.29 |
| 200 - 300 E | 0.07 | 0.32 | 0.21 |
| 300 - 400 E | 0.09 | 0.40 | 0.22 |
| 400 - 500 E | 0.06 | 0.42 | 0.14 |
| 500 - 600 E | 0.13 | 0.55 | 0.23 |
| 600 - 700 E | 0.16 | 0.55 | 0.29 |
| 700 - 800 E | 0.16 | 0.43 | 0.37 |
| 800 - 900 E | 0.10 | 1.13 | 0.09 |
| 900 - 1000 E | 0.05 | 0.55 | 0.08 |
| Eastern wing | 0.28 | 1.55 | 0.18 |
| Entire site | 0.11 | 0.59 | 0.19 |

It can be seen that the closure ratios range between 0.08 and 0.37, with an average for the entire curtain of 0.19; this indicates that the closure pattern has been effective in progressively filling fissures with grout.

Correlation with water test results

The main purpose of water testing each stage before grouting was to obtain a qualitative indication of the likely grout consumption; the detection of surface leaks was an additional reason for water testing stages close to the foundation surface. As is usually the case with grouting, the water tests gave only a very rough indication of likely grout takes at Corin Dam. After some 80 stages had been grouted, the grout takes for each stage were plotted on a graph against the corresponding rate of water loss, but the resultant plot had too wide a scatter to indicate any reliable correlation which could be of use in predicting grout takes. No quantitative use could therefore be made of the water test results during the curtain grouting.

However, it seemed that there must be some relationship between water tests and grout takes, even though there is obviously no simple correlation (probably due in large part to the varying penetration characteristics of water and grout of varying consistency in fissures ranging from minute cracks to open joints). The correlation of grout takes with water test results was discussed at some length with Mr G. Kelleher, the Commonwealth Department of Works Resident Engineer at the site, and he suggested that a statistical analysis of the results of curtain grouting could indicate some correlations which would otherwise be obscured by the volume of data (some 1,100 stages were water tested and grouted to form the grout curtain). He suggested dividing the water test results into 5 or 6 ranges, finding the average grout takes and rates of water loss for each range, and then plotting the results on a graph. If a linear relationship was revealed (which was considered likely), the standard deviation of the mean grout take for each range of rate of water loss could be calculated.

The leakage rates recorded for the 25-foot stages during water tests were divided into 6 ranges; 0 to 5 g.p.m., 5.1 to 10 g.p.m., 10.1 to 15 g.p.m., 15.1 to 20 g.p.m., 20.1 to 25 g.p.m. and 25.1 to 30 g.p.m. The curtain was divided into three parts for the analysis on the basis of different conditions of grouting; the spillway wing was considered separate from the main curtain because of the abnormally-high grout takes, while the eastern wing was considered separately because of high grout takes and absence of quaternary holes. In each of these sections of the curtain, the results were analysed separately for primary, secondary, tertiary and quaternary holes. During the analysis, the results were tabulated in groups according to the stage depth in case the volume of data warranted analysis of the variation of the relationship between water loss and grout take with depth.

The curtain below the core zone was the first area analysed by this method. To start with, the four closure phases of grouting were considered separately. The plot of average grout take against average rate of water loss for the primary holes was close to a straight line, and although the plots for secondary, tertiary and quaternary holes were more scattered, a linear relationship was apparent, particularly if the high water loss values were ignored (see Fig. 23).

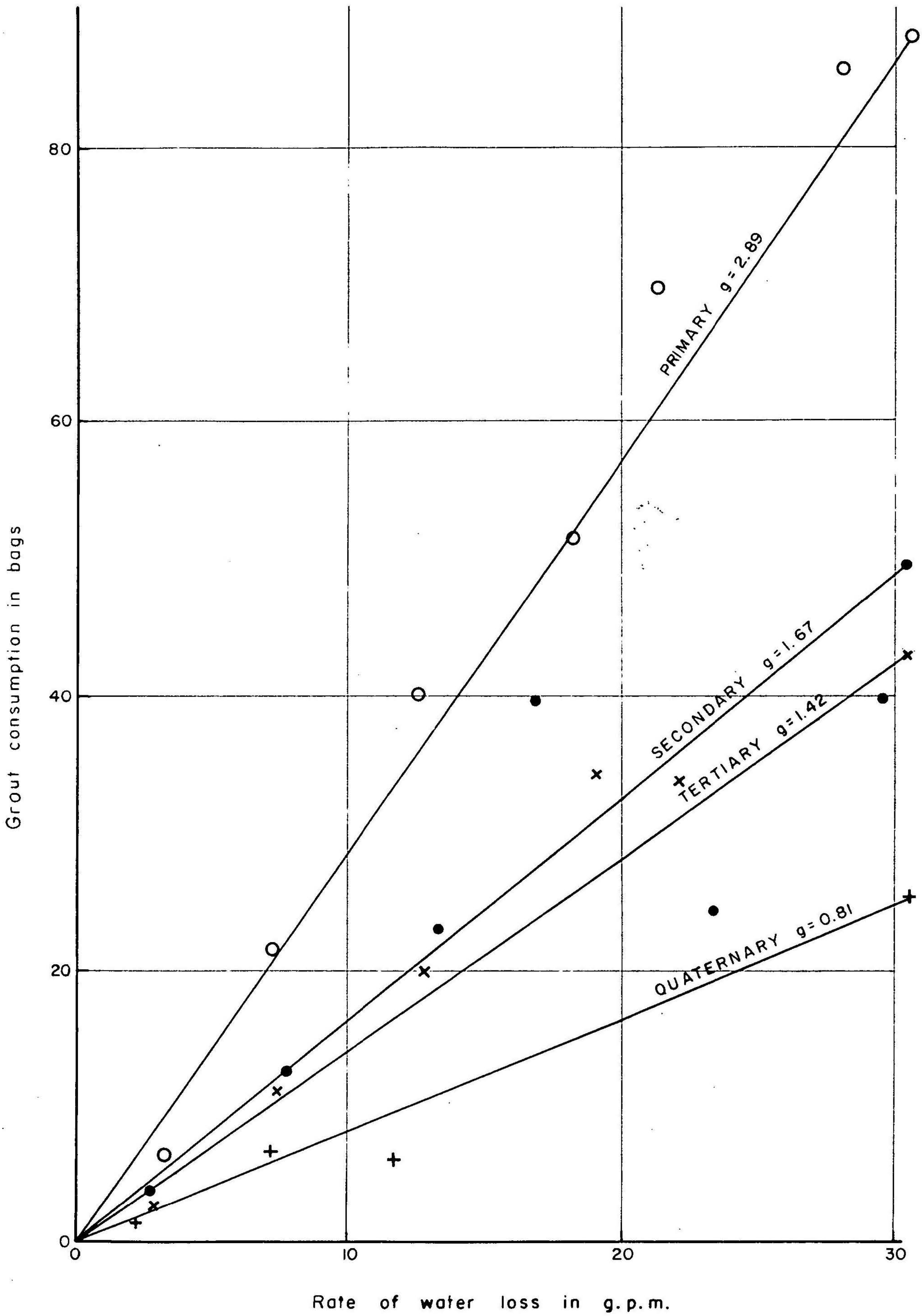


Fig. 23 :— Correlation of grout consumption in curtain holes with water test results.

Elimination of the higher values can be justified on two counts; firstly, there are generally only one or two values in the higher ranges (which do not constitute a good statistical sample), and secondly, the high rates of water leakage probably cause a substantial drop in pressure between the pressure gauge and the stage being tested. Assuming that, for each phase of grouting, the grout take in any stage is directly proportional to the corresponding rate of water loss during water testing, the gradient of the straight lines representing the best statistical fit in the graphs is given by

$$\text{Gradient } (g) = \frac{\text{sum of grout takes}}{\text{sum of rates of water loss}}$$

The calculated values of 'g' for the four phases of grouting are given in Fig. 23, and they show a progressive decrease from 2.89 for the primary holes to 0.81 for the quaternary holes. It is therefore evident that although water losses and grout takes decrease, as expected, during closure of the grouting pattern, they do not decrease proportionately; this explains the wide scatter which occurs when grout takes are plotted against water losses with no grouping. Even with the groupings selected as above, the plots still show a considerable scatter, indicating that other factors are operating within these groups which cause variations of the value 'g'. The scatter of the plots is shown in Fig. 23, and is further illustrated by the calculated standard deviation of the mean values of grout take for each range of water losses; they are tabulated below.

| Phase of grouting | Number of stages analysed | Standard deviation of mean grout take (bags) |
|---|---|---|
| Primary | 119 | 5.1 |
| Secondary | 105 | 15.2 |
| Tertiary | 135 | 5.3 |
| Quaternary | 65 | 2.9 |

The grout takes and water losses of the two wings of the grout curtain were analysed in a similar manner, and although the volume of data was considerably smaller, the same trend in the values of 'g' is evident. The results are summarised in the following table.

| Phase of grouting | Value of 'g' | Standard deviation of mean grout take (bags) |
|---|---|---|
| Spillway wing | | |
| Primary | 5.52 | 15.3 |
| Secondary | 2.94 | 12.5 |
| Tertiary | 1.15 | 6.1 |
| Quaternary | 1.04 | 0.8 |
| Eastern wing | | |
| Primary | 5.50 | 23.7 |
| Secondary | 4.36 | 10.2 |
| Tertiary | 1.43 | 3.4 |

As noted earlier, the results for the four phases of grouting were further subdivided according to the depth of grouting; these subdivisions were then analysed separately to see if there is any correlation of 'g' with depth of grouting. Again, the results are best summarised in tabular form.

| Depth of grouting | Primary | | Secondary | | Tertiary | | Quaternary | |
|---|---|---|---|---|---|---|---|---|
| | g | $S.D._{mean}$ | g | $S.D._{mean}$ | g | $S.D._{mean}$ | g | $S.D._{mean}$ |
| 25' - 50' | 2.42 | 10.3 | 1.54 | 14.2 | 1.64 | 17.6 | 0.71 | 2.1 |
| 50' - 75' | 4.02 | 10.2 | 1.83 | 18.4 | 1.32 | 2.9 | 0.92 | 4.6 |
| 75' - 100' | 3.35 | 11.2 | 1.90 | 7.2 | 1.22 | 3.4 | - | - |
| 100' - 125' | 2.64 | 12.3 | 1.40 | 13.0 | 1.79 | 0.9 | - | - |
| 125' - 150' | 1.65 | 13.4 | 1.33 | 16.8 | - | - | - | - |
| 150' - 175' | 1.59 | 12.2 | - | - | - | - | - | - |

In the primary holes, there is a definite trend of decreasing values of 'g' with increasing depth. The only grouting stage which does not fit this trend is the 25 to 50 foot stage; this is probably caused by partial grouting during the blanket grouting programme (the 0 to 25 foot stages have been omitted from all of these analyses because of interference from blanket grouting). The secondary holes also follow the same trend as the primary holes, except that the range of 'g' values is much smaller because of partial grouting. In the tertiary and quaternary holes, there is no correlation of 'g' values with depth of grouting. This is only to be expected as, by this stage of the closure pattern, the grout travel is restricted to reflect any general characteristics of the ungrouted rock.

The values for the standard deviation of mean grout take are moderately high, indicating a further variable affecting the value of 'g'; this variable is almost certainly the rock type. Analyses in previous sections have shown the correlation of grout take with rock type, and it was not considered necessary to further substantiate this correlation. An analysis of the variation of 'g' values with rock type would, it is expected, show that the thickly-bedded quartzite has the highest 'g' value, followed by thickly-bedded sandstone, interbedded sandstone and siltstone, laminated siltstone and well-bedded quartzite in decreasing order of 'g' values.

A further variable which has not been taken into account in any of these analyses, but which certainly affects the 'g' values, is the grout mix. Any correlation of grout consumption with water leakage rate implies a constant viscosity for the grout, and therefore a fixed grout mix. In the moderate to low take stages, most of the grout was injected at a water-cement ratio of 4:1 or 6:1, whereas in the high take stages, the ratio was generally increased to 1:1. Taking the thin grout mixes as the norm, this means that the calculated 'g' values for the high take stages are considerably lower than they would have been, had the stages been grouted entirely with a thin mix.

Summarising the analyses of grout take/water loss ratios, it has been shown that grout consumption can be correlated with water losses during water testing; however, at Corin Dam the ratio grout take/water loss for any stage has been affected by the phase of grouting, the depth of grouting, the rock type being grouted, and any variations in the composition of the grout. From the various calculated values of 'g' presented in the graph and tables, it appears that effective grouting has been achieved when the 'g' value drops to the range 1.5 to 1.0.

Considering the analyses in retrospect, it is likely that better correlations would have been obtained if smaller ranges of water leakage rates had been used for the original classification (say 3 g.p.m. ranges instead of 5 g.p.m.). The few stages where water losses of greater than 15 or 18 g.p.m. were obtained could then have been discarded without significantly reducing the volume of data, and the standard deviations of the means would almost certainly have been reduced considerably; it would also have eliminated many of the stages where thick grout mixes were extensively used.

## Blanket grouting

The results of blanket hole grouting cannot be analysed in such detail as the curtain grouting, because the grouting process is subject to many variations caused by the closeness to the bedrock surface. The frequent surface leaks which occurred during blanket grouting gave rise to varying grouting pressures and many changes in grout mix. The only analysis carried out on the results of blanket grouting was the preparation of a plan of the core zone foundations in which the bedrock has been classified statistically into 6 classes, according to the grout consumption in bags per 100 square feet of rock surface (in plan). The plan was prepared by plotting the cement consumption in bags at each hole, and then moving a transparent overlay with a square sample area of 400 square feet marked on it across the plan on a 10 foot grid. At each point, the total cement consumption within the square was divided by 4, and this figure was plotted at the grid point under the centre of the overlay. When the whole core zone foundation area had been covered in this manner, the figures at the 10 foot grid centres were contoured according to the 6 arbitrary classes; the resultant contour plan was then shaded to emphasise the areas of high grout takes (Plate 16). The takes in the blanket holes which were only 15 feet deep were adjusted to equivalent takes for 25 foot holes by assuming that the unit take for the 10 to 15 foot stage would be continued between 15 and 25 feet (i.e. the grout take for the second stage of the 15 foot holes was trebled).

### Correlation with geological features

The pattern of cement consumption produced by the analysis (see Plate 16) shows several areas of high grout consumption where takes of more than 100 bags per 100 square feet of rock surface are indicated; all of these high take areas are correlated with specific geological features such as faults or major fold axes. The tightness of the foundations on the west bank where the well-bedded quartzite with silty laminae crops out is also clearly evident on the contoured plan. The variations of grout consumption in the other rock types is not obvious however -

this is probably due to the erratic grouting pressures and grout mixes necessary in grouting near-surface rock. One other particular feature revealed by the analysis is the low grout consumption in the thickly-bedded sandstone adjacent to the 65° fault high up on the east bank. Analyses of the curtain grouting have shown that the sandstone generally accepted large quantities of grout, but as the 65° fault is characterised by quartz veining, which has cemented most of the fractures caused by faulting, the grout consumption in the fault zone is low.

## EFFECTIVENESS OF GROUTING

### Blanket grouting

During the blanket grouting, two cored holes were drilled in the east abutment in an attempt to check the effectiveness of the grouting. The holes were drilled with an air-powered Mindrill E 500 machine with screw feed, using an NMLC triple tube core barrel. Unfortunately, there were no drillers on site experienced in the use of the triple tube core barrel, and the cores recovered showed excessive grinding; any grout in joints would have been broken up and washed away in the drilling water. Once the placement of core material started in earnest, the grouting was not sufficiently far ahead of core placement to allow time for the grout to set before core drilling, and no further attempts at coring were made. Reliance was placed on water tests and grout takes of closure holes to indicate the effectiveness of blanket grouting.

During final foundation treatment after blanket grouting, local excavation of bedrock was necessary in many areas. This treatment showed that some of the open joints were filled with grout right to the surface. In a number of areas, however, excavation to depths of up to 4 feet failed to show extensive penetration of grout, and it was evident that the blanket grouting did not seal all the fractures in the near-surface rock. As noted in an earlier section, extensive treatment of the foundation surface was necessary to complete the impermeable barrier partly formed by the blanket grouting.

### Curtain grouting

The results of curtain grouting have been analysed in detail by several methods in this report. The unit take evaluation showed that grout saturation was achieved during quaternary grouting in all foundation rock types; further, the reduction in unit void was generally greater than the 90% considered necessary by Grant (1964) to form an effective curtain. The calculated closure ratios were all low (less than 0.4, with an average of 0.19), and this also indicates a high degree of grout saturation. On the other hand, the analyses apply to the curtain as a whole, or to extensive areas of the curtain. It is therefore quite possible that there are small localised areas where the curtain is by no means as impermeable as the analyses would indicate - in fact, considering the variable geological conditions of the foundations, local variations in the degree of grout saturation are quite likely. Reference has already been made to a spring which has formed immediately downstream of the dam, and this is probably due to such a localised 'gap' in the grout curtain. However, it is emphasised that the leakage rate from this spring is small, and the only concern with this leak is the high acidity of the water which is potentially agressive towards cement and concrete.

As yet, it is too early to assess the effectiveness of curtain grouting from observations of groundwater levels in drill holes and of water leakage in the valley downstream of the dam. While small leaks, such as the one which has already occurred, can be expected, it is highly unlikely that any major leakage will occur which will be attributable to an inadequately-constructed grout curtain.

## ACKNOWLEDGEMENTS

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I should also like to thank the Department of the Interior surveyors, Messrs C. Samundsett and M. Wells, for their frequent assistance in providing survey control for geological mapping throughout the project.

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# APPENDIX I

## DEFINITIONS OF SEMI-QUANTITATIVE DESCRIPTIVE TERMS

### Grade Scale

| | |
|---|---|
| Very coarse grained | – 1 mm. to 2 mm in diameter. |
| Coarse grained | – $\frac{1}{2}$ mm. to 1 mm in diameter. |
| Medium grained | – $\frac{1}{4}$ mm. to $\frac{1}{2}$ mm in diameter. |
| Fine grained | – $\frac{1}{8}$ mm. to $\frac{1}{4}$ mm in diameter. |
| Very fine grained | – $\frac{1}{16}$ mm. to $\frac{1}{8}$ mm in diameter. |

### Bedding

| | |
|---|---|
| Laminated | – less than 10 mm. thick. |
| Thinly bedded | – 10 mm. to 100 mm. thick. |
| Thickly bedded | – More than 100 mm. thick. |

### Hardness

| | |
|---|---|
| Hard to very hard | – Impossible to scratch with knife blade. |
| Moderately hard | – Shallow scratches with knife blade. |
| Soft | – Deep scratches with knife blade. |

### Percussive strength

| | |
|---|---|
| Strong to very strong | – Cannot be broken after repeated blows with a hammer. |
| Moderately strong | – Rock breaks after 3 or 4 heavy blows with a hammer. |
| Weak | – Rock breaks after one blow (includes brittle, fissile, friable, plastic and flaky rocks). |

### Joint spacing

| | |
|---|---|
| Closely-spaced | – Joints spaced less than 6 inches apart. |
| Moderately-spaced | – Joints spaced between 6 inches and 3 feet apart. |
| Widely spaced | – Joints spaced more than 3 feet apart. |

## APPENDIX 2.

### DETAILS OF DIAMOND DRILL HOLES.

(Locations shown on Plates 2 & 4.)

| Hole No. | Depth (feet) | Coordinates (in feet) Stromlo Grid | Dam Grid | R.L. of collar (ft) | Direction (degrees magnetic) | Inclination to horiz. (degrees) | Feature Investigated |
|---|---|---|---|---|---|---|---|
| PRELIMINARY INVESTIGATION | | | | | | | |
| P - 1 | 152 | 79819 S, 51745 W | 957 N, 506 E | 2930 | 086 | 30 | Damsite - River bed. |
| P - 1B | 112 | 79868 S, 51638 W | 948 N, 624 E | 2930 | 265 | 50 | Damsite - River bed. |
| P - 2 | 198 | 79516 S, 52552 W | 962 N, -357 E | 3264 | 270 | 45 | Cotter Fault |
| P - 3 | 134 | 79950 S, 51811 W | 1000 N, 1003 E | 3155 | 010 | 45 | Damsite - top of east abutment. |
| P - 4 | 84 | 79881 S, 51811 W | 876 N, 465 E | 2948 | 227 | 30 | Damsite - anticline on west abutment. |
| P - 5 | 121 | 79710 S, 51337 W | 1201 N, 852 E | 3090 | 165 | 55 | Damsite - fault on east abutment. |
| | 801 | | | | | | |
| FEASIBILITY INVESTIGATION | | | | | | | |
| D.D. 1 | 234 | 79767 S, 51693 W | 1025 N, 539 E | 2927 | 282 | 30 | Damsite - west abutment. |
| . . 2 | 161 | 79663 S, 51896 W | 1052 N, 313 E | 3020 | 282 | 40 | Damsite - west abutment |
| . . 3 | 195 | 79763 S, 51686 W | 1031 N, 544 E | 2927 | 068 | 60 | Damsite - river bed. |
| 4 | 200 | 79653 S, 52146 W | 974 N, 75 E | 3127 | 047 | 45 | Damsite - west abutment |
| 5 | 280 | 79555 S, 52363 W | 991 N, -163 E | 3215 | - | 90 | Damsite - west abutment (groundwater observation hole). |
| 6 | 250 | 80452 S, 52959 W | -58 N, -411 E | 3112 | 083 | 70 | Cotter Fault |
| 7 | 141 | 79878 S, 51621 W | 946 N, 645 E | 2937 | 057 | 30 | Damsite - east abutment. |
| 8 | 158 | 79894 S, 51444 W | 992 N, 817 E | 3050 | - | 90 | Damsite - anticline on east abutment |
| 9 | 201 | 79953 S, 51312 W | 983 N, 961 E | 3129 | 277 | 65 | Damsite - east abutment. |
| 10 | 165 | 79949 S, 51309 W | 987 N, 962 E | 3131 | 063 | 45 | Damsite - east abutment |
| 11 | 421 | 79877 S, 50796 W | 1233 N, 1419 E | 3303 | - | 90 | Leakage through east abutment (groundwater observation hole) |
| 12 | 176 | 79999 S, 51134 W | 1001 N, 1144 E | 3211 | 097 | 50 | Damsite - east abutment |
| 13 | 115 | 79180 S, 52181 W | 1406 N, -123 E | 3045 | - | 90 | Proposed spillway |
| 14 | 253 | 80940 S, 48237 W | - | 3202 | 280 | 30 | Proposed rockfill quarry |
| 15 | 44 | 79385 S, 52229 W | 1197 N, - 96 E | 3127 | - | 90 | Proposed spillway. |
| | 2994 | | | | | | |
| DESIGN INVESTIGATION. | | | | | | | |
| D.D.16 | 44 | 79229 S, 51792 W | 1495 N, 260 E | 2985 | - | 90 | Spillway |
| 17 | 31 | 79286 S, 51852 W | 1421 N, 223 E | 3011 | - | 90 | Spillway |
| 18 | 127 | 80223 S, 50567 W | - | 3110 | - | 90 | Proposed rockfill quarry |
| 19 | 150 | 80334 S, 50751 W | - | 3137 | - | 90 | Proposed rockfill quarry |
| 20 | 160 | 80056 S, 50384 W | - | 3109 | 050 | 45 | Proposed rockfill quarry |
| 21 | 151 | 80477 S, 50843 W | - | 3139 | - | 90 | Proposed rockfill quarry |
| 22 | 8 | 80300 S, 50440 W | - | 3030 | - | 90 | Proposed rockfill quarry |
| 23 | 75 | 80263 S, 50940 W | 821 N, 1417 E | 3239 | - | 90 | Proposed rockfill quarry |
| 24 | 71 | 79039 S, 51629 W | 1730 N, 346 E | 2925 | -- | 90 | Spillway stilling basin |
| 25 | 161 | 81422 S, 48673 W | - | 3129 | 100 | 30 | Proposed rockfill quarry |
| 26 | 100 | 79300 S, 51380 W | 1571 N, 669 E | 2901 | 126 | 60 | Tunnel outlet portal |
| 27 | 160 | 79178 S, 51493 W | 1647 N, 522 E | 2920 | 126 | 0 | Tunnel outlet portal |
| 28 | 236 | 81995 S, 48469 W | - | 3160 | 082 | 30 | Proposed rockfill quarry |
| 29 | 110 | 80319 S, 51606 W | 537 N, 812 E | 3000 | 210 | 55 | Tunnel inlet portal |
| 30 | 88 | 80433 S, 51710 W | 394 N, 754 E | 2950 | 030 | 30 | Tunnel inlet portal |
| 31 | 130 | 80238 S, 51527 W | 641 N, 858 E | 3023 | - | 90 | Valve tower shaft. |
| 32 | 269 | 84961 S, 52170 W | - | 3331 | - | 90 | Rockfill quarry |
| 33 | 185 | 84963 S, 52014 W | - | 3374 | 090 | 45 | Rockfill quarry |
| 34 | 256 | 84673 S, 52059 W | - | 3281 | - | 90 | Rockfill quarry |
| 35 | 218 | 84760 S, 52267 W | - | 3275 | - | 90 | Rockfill quarry |
| 36 | 40 | 80151 S, 52196 W | 561 N, 157 E | 2965 | - | 90 | Damsite - overburden thickness |
| 37 | 63 | 80069 S, 52212 W | 490 N, 201 E | 2987 | - | 90 | Damsite - overburden thickness |
| 38 | 53 | 79900 S, 51537 W | 954 N, 732 E | 2993 | - | 90 | Damsite - fault on east abutment |
| 39 | 75 | 79926 S, 51517 W | 937 N, 759 E | 3010 | - | 90 | Damsite - fault on east abutment |
| 40 | 40 | 79900 S, 51487 W | 972 N, 778 E | 3020 | - | 90 | Damsite - fault on east abutment |
| 41 | 175 | 85084 S, 52300 W | - | 3291 | - | 90 | Rockfill quarry |
| 42 | 180 | 85309 S, 52153 W | - | 3223 | - | 90 | Rockfill quarry |
| | 3356 | | | | | | |
| OBSERVATION HOLE DRILLED AT CONSTRUCTION STAGE | | | | | | | |
| CF1 | 150 | 78972 S, 52433 W | 1487 N, 502 E | 3091 | - | 90 | To observe change of groundwater level with filling of reservoir |

## APPENDIX 3.

## CALIBRATION OF COMPONENTS USED FOR WATER PRESSURE TESTING OF DRILL HOLES

### Packers

Both NX mechanical and BX hydraulic packers were used during the Corin Damsite investigation. The actual packers used were calibrated to determine the pressure losses of water flowing through the packers at a wide range of flow rates; an NX hydraulic packer was also calibrated so that comparisons could be made with the other packer types.

The calibration equipment consisted of a 20-foot length of 3-inch-diameter pipe, which was used to simulate the drill hole; a valve was attached to one end for controlling the rate of leakage from the "drill hole". A hole was drilled in the pipe, to which a pressure gauge was connected to measure the water pressure in the "test section". The supply line, with the packer attached, was then inserted in the appropriate position in the pipe, and the packer was sealed (with mechanical packers this was done by means of a turnbuckle which had one end attached to the pipe and the other end attached to the supply line above the packer). The supply line above the packer was equipped with a pressure gauge, a flow meter, and a valve to control the flow and pressure of water from the supply tank. Water pressure was exerted by the gravity head existing between the calibration equipment and the supply tank; the maximum head obtainable was 440 feet (194 p.s.i.).

The flow rate and supply line pressure were controlled by adjustments of the two valves. Readings of pressure above and below the packers were taken for flows ranging from 4 to 80 g.p.m., and the pressure losses were plotted against flow for each packer tested. The graphs so obtained, which are included in this Appendix, were then used to correct the field data on the computation sheets.

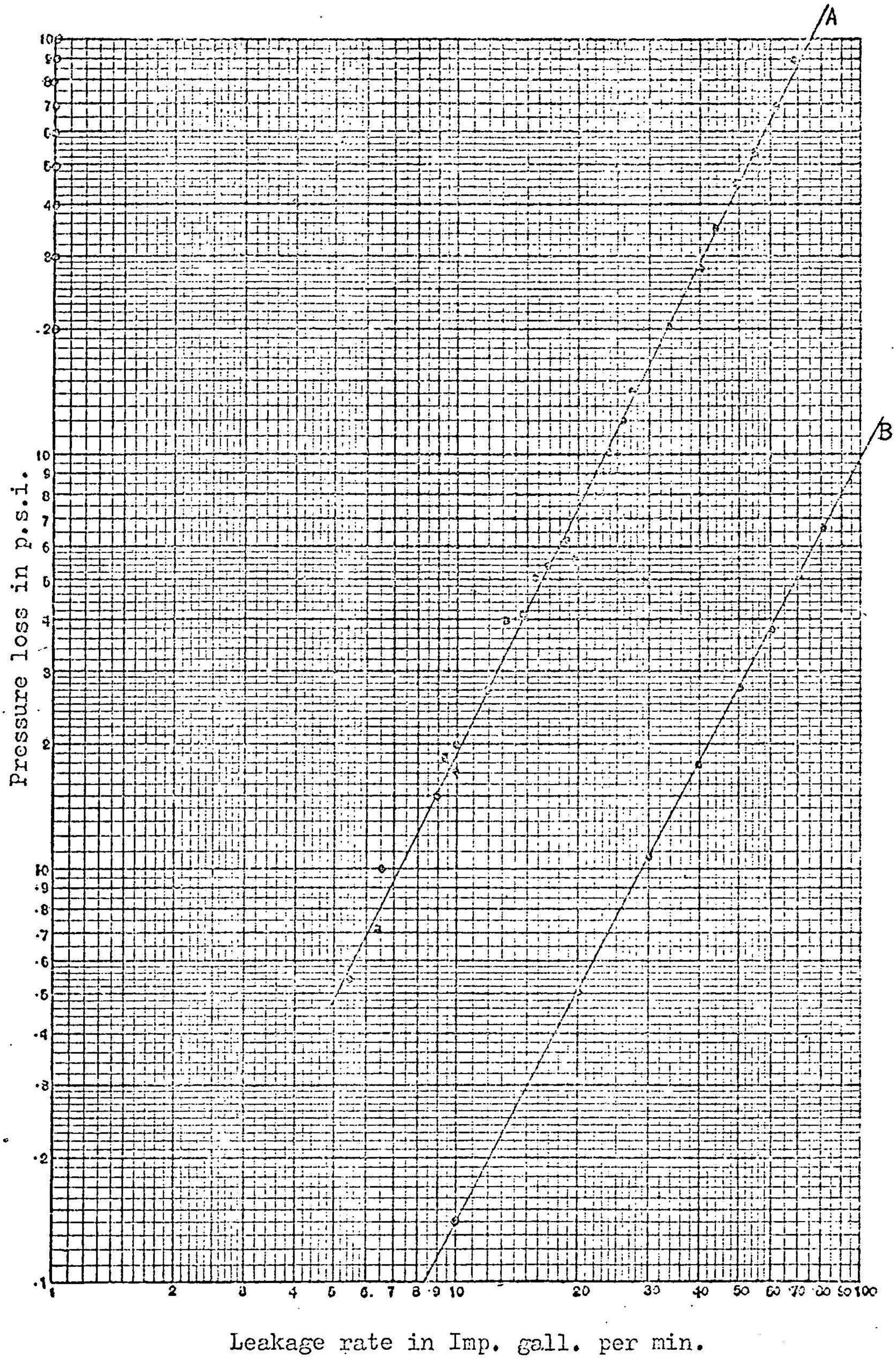
### Supply line

During the feasibility investigation, flush-coupled excasing was used as the supply line for water pressure tests, and friction losses were obtained from the standard graph of friction losses in pipes. In the design investigation, however, NX drill rod with steamflow couplings was used. Because of the constrictions in the couplings, standard graphs of pipe friction could not be used, and it was necessary to calibrate the drill rod.

The calibrations were carried out on four 10-foot lengths of NX drill rod connected by streamflow couplings. Similar couplings were necessary at each end of the drill rod string so that gauges and a water meter could be connected; a total of 5 couplings were therefore necessary in the 40 feet of rods under calibration. Water was passed through the rods at flow rates ranging between 10 and 82 g.p.m., and the pressure losses were calculated from the readings of pressure gauges at each end of the drillrod string. Because of the extra coupling, it was necessary to calculate the total loss in the 5 couplings at any given flow rate by subtracting the friction in the 40 feet of drill rod (read from standard graphs). Thus, the loss for one coupling was calculated, and this was subtracted from the total loss to give the pressure loss for 40 feet of rod and 4 couplings.

## Results

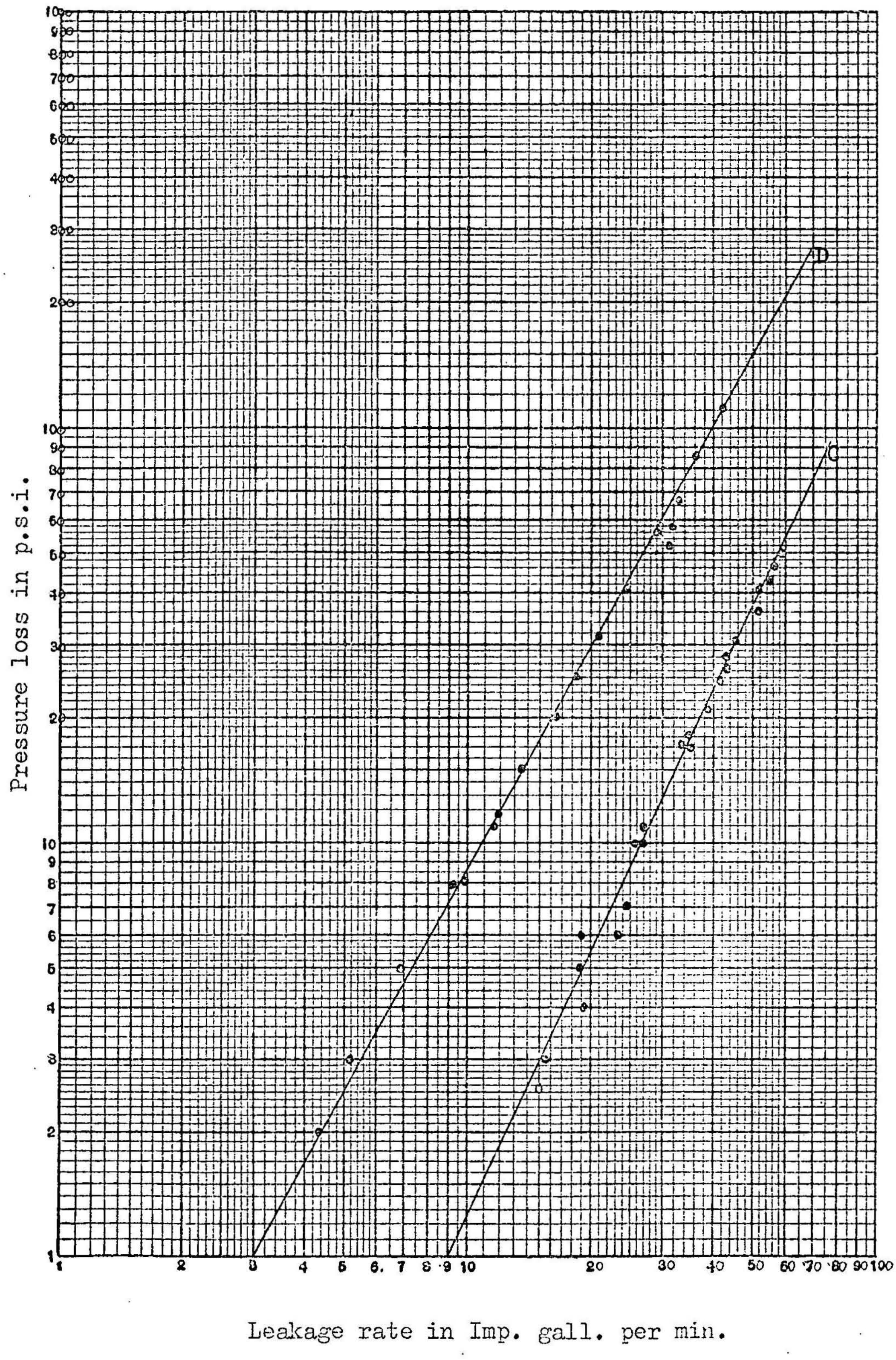
The results of the calibration tests are plotted on the following series of graphs. Each packer has its own friction characteristics, and for accurate water pressure tests results individual packers should be calibrated. However, the following graphs can be used with a reasonable degree of accuracy if it is impracticable to calibrate equipment on any particular project; this course of action is certainly preferable to ignoring friction losses in packers altogether.




Graph showing pressure loss due to friction at various flow rates for:-

(A) Four-rubber N-size mechanical packer with 20 feet of perforated N-rod as injector tube

(B) 10 feet of N-rod with one streamflow coupling




Graph showing pressure loss due to friction at various flow rates for:-

(C) N-size hydraulic packer

(D) B-size hydraulic packer

APPENDIX 4

GROUNDWATER LEVELS IN OBSERVATION HOLES

| Date | R.L. of groundwater in feet | | |
|---|---|---|---|
| | D.D. 5 (Collar RL 3315') | D.D. 10 (Collar RL 3131') | D.D. 11 (Collar RL 3303') |
| 28/4/64 | 2969 | 3034 | - |
| 22/5/64 | 2950 | 3031 | - |
| 3/6/64 | 2946 | 3029 | 3020 |
| 11/6/64 | 2946 | 3028 | 3015 |
| 4/8/64 | 2960 | 3039 | - |
| 3/9/64 | 2961 | - | 3026 |
| 24/9/64 | 2961 | - | 3023 |
| 13/10/64 | 2975 | 3049 | 3035 |
| 21/10/64 | - | 3043 | 3033 |
| 28/10/64 | 2960 | 3037 | 3018 |
| 10/11/64 | 2961 | 3044 | 3015 |
| 20/11/64 | 2953 | 3035 | 3029 |
| 3/12/64 | 2947 | 3034 | 3026 |
| 12/1/65 | 2944 | 3037 | 3020 |
| 22/1/65 | 2943 | 3029 | 3016 |
| 28/1/65 | 2943 | 3029 | 3017 |
| 5/2/65 | 2943 | 3028 | 3016 |
| 11/2/65 | 2943 | 3028 | 3016 |
| 18/2/65 | 2943 | 3028 | 3015 |
| 24/2/65 | 2943 | 3028 | 3015 |
| 4/3/65 | 2943 | 3028 | - |
| 17/3/65 | 2943 | 3025 | 3015 |
| 25/3/65 | 2943 | 3024 | 3014 |
| 31/3/65 | 2943 | 3023 | 3015 |
| 7/4/65 | 2943 | 3024 | 3014 |
| 14/4/65 | 2943 | 3024 | 3014 |
| 19/5/65 | 2942 | 3023 | 3011 |
| 18/8/65 | below 2933 | 3022 | 3007 |
| 9/12/65 | 2961 | 3039 | 3014 |
| 22/12/65 | 2954 | 3040 | 3016 |
| 6/2/67 | 2949 | hole caved | 2988 |
| 15/3/67 | below 2935 | " | 2983 |
| 28/3/67 | " | " | 2983 |
| 6/7/67 | " | " | 2978 |
| 6/9/67 | " | " | 2982 |
| 23/10/67 | 2943 | " | 2992 |
| 6/12/67 | 2941 | " | 2988 |
| 23/1/68 | 2935 | " | 2983 |
| 27/3/68 | below 2935 | " | 2982 |
| 24/4/68 | " | " | 2982 |

Appendix 4

| Date | Reservoir level | R.L. of groundwater in feet D.D. 5 (Collar RL 3215') | D.D. 11 (Collar RL 3303') | C.F. 1 (Collar RL 3091') |
|---|---|---|---|---|
| 14/5/68 | - | below 2935 | 2982 | below 2941 |
| 23/5/68 | 2973.9 | 2965 | 2982 | " |
| 4/6/68 | 2987.5 | 2982 | 2986 | " |
| 12/6/68 | 2999.0 | 2984 | 2997 | " |
| 26/6/68 | 3007.0 | 2982 | 3005 | " |
| 3/7/68 | 3009.3 | 2981 | 3008 | " |
| 10/7/68 | 3011.8 | 2981 | 3009 | " |
| 18/7/68 | 3014.0 | 2982 | 3011 | " |
| 31/7/68 | 3017.8 | 2983 | 3013 | " |
| 7/8/68 | 3019.7 | 2985 | 3015 | " |
| 13/8/68 | 3026.2 | 3009 | 3018 | " |
| 22/8/68 | 3032.8 | 3002 | 3022 | " |
| 2/9/68 | 3044.4 | 3017 | 3026 | 2949 |
| 10/9/68 | 3050.0 | 3015 | 3028 | 2951 |
| 17/9/68 | 3054.1 | 3016½ | - | 2952 |
| 26/9/68 | 3057.8 | 3018½ | 3030 | 2953 |
| 4/10/68 | 3062.5 | 3023 | 3032 | 2955 |
| 11/10/68 | 3072.5 | 3042 | 3034 | 2959½ |
| 18/10/68 | 3078.1 | 3044 | 3038 | 2961½ |
| 25/10/68 | 3081.3 | 3047 | 3038 | - |
| 4/11/68 | 3085.0 | 3050½ | 3038 | - |
| 8/11/68 | 3087.0 | 3056 | 3043½ | 2970 |
| 15/11/68 | 3092.0 | 3062 | 3048 | 2975 |
| 22/11/68 | 3095.0 | 3060 | 3052½ | 2976 |
| 29/11/68 | 3096.9 | 3059½ | 3054 | 2977 |
| 6/12/68 | 3098.2 | 3061 | 3055½ | 2978 |
| 12/2/69 | 3098.2 | 3063½ | 3065 | 2980 |
| 19/2/69 | 3099.6 | 3063½ | 3065 | 2980 |
| 18/3/69 | 3099.6 | 3063 | - | 2979½ |
| 25/3/69 | 3099.9 | 3063 | 3066½ | 2979½ |
| 1/4/69 | 3101.7 | 3065 | 3067 | 2980 |
| 8/4/69 | 3103.0 | 3066 | 3067 | 2980 |
| 17/4/69 | 3112.9 | 3079½ | 3072½ | 2985 |
| 2/5/69 | 3121.2 | 3077 | 3079 | 2988½ |
| 9/5/69 | 3122.6 | 3079 | 3081 | 2990 |
| 16/5/69 | 3124.1 | 3080½ | 3082½ | 2990½ |
| 27/5/69 | 3126 | 3082 | 3091 | 2989 |
| 2/6/69 | 3127 | 3084 | 3095 | 2990 |
| 10/6/69 | 3130 | 3080 | 3094 | 2992 |
| 18/6/69 | 3131.80 | 3084½ | 3093 | 2993 |
| 25/6/69 | 3135 | 3093½ | 3093 | 2996½ |
| 1/7/69 | 3137 | 3094½ | 3094 | 2997 |
| 10/7/69 | 3137.50 | 3095 | - | - |
| 23/7/69 | 3137.50 | 3096 | 3097 | - |
| 28/7/69 | 3137.50 | 3095½ | - | 2995 |
| 4/8/69 | 3137.50 | 3107 | 3095 | 2993 |

APPENDIX 5

ROCK CLASSIFICATION USED IN TUNNEL MAPPING

(after Pender, Hosking and Mattner, 1963)*

| Rock Class | Rock Condition | Characteristics | Typical support used on works of the Snowy Mountains Authority. |
|---|---|---|---|
| 5 | Excellent | Sound, compact, usually dry rock, either unjointed or with tightly closed, strongly cemented joints. Tends to break across the rock on excavation, rather than along joint planes, hence the traces of blast holes usually remain. | No support, except in large excavations such as power stat |
| 5S | Excellent (but highly stressed) | As above, but "spalling" or "popping" occurs. | Not as yet encountered on these works. |
| 4. | Good | Hard rock generally dry with tightly closed weakly cemented joints; some slightly open joints with water seepages or flows may occur. May contain some narrow sheared or crushed zones. Tends to break along the weakly cemented joints on excavation, rather than across the rock. The percentage breakage along joint planes in any particular case largely depends upon the orientation of the excavation surfaces in relation to the main joint direction. | Mainly unsupported or with few rock bolts to pin shoulders if orientation of excavation is adverse in relation to mai jointing. Generally less than one bolt per linear foot of 20 feet diameter tunnel. |
| 3. | Very fair | Mainly hard rock, but considerably loosened by the opening up of weakly cemented joints on excavation, or due to the presence of slightly open joints, or narrow sheared or crushed zones. May be dry but usually wet. Tends to break entirely along joint planes on excavation regardless of the orientation of excavation surfaces in relation to the joint planes. | Light steel sets or "completely" supported in rock by rock bolts, occasionally supplemented by steel channel and wire mesh. Usually 2 to 3 bolts per linear foot of 20 feet diameter tunnel. |
| 2. | Fair | Partly hard rock but usually containing more than 10 percent of soft material classifiable as soil, i.e., crushed zones loosely jointed sheared zones or highly to completely altered zones. | Generally heavy steel sets in tunnels and light steel sets, or rock bolts, with mesh in shafts. |
| 1. | Poor | Consists of more than 50 percent soft material classifiable as soil, i.e., crushed zones, loosely jointed sheared zones or highly to completely altered zones. Can be excavated without the use of explosives. Exerts pressure on supports. | Generally heavy steel sets with close timbering in both tunnels and shafts. Invert struts occasionally required in tunnels. |
| 0 | Very Poor | Consists entirely of soft material classifiable as soil, usually crushed or completely altered rock. Exerts pressure on top and sides of supports. Includes squeezing and swelling ground. | Heavy steel sets at close spacing in both tunnels and shaft: Invert struts required in tunnel. Timber or steel spiling required. Pilot drifts and other special tunnelling techn possibly required. |

* Grouted rock bolts for permanent support of major underground works. J. Inst. Engrs. Aust. V. 35, No.7-8, pp. 129-150.

APPENDIX 6

GEOLOGICAL LOG OF DIVERSION TUNNEL.

All geological data and relevant engineering data from the tunnel excavation are given on the appended log sheets. The following explanatory notes are pertinent to the presentation of information on the log sheets.

1. The geological symbols used are the same as on the geological plans, except that the rock type symbols for siltstone and interbedded sandstone and siltstone are omitted for the sake of clarity.

2. In the excavation column, the vertical lines represent the position of the working face after each blast. From chainage 50 to 605 feet, the figures to the left of the face indicate the number of holes in the round, the advance in feet and the amount of explosive used to blast the round; from chainage 605 to 1350 feet, the figures are to the right of the corresponding face.

3. The vertical lines in the support column represent the actual positions of steel sets.

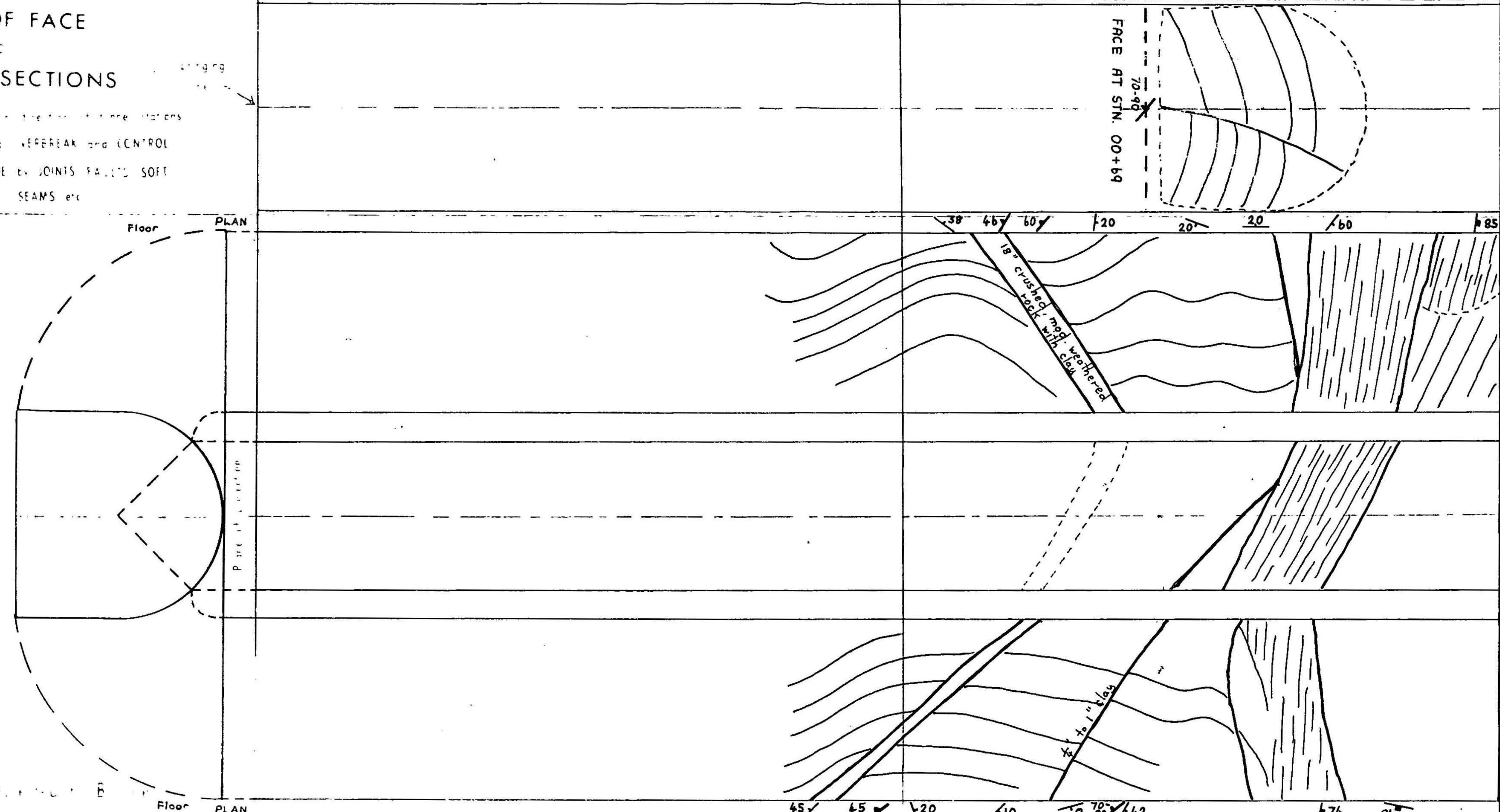
# CORIN DAM DIVERSION TUNNEL – GEOLOGICAL LOG



0 10 20 30 FEET

| | | |
|---|---|---|
| TUNNEL STATIONS | 00+50 … T.P. 00+96 | For section from 00+96 to 01+50 see Sheet 2 |
| DIRECTION INVERT LEVEL SLOPE | Direction : 059°58'09' → Slope 0·053% Invert level 2924 feet | |
| PROGRESS heading at date | 3/6/66, 6/6/66, 9/6/66, 10/6/66, 11/6/66, 14/6/66, 15/6/66, 16/6/66, 17/6/66, 18/6/66, 21/6/66 | |
| ROCK TYPE, degree of weathering or alteration | INTERBEDDED SANDSTONE AND SILTSTONE fresh to slightly weathered; SILTSTONE cleaved, fresh; SILTSTONE laminated, fresh | |

LOG OF FACE AND CROSS SECTIONS




STRUCTURES

Bedding planes most prominent partings. Direction of bedding variable due to folding. Bedding planes spaced 6" to 2 feet apart and surfaces generally weathered.

Faults: ① 40°-60° to tunnel and dipping 45°-60° west
② 45° to tunnel and dipping 70°-80° west

Fault zone of cleaved siltstone. Cleavage strike at 80° to tunnel and dips around 75° east.

Most prominent joints strike 80° to tunnel and dips 75° to 85° east, generally around 6" apart.

| ROCK CONDITION / PERCENT BREAKAGE ALONG JOINTS | Interbedded sandstone and siltstone | Siltstone cleaved | Siltstone laminated |
|---|---|---|---|
| Rock condition | 3 | 1 | 3 |
| Percent breakage along joints | 80 | 90 | 90 |

GROUND WATER — initial quantity date; quantity date; temperature; cumulative total

| EXCAVATION | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Advance No of holes | 17:3½ | 17:? | 21:4 | 29:4 | 24:3 | 25:3 | 18:3 | 35:3 | 32:3 | 42:3 | 40:3 | 44:6½ | 40:? | 40:5 | 40:5 |
| Explosives lb | 3¼ | 7 | 7½ | 23 | 20 | 30 | 20 | 50 | 43 | 55 | 53 | 100 | 89 | 81 | 65 |

SUPPORT DURING CONSTRUCTION — WEST WALL; ROOF; EAST WALL

LOGGED: E. J. Best and G.A.M. Henderson DATE LOGGED: 22/6/66 DRAWN: G.A.M. Henderson

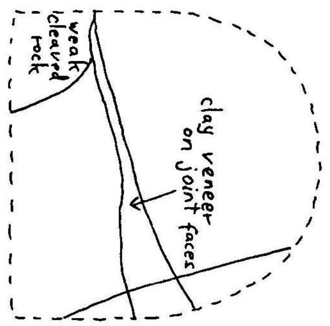
0 10 20 30 FEET

# CORIN DAM DIVERSION TUNNEL—GEOLOGICAL LOG.

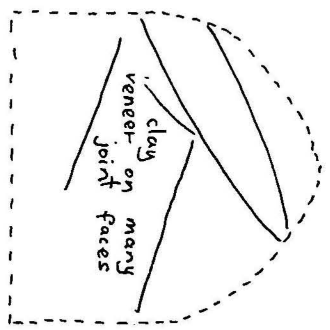


| | | |
|---|---|---|
| TUNNEL STATIONS | For section from 00+41 to 00+96 see Sheet 1 | T.P. 00+96 01+00 … 01+50 |
| DIRECTION, INVERT LEVEL, SLOPE | | Direction: 064° → (approx.) Slope: 0·053% Invert level 2924 feet |
| PROGRESS HEADING AT, DATE | | 21/6/66 22/6/66 23/6/66 24/6/66 25/6/66 27/6/66 |
| ROCK TYPE DEGREE OF WEATHERING OR ALTERATION | | SILTSTONE laminated, fresh |

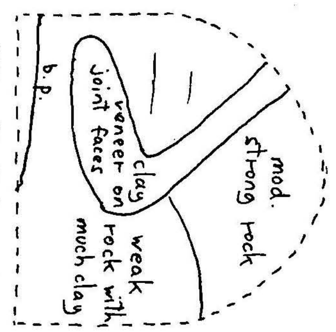
FACE AT STN. 00+97

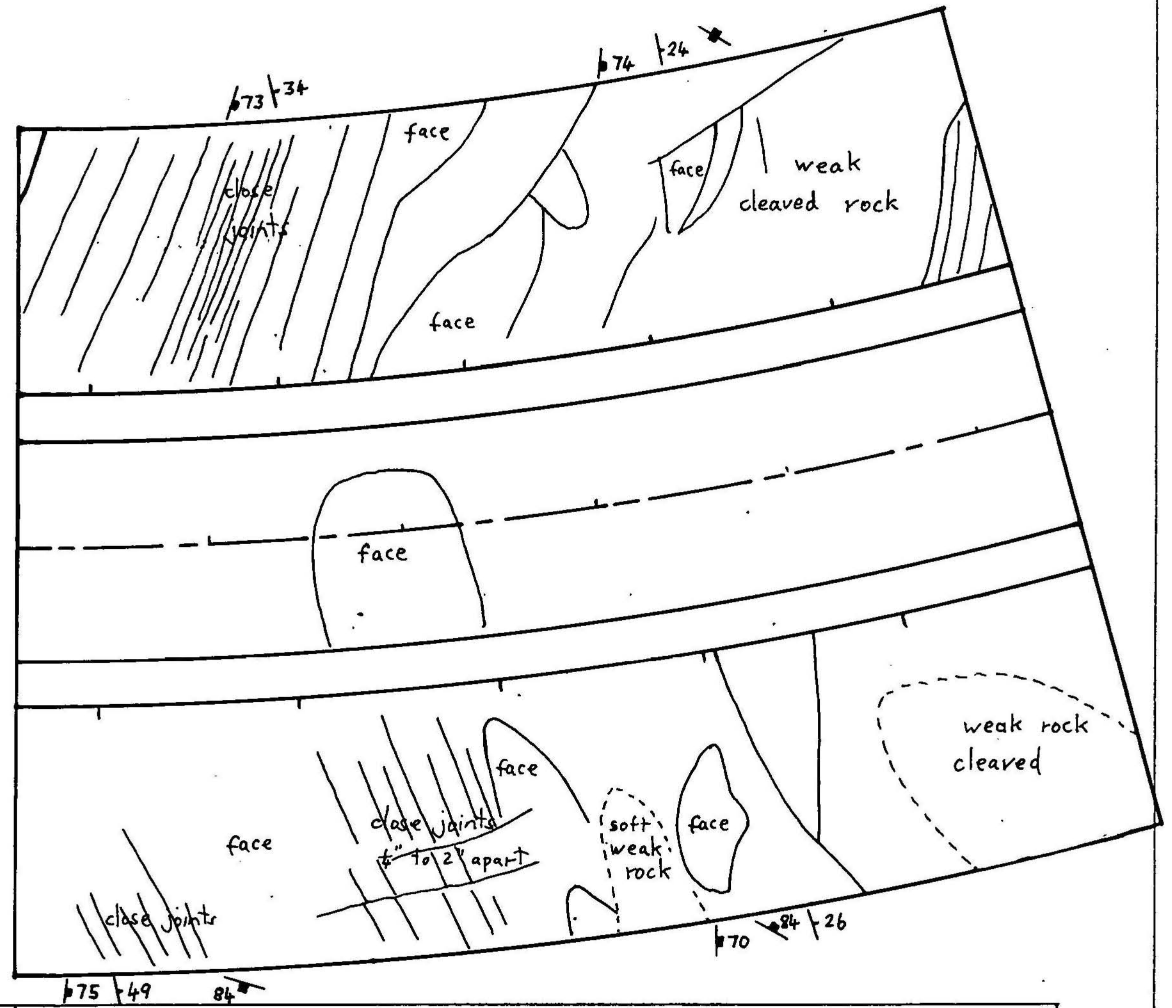



FACE AT STN. 01+22




FACE AT STN. 01+42




| | | | |
|---|---|---|---|
| STRUCTURES JOINTING PATTERN—ATTITUDE SPACING. PERSISTENCE CHARACTER—SURFACE SEPARATIONS COATING FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL | | | Most prominent joint direction 70° to 80° to tunnel and dips 70°-75° to east; becomes a cleavage in places. Other joint direction 10° to 40° to tunnel and near vertical. Bedding planes not prominent; dips range from 24° to 49° to east in direction of tunnel. Many joint planes clay coated. |
| ROCK CONDITION | | | 2 to 3 |
| PERCENT BREAKAGE ALONG JOINTS | | | 70 to 90 |
| GROUND WATER INITIAL QUANTITY, DATE | | | Small water flow 4ft above invert on east wall |
| QUANTITY, DATE | | | |
| TEMPERATURE | | | |
| CUMULATIVE TOTAL million gal./day | | | |

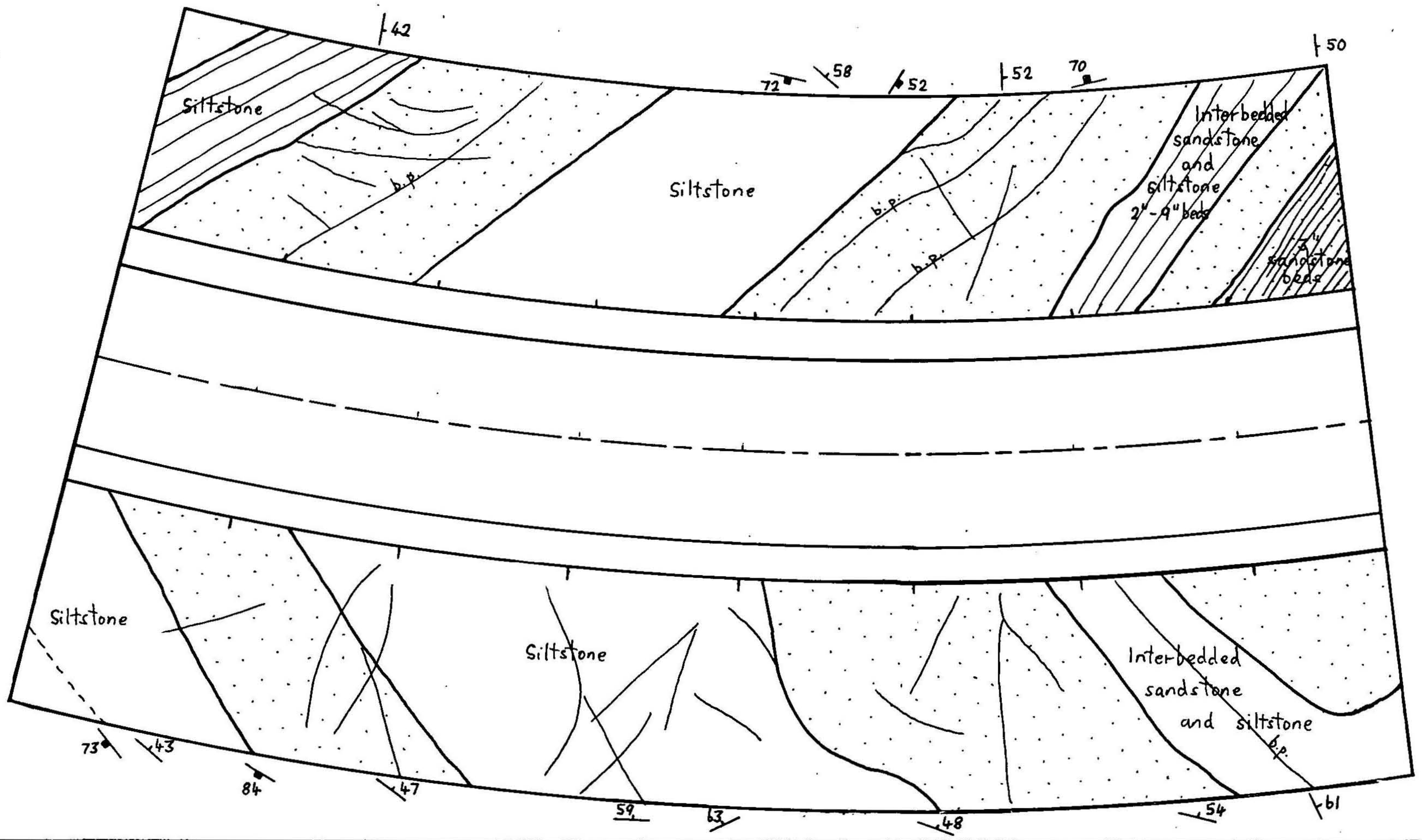
| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| EXCAVATION ADVANCE No of HOLES | 43:5 | 38:5 | 39:5 | 38:5 | 37:5 | 38:5 | 38:5 | 37:5 | 40:5 | 40:5 | 39:5 |
| EXPLOSIVES, lb | 67 | 55 | 65 | 67 | 63 | 65 | 65 | 63 | 89 | 88 | 79 |
| SUPPORT DURING CONSTRUCTION Springing line SOUTH WALL | | | | | | | | | | | |
| ROOF | | | | | | | | | | | |
| NORTH WALL | | | | | | | | | | | |

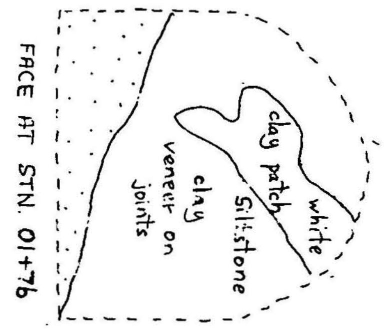
LOGGED: G.A.M. HENDERSON. DATE LOGGED: 22 27/6/66. DRAWN: G.A.M. HENDERSON. To accompany Record 1969/111 I55/A16/571(2)

0 10 20 30 FEET

# CORIN DAM DIVERSION TUNNEL—GEOLOGICAL LOG.

| | | | | |
|---|---|---|---|---|
| TUNNEL STATIONS | 01+50 | 02+00 | 02+28 | For section from 02+28 to 03+00 see Sheet 4 |
| DIRECTION, INVERT LEVEL, SLOPE | Direction: 032° → (approx.) Slope: 0·053 % | Invert level: 2923 feet | | |
| PROGRESS HEADING AT, DATE | 28/6/66 · 29/6/66 · 30/6/66 | 1/7/66 | 2/7/66 | |
| ROCK TYPE DEGREE OF WEATHERING OR ALTERATION | SANDSTONE AND SILTSTONE fresh | | | |




**STRUCTURES**
JOINTING PATTERN—ATTITUDE SPACING, PERSISTENCE CHARACTER—SURFACE SEPARATIONS, COATING
FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL

Bedding planes are the most prominent partings and range from 2 inches to several feet apart according to the rock type. Attitude of bedding planes is variable due to folding. Other joint directions are irregular and not persistent.

Many joint surfaces are coated with clay.

| | |
|---|---|
| ROCK CONDITION | 3 to 4 |
| PERCENT BREAKAGE ALONG JOINTS | 30% in sandstone 50% to 80% in siltstone |
| GROUND WATER — INITIAL QUANTITY, DATE | |
| QUANTITY, DATE | |
| TEMPERATURE | |
| CUMULATIVE TOTAL million gal/day | |

| EXCAVATION | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No of HOLES | 40:5 | 40:5 | 42:5 | 42:5 | 42:5 | 42:5 | 38:5 | 38:5 | 40:5 | 40:5 | 38:5 | 40:5 | 42:5 | 42:5 | 40:5 |
| EXPLOSIVES, lb | 85 | 75 | 77 | 77 | 80 | 79 | 70 | 70 | 77 | 77 | 75 | 100 | 80 | 82 | 75 |

| SUPPORT DURING CONSTRUCTION | |
|---|---|
| SOUTH WALL | |
| Springing line — ROOF | |
| NORTH WALL | |

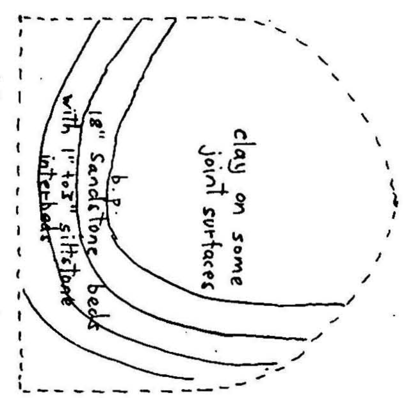
LOGGED: G.A.M. HENDERSON. DATE LOGGED: 4,5/7/66. DRAWN: G.A.M. HENDERSON. *To accompany Record 1969/111* I 55/A16/571(3)

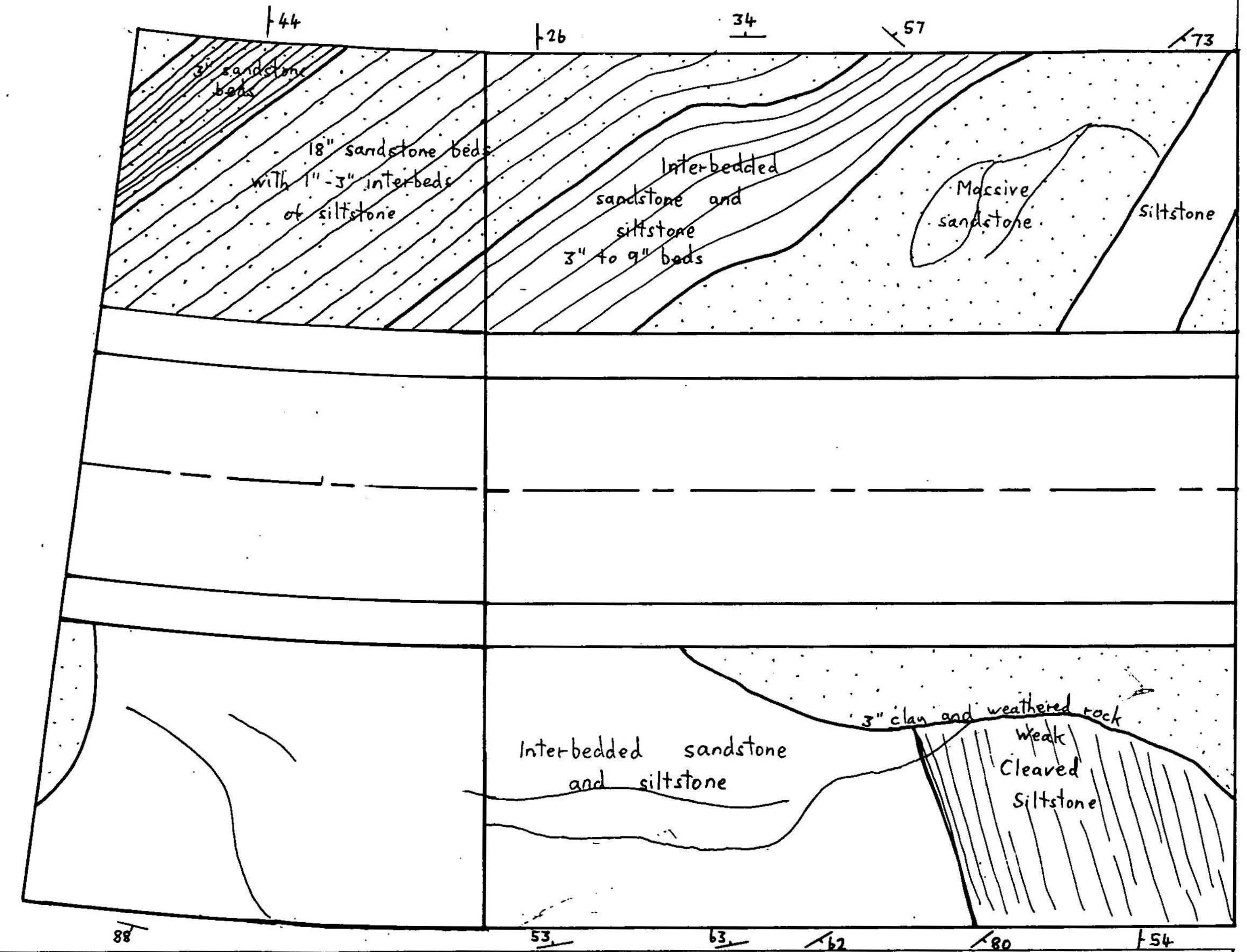
**TUNNEL STATIONS** — For section from 01+50 to 02+28 see Sheet 3 — 02+28 — 02+50 — T.P. 02+53 — 03+00

**DIRECTION, INVERT LEVEL, SLOPE** — Direction: 014° 58' 09" Slope: 0.053% Invert level: 2923 feet.

**PROGRESS** HEADING AT DATE — 4/7/66 | 5/7/66 | 6/7/66 | 7/7/66 | 8/7/66

**ROCK TYPE** DEGREE OF WEATHERING OR ALTERATION — SANDSTONE AND SILTSTONE fresh




**STRUCTURES**
JOINTING PATTERN–ATTITUDE SPACING
PERSISTENCE CHARACTER–
SURFACE SEPARATIONS COATING
FAULTS SHEARED AND CRUSHED ZONES
ATTITUDE RELATIVE
DISPLACEMENT WIDTH
CHARACTER OF MATERIAL

Bedding planes are the most prominent partings and are spaced as shown above; attitude of bedding is variable due to folding. Cleavage in weak, cleaved siltstone strikes at 40° to tunnel and dips 80° east. Other joint directions are irregular and not persistent.

Many joint surfaces are clay coated. The 3" clay seam on the east wall appears to be a fault.

**ROCK CONDITION** — 3 to 4 — 2 in cleaved siltstone

PERCENT BREAKAGE ALONG JOINTS — 30% in massive sandstone — 50-80% elsewhere

**GROUND WATER** — INITIAL QUANTITY, DATE / QUANTITY, DATE / TEMPERATURE / CUMULATIVE TOTAL million gal/day

**EXCAVATION**

| | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No. of HOLES | 42:5 | 46:5 | 48:5 | 46:5 | 60:5 | 60:5 | 46:5 | 47:5 | 80:5 | 80:5 | 46:5 | 58:5 | 60:5 | 46:5 |
| EXPLOSIVES, lb | 80 | 110 | 100 | 112 | 100 | 100 | 100 | 91 | 120 | 100 | 115 | 100 | 100 | 88 |

**SUPPORT DURING CONSTRUCTION** — Springing line — SOUTH WALL / ROOF / NORTH WALL

LOGGED: G.A.M. HENDERSON. DATE LOGGED: 5,7,11/7/66. DRAWN: G.A.M. HENDERSON. To accompany Record 1969/111 I55/A16/571(4)

0 10 20 30 FEET

# CORIN DAM DIVERSION TUNNEL – GEOLOGICAL LOG.

TUNNEL STATIONS: 03+00 | 03+50 | 04+00 | 04+50

DIRECTION INVERT LEVEL SLOPE: Direction : 014° 58' 09" → | Slope: 0·053% | Invert level : 2922 feet

PROGRESS HEADING AT DATE: 9/7/66 | 11/7/66 | 12/7/66 | 13/7/66 | 14/7/66

ROCK TYPE DEGREE OF WEATHERING OR ALTERATION: SANDSTONE AND SILTSTONE fresh

LOG OF FACE AND CROSS SECTIONS

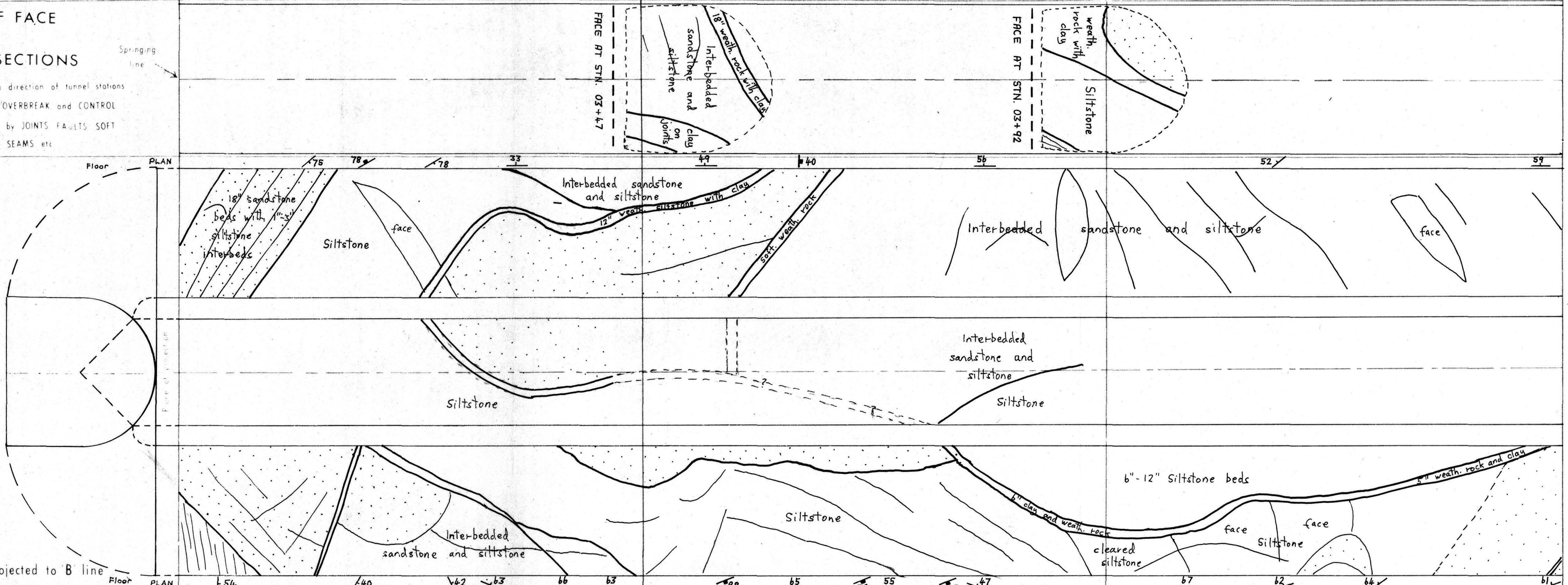
Looking in direction of tunnel stations showing OVERBREAK and CONTROL of SHAPE by JOINTS FAULTS SOFT SEAMS etc

WEST WALL *

ROOF *

EAST WALL *

* Geology projected to 'B' line




STRUCTURES
JOINTING PATTERN – ATTITUDE SPACING PERSISTENCE CHARACTER – SURFACE SEPARATIONS COATING
FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL

Bedding planes most prominent partings; attitude variable due to folding. Other joints are irregular and not persistent.

Bedding planes striking nearly parallel to tunnel direction and dipping between 47° and 67° west are most prominent partings. Other joints irregular and not persistent

5" to 6" seam of clay and weathered rock on east wall between chainages 03+82 and 04+49 is parallel to bedding

Clay on many bedding and joint planes throughout

ROCK CONDITION: 3 to 4

PERCENT BREAKAGE ALONG JOINTS: 50 – 80 %

GROUND WATER – INITIAL QUANTITY DATE / QUANTITY DATE / TEMPERATURE / CUMULATIVE TOTAL million gal day

EXCAVATION

| ADVANCE No of HOLES | EXPLOSIVES lb |
|---|---|
| 38:5 | 76 |
| 40:5 | 75 |
| 42:5 | 70 |
| 40:5 | 60 |
| 38:5 | 80 |
| 38:5 | 75 |
| 40:5 | 75 |
| 40:5 | 70 |
| 36:5 | 75 |
| 38:5 | 77 |
| 37:5 | 60 |
| 37:5 | 62 |
| 40:5 | 75 |
| 40:5 | 75 |
| 40:5 | 75 |
| 42:5 | 86 |
| 36:5 | 59 |
| 36:5 | 61 |
| 40:5 | 75 |
| 40:5 | 75 |
| 42:5 | 80 |
| 36:5 | 62 |
| 36:5 | 71 |
| 38:5 | 70 |
| 38:5 | 70 |
| 38:5 | 70 |
| 40:5 | 80 |
| 40:5 | 81 |
| 32:5 | 53 |
| 32:5 | 61 |

SUPPORT DURING CONSTRUCTION – Springing line – WEST WALL / ROOF / EAST WALL

LOGGED: G. A. M. HENDERSON  DATE LOGGED: 11 13 15 18/7/66  DRAWN: G. A. M. HENDERSON.  To accompany Record 1969/111  I55/A16/571(5)

# CORIN DAM DIVERSION TUNNEL – GEOLOGICAL LOG.

0 10 20 30 FEET

TUNNEL STATIONS: 04+50 | 05+00 | 05+50 | 06+00

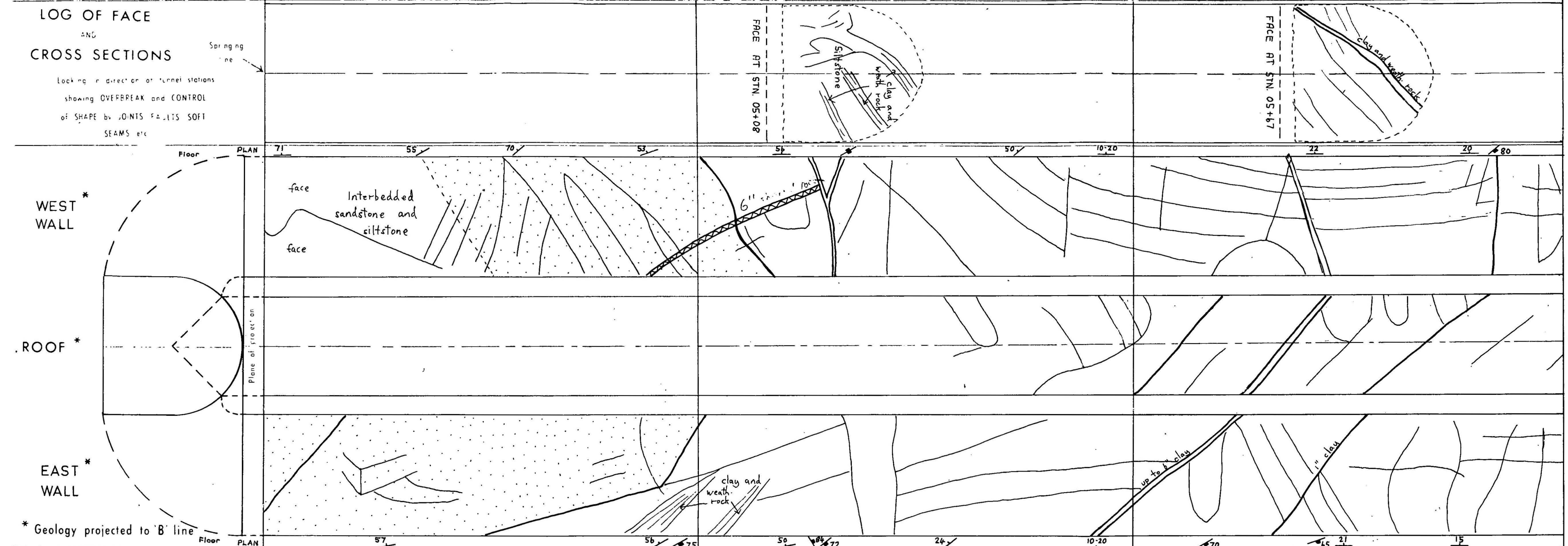
DIRECTION INVERT LEVEL SLOPE: Direction : 014° 58' 09" | Slope : 0·053 % | Invert level : 2921 feet

PROGRESS HEADING AT DATE: 15/7/66 | 16/7/66 | 18/7/66 | 19/7/66 | 20/7/66

ROCK TYPE DEGREE OF WEATHERING OR ALTERATION: SANDSTONE AND SILTSTONE fresh | SILTSTONE fresh

LOG OF FACE AND CROSS SECTIONS

Looking in direction of tunnel stations showing OVERBREAK and CONTROL of SHAPE by JOINTS FAULTS SOFT SEAMS etc




STRUCTURES

JOINTING PATTERN – ATTITUDE SPACING PERSISTENCE CHARACTER – SURFACE SEPARATIONS COATING
FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL

Bedding planes are the most prominent partings and are spaced more than 18" apart; bedding strikes at 0° to 40° to tunnel and dips between 53° and 71° west.

Widely spaced joints striking around 30° to tunnel direction and dipping between 65° and 80° east are prominent and form many faces. Bedding planes are spaced 9 inches to 3 feet apart and are also prominent; beyond 05+40 to 06+00 bedding strikes parallel to the tunnel and dips range between 10° and 22°.

Clay seams are confined to certain of the steeply dipping joints.

ROCK CONDITION: 3 to 4 | 2 to 3 | 3

PERCENT BREAKAGE ALONG JOINTS: 50 - 70 % | 30% | 70 - 90 %

GROUND WATER: INITIAL QUANTITY, DATE; QUANTITY DATE; TEMPERATURE; CUMULATIVE TOTAL million gal / day

EXCAVATION

| ADVANCE No of HOLES | 34:5 | 30:5 | 34:5 | 36:5 | 35:5 | 35:5 | 40:5 | 40:5 | 38:5 | 38:5 | 34:6 | 42:5 | 42:5 | 40:5 | 38:5 | 38:5 | 38:5 | 42:5 | 42:8 | 42:8 | 42:8 | 42:6 | 42:8 | 42:8 | 40:8 | 35:9 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| EXPLOSIVES. lb | 65 | 70 | 76 | 77 | 70 | 70 | 75 | 75 | 70 | 88 | 75 | 80 | 91 | 75 | 80 | 80 | 84 | 95 | 130 | 100 | 120 | 120 | 132 | 100 | 120 | 100 |

SUPPORT DURING CONSTRUCTION: WEST WALL; Springing line; ROOF; EAST WALL

LOGGED : G. A. M. HENDERSON. DATE LOGGED : 18, 20, 21/7/66. DRAWN : G. A. M. HENDERSON. *To accompany Record 1969/111* I 55/A16/571(6)

# CORIN DAM DIVERSION TUNNEL – GEOLOGICAL LOG.



0 10 20 30 FEET

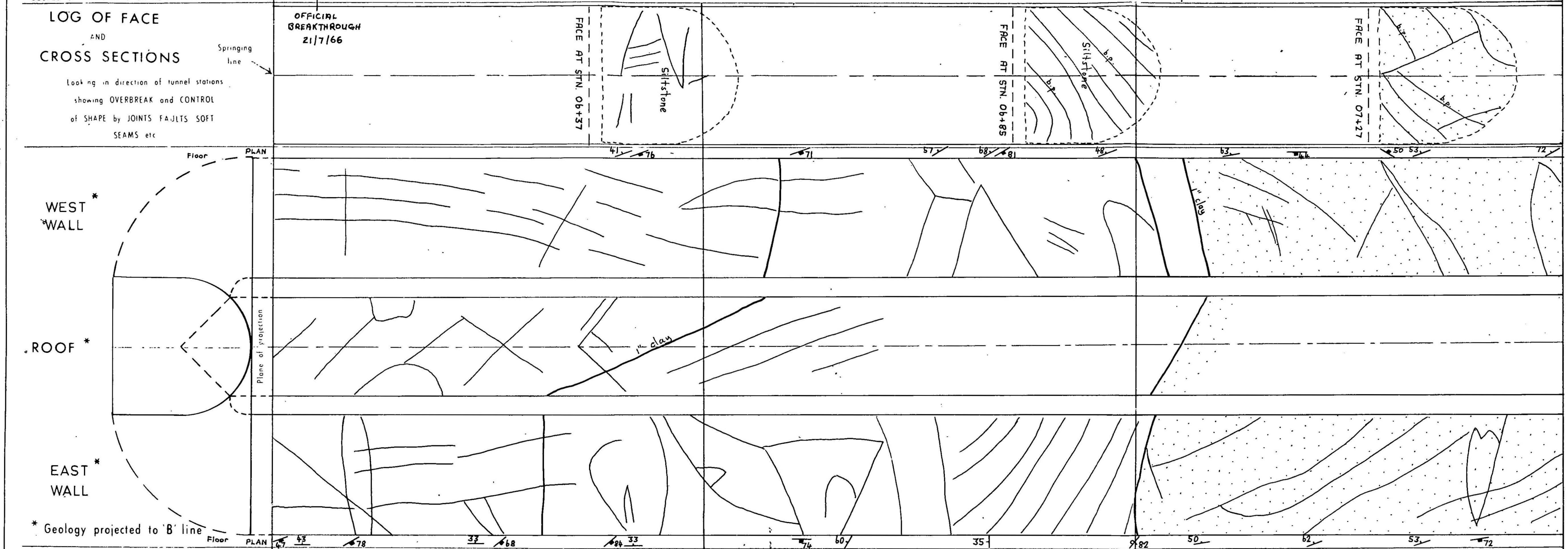
| TUNNEL STATIONS | 06+00 | 06+50 | 07+00 | 07+50 |
|---|---|---|---|---|
| DIRECTION, INVERT LEVEL, SLOPE | Direction : 014° 58' 9" | Slope : 0·053 % | Invert level : 2921 feet | |
| PROGRESS – HEADING AT DATE | 20/7/66, 20/7/6, 19/7/66, 18/7/66 | 16/7/66, 15/7/66 | 14/7/66, 13/7/66, 12/7/66 | |
| ROCK TYPE – DEGREE OF WEATHERING OR ALTERATION | SILTSTONE fresh | | SANDSTONE fresh | |

OFFICIAL BREAKTHROUGH 21/7/66

LOG OF FACE AND CROSS SECTIONS

Looking in direction of tunnel stations showing OVERBREAK and CONTROL of SHAPE by JOINTS FAULTS SOFT SEAMS etc

Springing line




* Geology projected to 'B' line

## STRUCTURES

JOINTING PATTERN – ATTITUDE SPACING PERSISTENCE CHARACTER – SURFACE SEPARATIONS COATING
FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL

Prominent joints strike around 40° to tunnel direction and dip between 71° and 84° east, and are widely spaced. Bedding is also prominent; strike of beds is parallel to tunnel at 06+00 feet but swings around to about 30° to the tunnel at 06+50 feet and beyond. Dips range from 33° to 60° west.

Clay seams are not common; only one seam, on a steeply dipping joint plane, was noted.

Bedding planes are the most prominent partings; bedding strikes around 30° to the tunnel and dips range from 50° to 72° west. Other joints are irregular.

The contact between the sandstone and siltstone contains about 1 inch of clay and appears to be faulted. Clay occurs on some bedding planes.

| | Siltstone | Sandstone |
|---|---|---|
| ROCK CONDITION | 4 | 3 |
| PERCENT BREAKAGE ALONG JOINTS | 70 - 90 % | 30 - 50 % |

GROUND WATER – INITIAL QUANTITY, DATE; QUANTITY, DATE; TEMPERATURE; CUMULATIVE TOTAL million gal / day

| EXCAVATION | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No of HOLES | 46:5 | 45:8 | 45:8 | 42:7 | 42:7 | 40:8 | 42:8 | 42:8 | 42:6 | 42:5 | 34:5 | ?:5. | 42:5 | 42:5 | 36:5 | 36:5 | 42:5 | 42:5 | 38:5 | 36:5 | 42:5 | 44:5 | 36:5 | 42:5 | 42:5 | 40:5 |
| EXPLOSIVES, lb | 80 | 85 | 100 | 138 | 145 | 125 | 132 | 125 | 95 | 87 | 70 | 65 | 80 | 82 | 70 | 71 | 80 | 84 | 75 | 72 | 76 | 86 | 68 | 85 | 85 | 60 |

SUPPORT DURING CONSTRUCTION – Springing line – WEST WALL, ROOF, EAST WALL

LOGGED : G.A.M. HENDERSON. DATE LOGGED : 13, 15, 18, 21/7/66. DRAWN : G.A.M. HENDERSON.

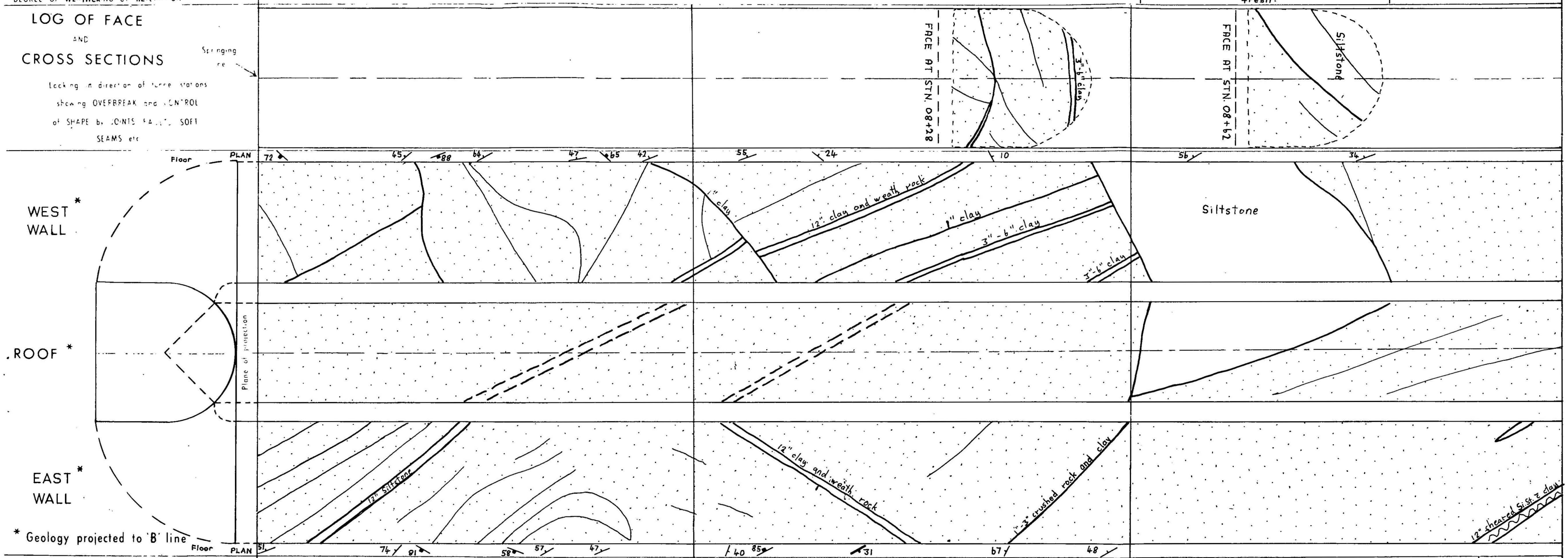
To accompany Record 1969/111

I55/A16/571(7)

# CORIN DAM DIVERSION TUNNEL—GEOLOGICAL LOG.



| TUNNEL STATIONS | 07+50 | 08+00 | 08+50 | 09+00 |
|---|---|---|---|---|
| DIRECTION INVERT LEVEL SLOPE | Direction: 014° 58' 9" ← | Slope: 0·053% | Invert level: 2920 feet | |
| PROGRESS HEADING AT DATE | 11/7/66 ; 9/7/66 | 8/7/66 ; 7/7/66 | 6/7/66 ; 5/7/66 | |
| ROCK TYPE DEGREE OF WEATHERING OR ALTERATION | SANDSTONE fresh | | SANDSTONE AND SILTSTONE fresh. ; SANDSTONE fresh | |




| STRUCTURES | 07+50 – 08+00 | 08+00 – 08+50 | 08+50 – 09+00 | |
|---|---|---|---|---|
| JOINTING PATTERN–ATTITUDE SPACING PERSISTENCE CHARACTER–SURFACE SEPARATIONS COATING; FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL | Bedding planes are the most prominent partings; strike of bedding is at 25°-40° to tunnel and dip is 45°-70° to S.W. A set of joints is also present striking at 30° to tunnel and dipping steeply to the S.E. Other joints are irregular. Generally little clay on joints. | Several thin siltstone interbeds are present which have weathered to clay. Dip of bedding is 10°-25° to N.W., indicating folding which was probably caused by the fault at 08+51; the fault zone is narrow and does not constitute a major zone of weakness in the rock. | Bedding planes are the prominent joints. They are widely spaced (4"-30", generally 8"-20") and only a few have a veneer of clay. The siltstone is more closely jointed (2"-15" spacing) but joints are tight. | See following sheet |
| ROCK CONDITION | 3 | | 3-4 | 3 |
| PERCENT BREAKAGE ALONG JOINTS | 40-60% | | 30-50% | 50-70% |

GROUND WATER: INITIAL QUANTITY, DATE; QUANTITY, DATE; TEMPERATURE; CUMULATIVE TOTAL million gal/day

| EXCAVATION | ADVANCE No of HOLES | EXPLOSIVES lb |
|---|---|---|
| | 46:5 | 97 |
| | 45:5 | 95 |
| | 40:5 | 82 |
| | 40:5 | 85 |
| | 40:5 | 70 |
| | 42:5 | 70 |
| | 28:5 | 42 |
| | 28:5 | 40 |
| | 42:5 | 90 |
| | 40:5 | 89 |
| | 45:5 | 100 |
| | 37:5 | 70 |
| | 37:5 | 61 |
| | 40:5 | 78 |
| | 40:5 | 80 |
| | 40:5 | 100 |
| | 28:5 | 42 |
| | 28:5 | 40 |
| | 40:5 | 76 |
| | 40:5 | 75 |
| | 50:5 | 100 |
| | 38:8 | 91 |
| | 38:8 | 85 |
| | 42:5 | 75 |
| | 40:5 | 75 |
| | 44:7 | 100 |
| | 42:7 | 100 |
| | 37:5 | 41 |

SUPPORT DURING CONSTRUCTION — Springing line: WEST WALL; ROOF; EAST WALL

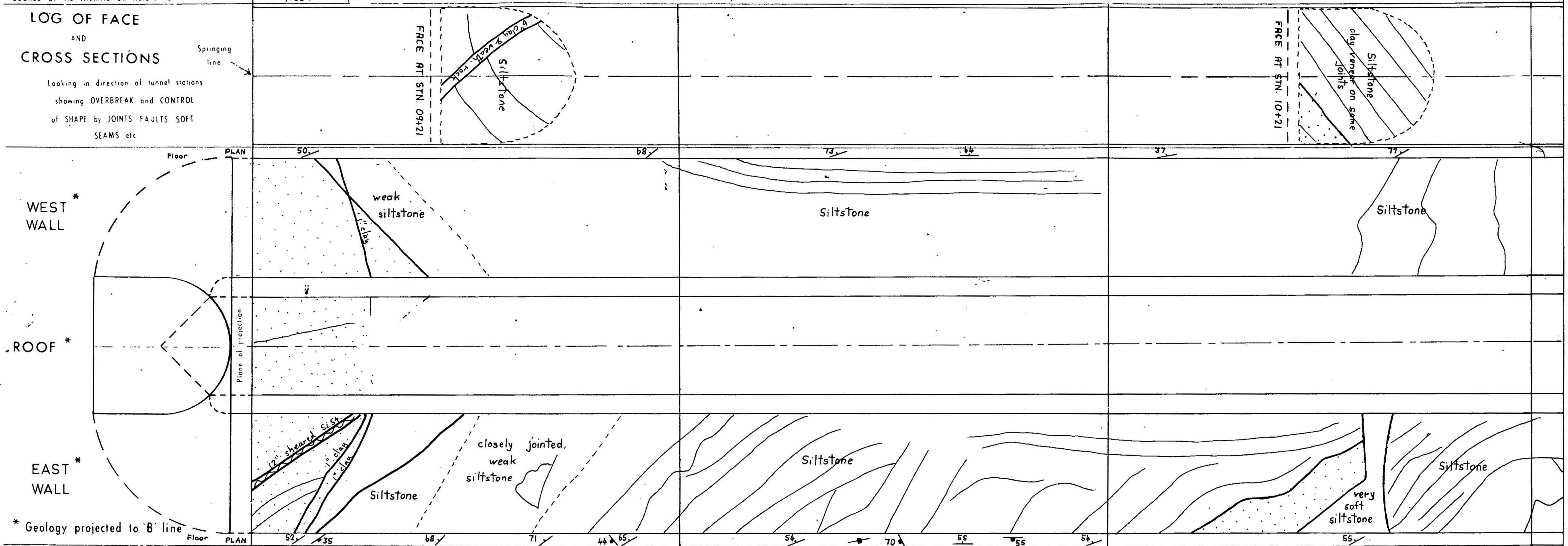
LOGGED: G. A. M. HENDERSON. DATE LOGGED: 6, 7, 11, 13/7/66. DRAWN: G. A. M. HENDERSON. To accompany Record 1969/111

# CORIN DAM DIVERSION TUNNEL—GEOLOGICAL LOG.



0 10 20 30 FEET

| TUNNEL STATIONS | 09+00 | 09+50 | 10+00 | T.P. 10+54 |
|---|---|---|---|---|
| DIRECTION. INVERT LEVEL SLOPE | Direction : 014° 58' 09" | Slope : 0·053% | Invert level : 2919 feet | |
| PROGRESS HEADING AT DATE | 5/7/66 · 4/7/66 · 2/7/66 | 1/7/66 · 30/6/66 | 29/6/66 · 28/6/66 | |
| ROCK TYPE DEGREE OF WEATHERING OR ALTERATION | SANDSTONE fresh · SILTSTONE fresh | | | |




**STRUCTURES**
JOINTING PATTERN—ATTITUDE SPACING PERSISTENCE CHARACTER—SURFACE SEPARATIONS COATING
FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL

Joints spaced 2"-20" & generally weathered - several clay seams up to 1" wide are present.

Fresh laminated siltstone with some beds showing cleavage parallel to bedding. Bedding plane joints are prominent, and are generally iron-stained. Other irregular joints are also present. No clay seams are present, other than those noted on log.

Prominent set of bedding plane joints striking parallel to tunnel and dipping 55° to W. Another set is also developed, with strike at 65° to tunnel and dip of 70° to S.E. Joints are generally fresh and clean.

Several bedding plane joints with iron staining have caused significant overbreak.

No regular pattern of joints other than bedding plane joints. Joints are fresh and clean generally – a few have a veneer of grey clay.

See following sheet

Several joints dipping 45°N.E. have caused overbreak in E. wall.

| | | | | | | |
|---|---|---|---|---|---|---|
| ROCK CONDITION | 3 | 3-4 | 4 | | 3-4 | 3 |
| PERCENT BREAKAGE ALONG JOINTS | 50-70% | 80-90% | 70-90% | 80% | 60-80% | 50-80% |

GROUND WATER: INITIAL QUANTITY. DATE / QUANTITY. DATE / TEMPERATURE. / CUMULATIVE TOTAL million gal / day

| EXCAVATION | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No of HOLES | 38 : 5 | 37 : 5 | 40 : 5 | 40 : 5 | 42 : 5 | 40 : 5 | 37 : 6 | 37 : 5 | 40 : 5 | 40 : 5 | 38 : 5 | 38 : 5 | 40 : 8 | 40 : 5 | 40 : 5 | 38 : 5 | 40 : 5 | 40 : 5 | 42 : 5 | 45 : 5 | 45 : 5 | 38 : 5 | 38 : 6 | 40 : 5 | 38 : 4 | 42 : 5 | 42 : 5 | 42 : 6 | 42 : 6 | 46 : 6 |
| EXPLOSIVES. lb | 75 | 65 | 80 | 78 | 80 | 80 | 63 | 60 | 75 | 80 | 77 | 75 | 100 | 70 | 80 | 75 | 75 | 75 | 75 | 73 | 75 | 75 | 75 | 70 | 50 | 80 | 82 | 80 | 85 | 93 |

SUPPORT DURING CONSTRUCTION: WEST WALL / Springing line / ROOF / EAST WALL

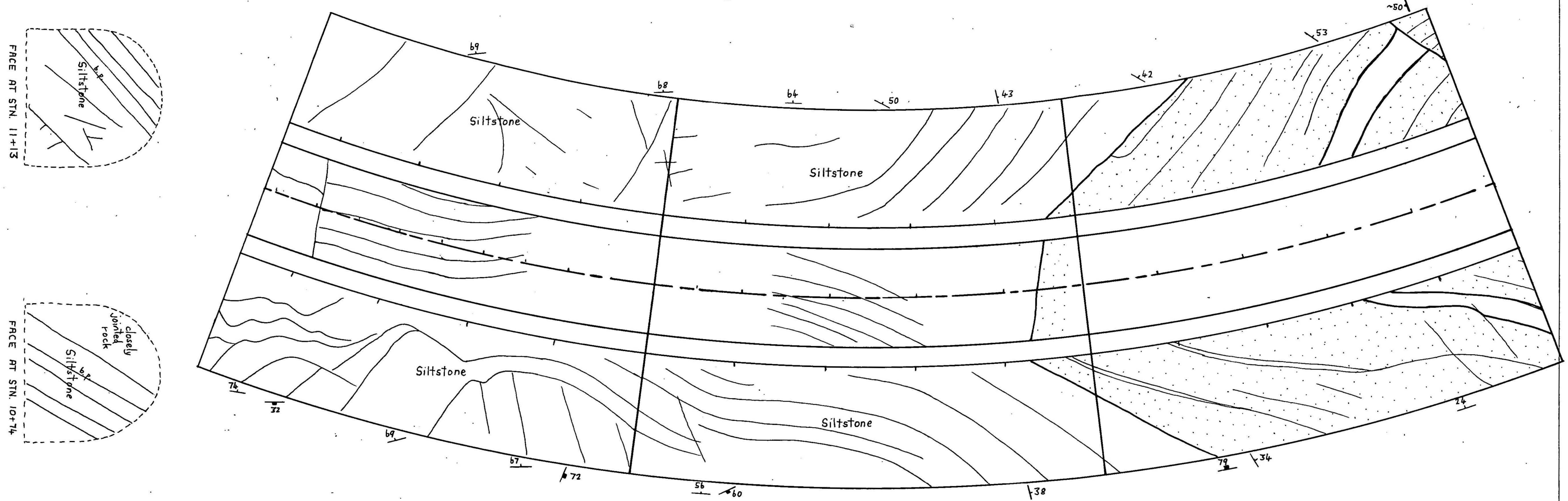
LOGGED : G.A.M. HENDERSON.    DATE LOGGED : 4,5,6/7/66.    DRAWN : G.A.M. HENDERSON.    *To accompany Record 1969/111*    I 55 / A16 / 571(9)

# CORIN DAM DIVERSION TUNNEL – GEOLOGICAL LOG.



0 10 20 30 FEET

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| TUNNEL STATIONS | T.P. 10+54 | | 11+00 | | | 11+50 | | | | 12+00 |
| DIRECTION, INVERT LEVEL, SLOPE | | | Slope : 0.053% | | | | Invert level : 2918 feet | | | |
| PROGRESS HEADING AT, DATE | 27/6/66 | 25/6/66 | 24/6/66 | 23/6/66 | 22/6/66 | 21/6/66 | 20/6/66 | 18/6/66 | 17/6/66 | |
| ROCK TYPE DEGREE OF WEATHERING OR ALTERATION | SILTSTONE fresh | | | | | SANDSTONE AND SILTSTONE fresh | | | | |




| | | | |
|---|---|---|---|
| **STRUCTURES** JOINTING PATTERN – ATTITUDE, SPACING PERSISTENCE CHARACTER – SURFACE SEPARATIONS COATING FAULTS SHEARED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL | No regular joint sets, other than bedding plane joints which are spaced 2"-24" apart (generally 8"-16"). A few major joints have up to 1" grey clay; all others are fresh tight and clean. A few parallel joints, dipping 50°-60° N.E. are developed. | Bedding plane joints and a set dipping 70°-80° N.E. are developed. Other joints are irregular. Some joints are stained brown. | Thin film of clay on most bedding plane joints; brown staining is also common adjacent to joints. |
| ROCK CONDITION | 3 / 3-4 | | 3 |
| PERCENT BREAKAGE ALONG JOINTS | 50-80% | | 70-80% |
| GROUND WATER: INITIAL QUANTITY, DATE | | | |
| QUANTITY, DATE | | | |
| TEMPERATURE | | | |
| CUMULATIVE TOTAL million gal/day | | | |

**EXCAVATION**

| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No of HOLES | 40:7 | 44:8 | 38:4 | 39:6 | 38:5 | 39:6 | 38:5 | 43:5 | 39:5 | 37:5 | 41:8 | 38:5 | 38:5 | 38:5 | 39:5 | 40:4 | 38:5 | 39:5 | 43:5 | 39:5 | 46:5 | 45:5 | 39:5 | 53:6 | 40:5 | 38:4 | 47:4 | 38:4 | 39:5 |
| EXPLOSIVES, lb | 80 | 75 | 55 | 82 | 75 | 85 | 65 | 87 | 71 | 55 | 95 | 73 | 75 | 65 | 71 | 73 | 75 | 71 | 82 | 75 | 72 | 70 | 77 | 110 | 83 | 72 | 70 | 72 | 73 |

| | |
|---|---|
| SUPPORT DURING CONSTRUCTION – Springing line – WEST WALL | |
| ROOF | |
| EAST WALL | |

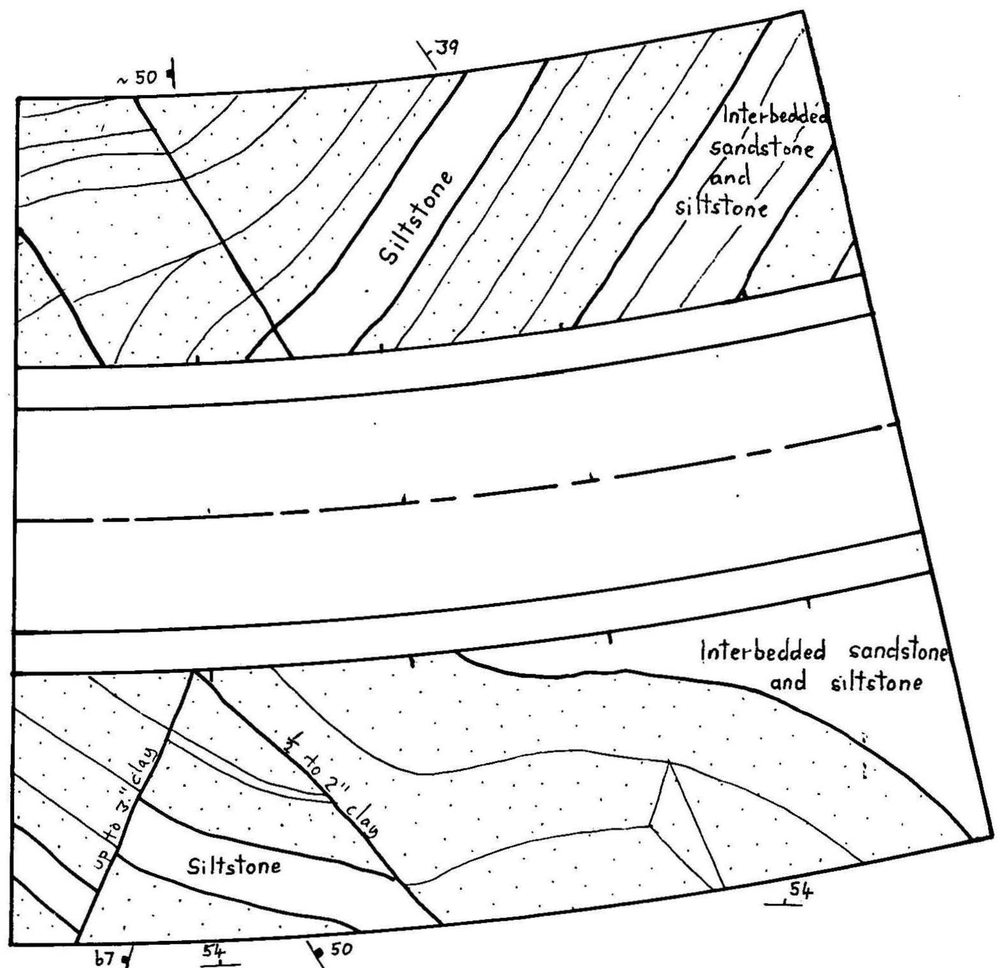
LOGGED : G.A.M. HENDERSON. DATE LOGGED : 22, 24, 27, 29/6/66. DRAWN : G.A.M. HENDERSON. To accompany Record 1969/111 I55/A16/571(10)

# CORIN DAM DIVERSION TUNNEL—GEOLOGICAL LOG.



0 10 20 30 FEET

| TUNNEL STATIONS | 12+00 | T.P. 12+46 | For section from 12+46 to 13+50 see Sheet 12 |
|---|---|---|---|
| DIRECTION, INVERT LEVEL, SLOPE | Direction: Slope: 0·053% Invert level: 2918 feet | | |
| PROGRESS HEADING AT, DATE | 16/6/66 \| 15/6/66 \| 14/6/66 \| 11/6/66 \| 10/6/66 | | |
| ROCK TYPE DEGREE OF WEATHERING OR ALTERATION | SANDSTONE AND SILTSTONE fresh | | |




| | | |
|---|---|---|
| STRUCTURES: JOINTING PATTERN–ATTITUDE SPACING, PERSISTENCE CHARACTER–SURFACE SEPARATIONS COATING; FAULTS SHEARED AND CRUSHED ZONES: ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL | Heavily stained joints present, often with a veneer of clay. Several clay seams up to 3" wide present. Joints spaced 3"-24", mostly 8"-15". | See following sheet |
| ROCK CONDITION | 3 | |
| PERCENT BREAKAGE ALONG JOINTS | 80-90% | 70-90% |

| GROUND WATER | |
|---|---|
| INITIAL QUANTITY, DATE | |
| QUANTITY DATE | |
| TEMPERATURE | |
| CUMULATIVE TOTAL million gal day | |

| EXCAVATION | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No of HOLES | 48:4 | 40:5 | 53:5 | 47:5 | 40:5 | 27:4 | 38:4 | 37:4 | 46:4 | 43:4 | 44:4 |
| EXPLOSIVES, lb | 68 | 73 | 70 | 67 | 68 | 42 | 56 | 54 | 65 | 57 | 63 |

| SUPPORT DURING CONSTRUCTION | |
|---|---|
| WEST WALL (Springing line) | |
| ROOF | |
| EAST WALL (Springing line) | |

LOGGED: G.A.M. HENDERSON. DATE LOGGED: 19/7/66. DRAWN: G.A.M. HENDERSON. To accompany Record 1969/111 I55/A16/571(11)

# CORIN DAM DIVERSION TUNNEL — GEOLOGICAL LOG.

0 10 20 30 FEET

**TUNNEL STATIONS** — For section from 12+00 to 12+46 see Sheet 11 — T.P. 12+46 — 13+00

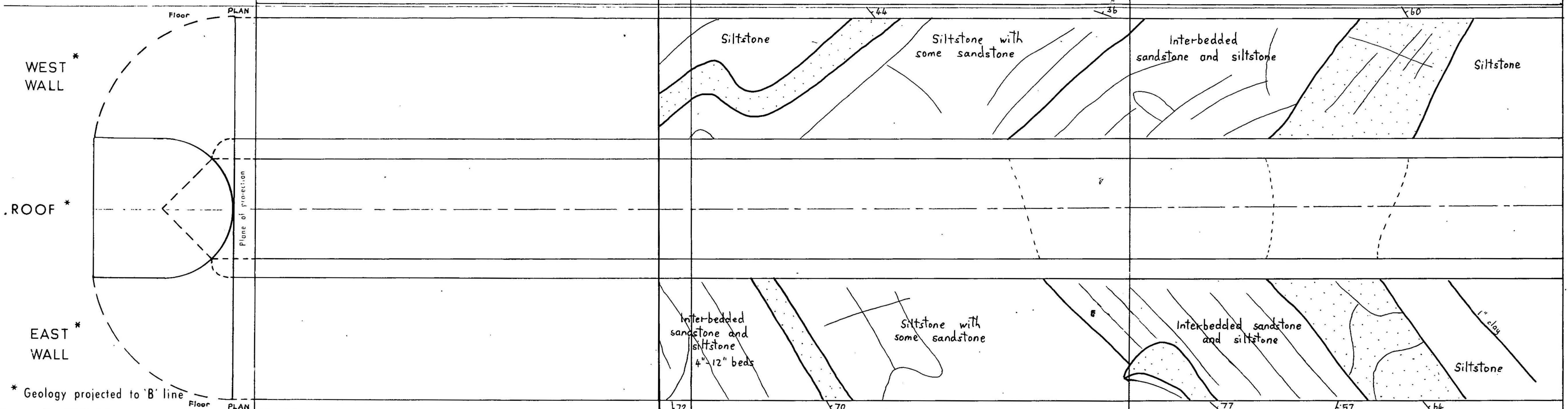
**DIRECTION INVERT LEVEL SLOPE** — Direction: 319° 48′ 38″ ← — Slope: 0·053% — Invert level: 2917 feet

**PROGRESS** HEADING AT DATE

| 10/6/66 | 9/6/66 | 8/6/66 | 3/6/66 | 2/6/66 | 1/6/66 | 31/5/66 | 30/5/66 | 28/5/66 | 27/5/66 | 26/5/66 | 25/5/66 | 24/5/66 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|

**ROCK TYPE**
DEGREE OF WEATHERING OR ALTERATION — SANDSTONE AND SILTSTONE fresh

**LOG OF FACE** AND **CROSS SECTIONS**
Looking in direction of tunnel stations showing OVERBREAK and CONTROL of SHAPE by JOINTS FAULTS SOFT SEAMS etc
Springing line




**STRUCTURES**
JOINTING PATTERN – ATTITUDE SPACING PERSISTENCE CHARACTER – SURFACE SEPARATIONS COATING
FAULTS HEALED AND CRUSHED ZONES ATTITUDE RELATIVE DISPLACEMENT WIDTH CHARACTER OF MATERIAL

Interbedded sandstone and siltstone with thin clay seams commonly present on interfaces. Bedding plane joints are the only well-developed set. Joints striking 30° to tunnel and dipping 75° to N. are developed in the sandstone beds. Joint spacing 1" to 24", commonly 3" to 12".

**ROCK CONDITION** — 3
PERCENT BREAKAGE ALONG JOINTS — 70-90%

**GROUND WATER** — INITIAL QUANTITY DATE; QUANTITY, DATE; TEMPERATURE; CUMULATIVE TOTAL million gal day

**EXCAVATION**

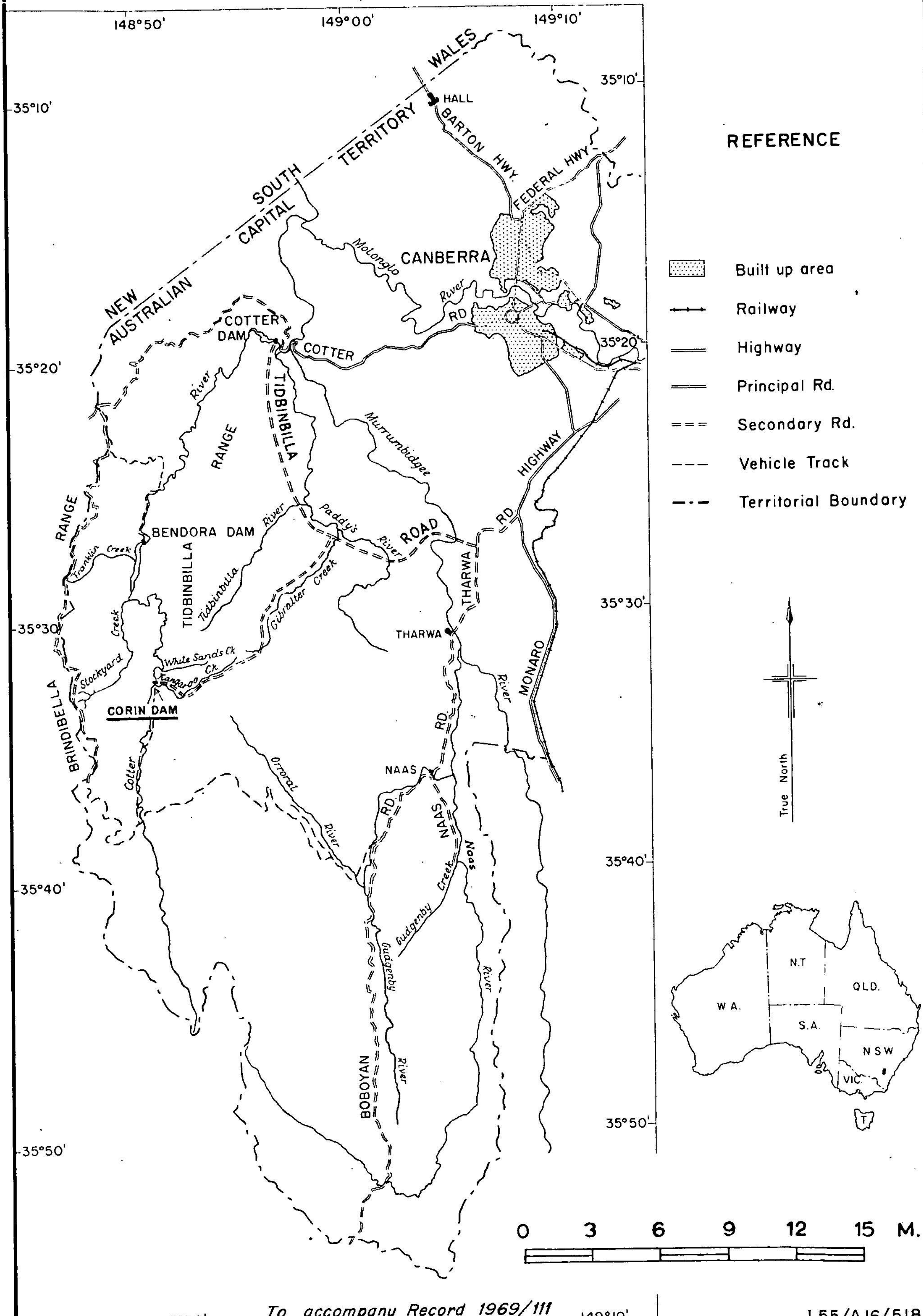
| | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ADVANCE No of HOLES | 46:4 | 41:4 | 48:4 | 48:4 | 47:4 | 43:4 | ? ? | 48:? | 54:4 | 55:4 | 48:4 | 50:4 | 63:4 | 48:4 | 72 | 21:2 | 55:5 | 58:5 | 61:3½ | 30:2 | 63:8 | 82:3 | 66:3 | 77:3 | 73 3 |
| EXPLOSIVES lb | 68 | 22 | 40 | 75 | 74 | 26 | ? | 65 | 40 | 40 | 37 | 40 | 38 | 29½ | 43 | 9½ | 90 | 35 | 40 | 6 | 75 | 40 | 34 | 37 | 9 |

**SUPPORT DURING CONSTRUCTION** — Springing line — WEST WALL; ROOF; EAST WALL

LOGGED: G.A.M. HENDERSON. DATE LOGGED: 19/7/66. DRAWN: G.A.M. HENDERSON. *To accompany Record 1969/111* I 55/A16/571 (12)

PLATE 1

# CORIN DAM LOCALITY MAP




*To accompany Record 1969/111*

I 55/A16/518

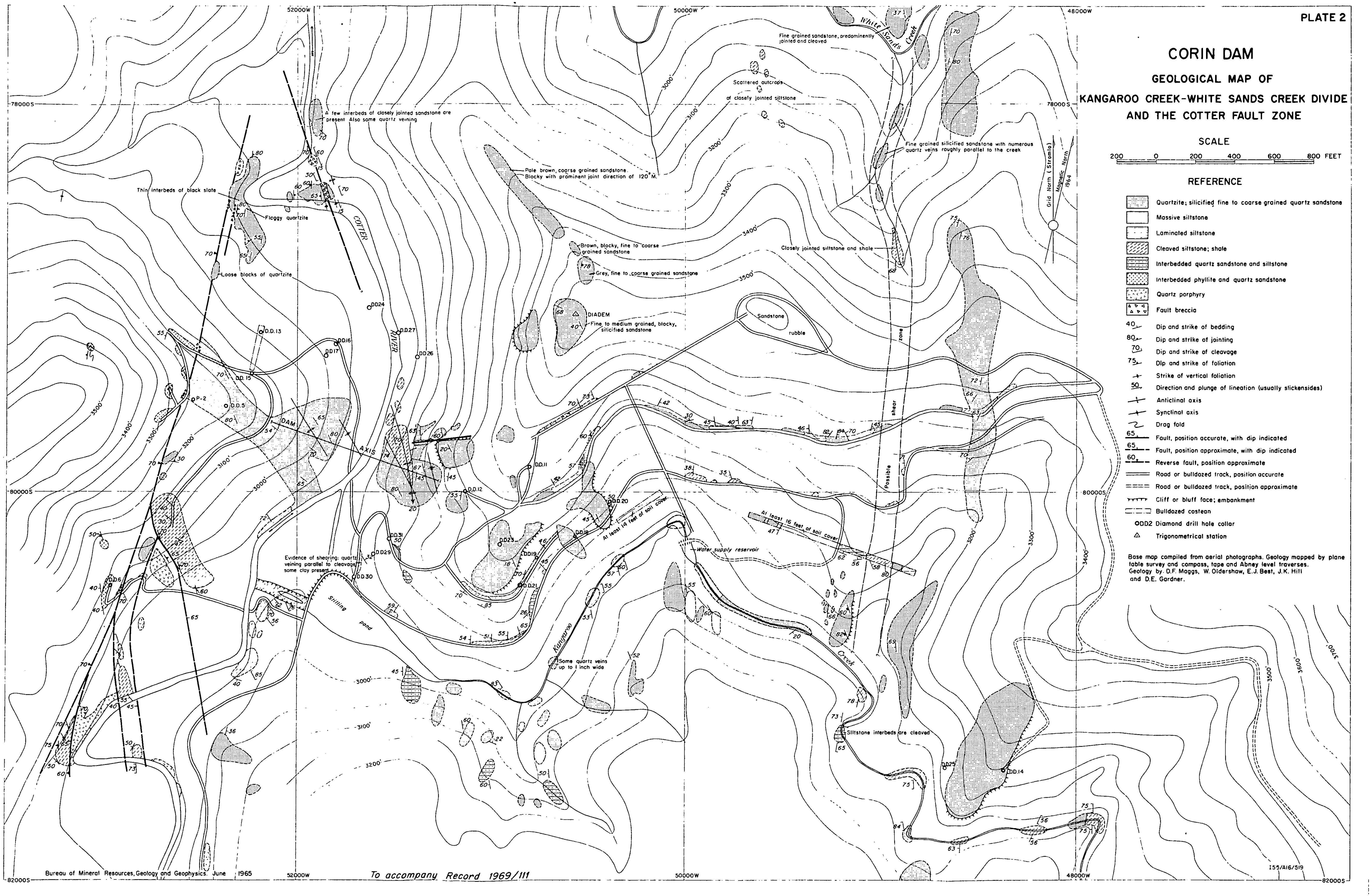
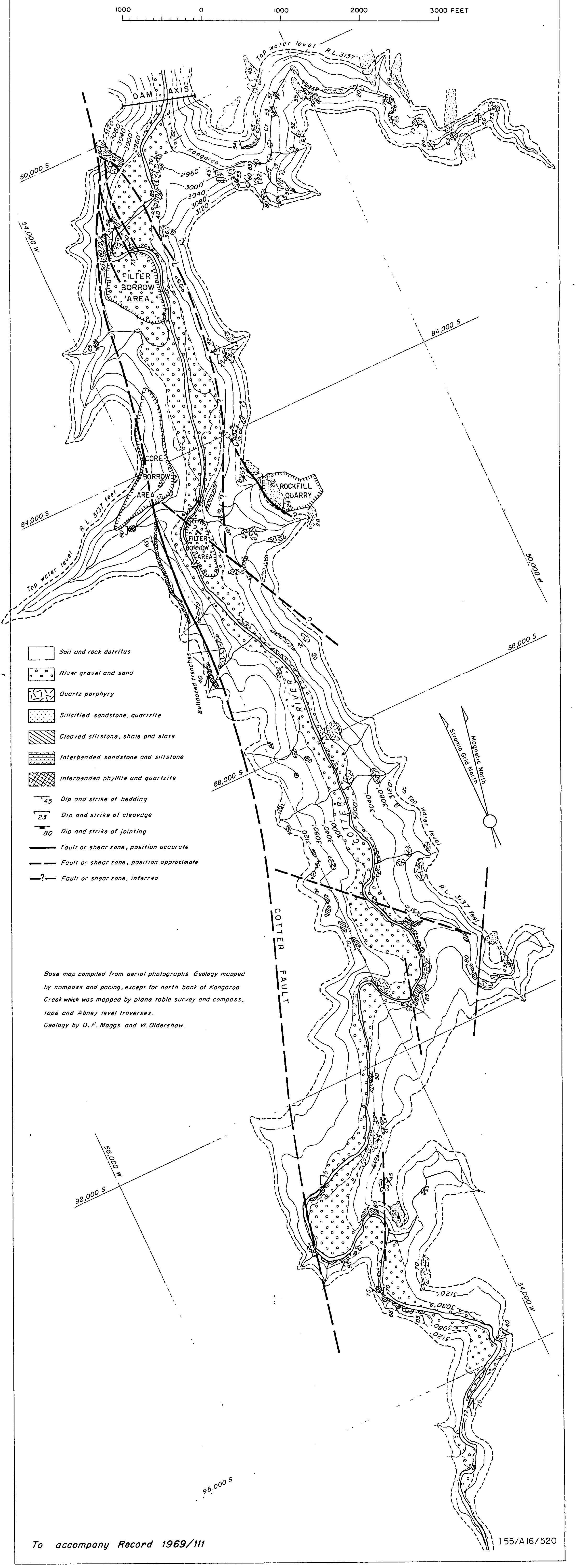


PLATE 2
CORIN DAM
GEOLOGICAL MAP OF
KANGAROO CREEK-WHITE SANDS CREEK DIVIDE
AND THE COTTER FAULT ZONE
SCALE
200 0 200 400 600 800 FEET
REFERENCE
Quartzite; silicified fine to coarse grained quartz sandstone
Massive siltstone
Laminated siltstone
Cleaved siltstone; shale
Interbedded quartz sandstone and siltstone
Interbedded phyllite and quartz sandstone
Quartz porphyry
Fault breccia
Dip and strike of bedding
Dip and strike of jointing
Dip and strike of cleavage
Dip and strike of foliation
Strike of vertical foliation
Direction and plunge of lineation (usually slickensides)
Anticlinal axis
Synclinal axis
Drag fold
Fault, position accurate, with dip indicated
Fault, position approximate, with dip indicated
Reverse fault, position approximate
Road or bulldozed track, position accurate
Road or bulldozed track, position approximate
Cliff or bluff face; embankment
Bulldozed costean
DD2 Diamond drill hole collar
Trigonometrical station
Base map compiled from aerial photographs. Geology mapped by plane table survey and compass, tape and Abney level traverses.
Geology by D.F. Maggs, W. Oldershaw, E.J. Best, J.K. Hill and D.E. Gardner.
Grid North (Stromlo)
Magnetic North 1964
52000W
50000W
48000W
78000S
80000S
82000S
White Sands Creek
Fine grained sandstone, predominently jointed and cleaved
Scattered outcrops of closely jointed siltstone
A few interbeds of closely jointed sandstone are present. Also some quartz veining
Fine grained silicified sandstone with numerous quartz veins roughly parallel to the creek
Pale brown, coarse grained sandstone. Blocky with prominent joint direction of 120° M.
Thin interbeds of black slate
Flaggy quartzite
COTTER
RIVER
Brown, blocky, fine to coarse grained sandstone
Grey, fine to coarse grained sandstone
Closely jointed siltstone and shale
Loose blocks of quartzite
DIADEM
Fine to medium grained, blocky, silicified sandstone
Sandstone rubble
DAM AXIS
Possible shear zone
At least 14 feet of soil cover
At least 16 feet of soil cover
Water supply reservoir
Evidence of shearing: quartz veining parallel to cleavage, some clay present
Stilling pond
Kangaroo
Creek
Some quartz veins up to 1 inch wide
Siltstone interbeds are cleaved
155/A16/519
Bureau of Mineral Resources, Geology and Geophysics. June 1965
To accompany Record 1969/111

# CORIN DAM
## GEOLOGICAL PLAN OF RESERVOIR AREA

PLATE 3.

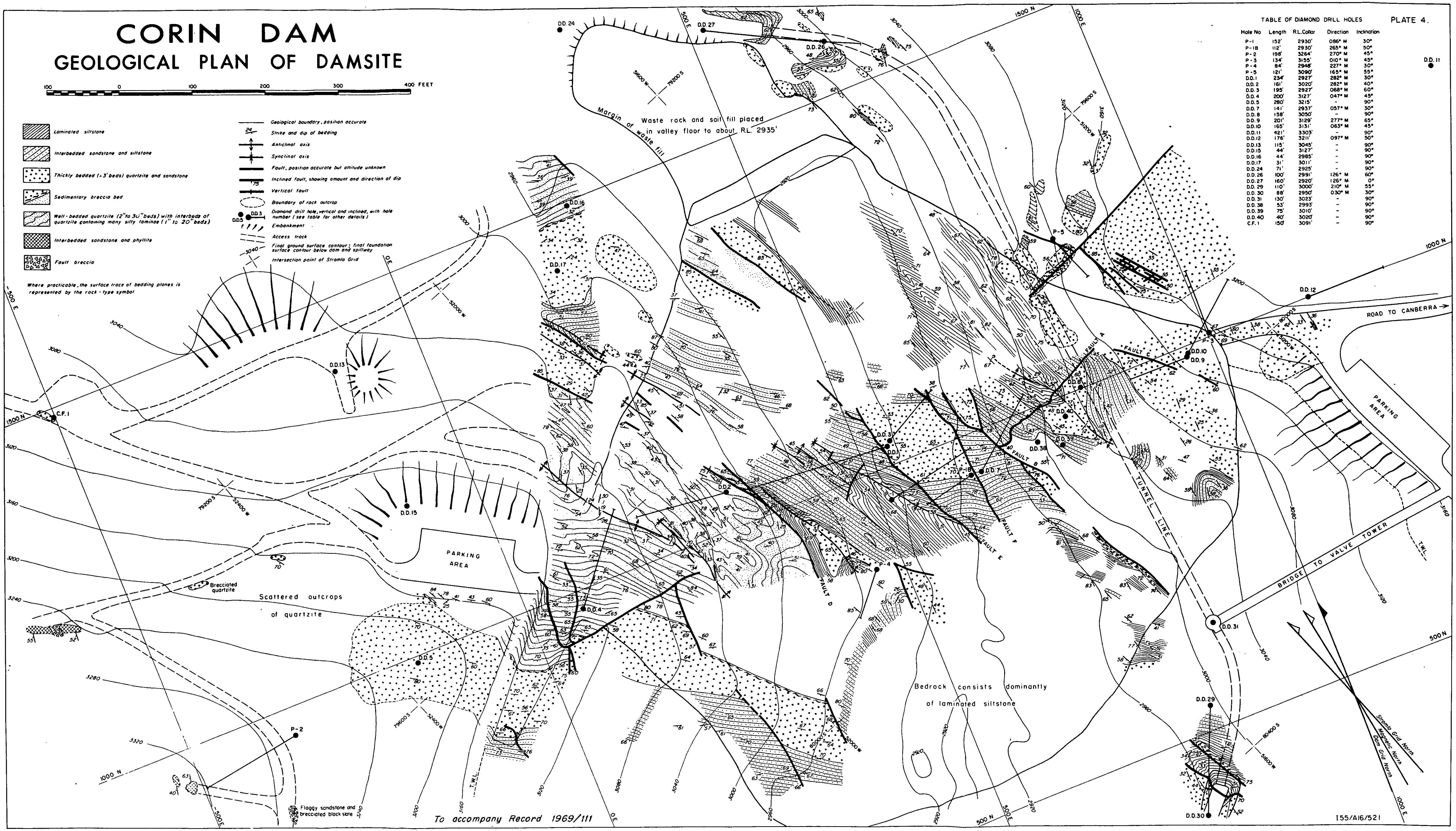



To accompany Record 1969/111

I55/A16/520

# CORIN DAM
## GEOLOGICAL PLAN OF DAMSITE

PLATE 4.




### TABLE OF DIAMOND DRILL HOLES

| Hole No | Length | R.L.Collar | Direction | Inclination |
|---|---|---|---|---|
| P-1 | 152' | 2930' | 086° M | 30° |
| P-1B | 112' | 2930' | 265° M | 50° |
| P-2 | 198' | 3264' | 270° M | 45° |
| P-3 | 134' | 3155' | 010° M | 45° |
| P-4 | 84' | 2948' | 227° M | 30° |
| P-5 | 121' | 3090' | 165° M | 55° |
| D.D.1 | 234' | 2927' | 282° M | 30° |
| D.D.2 | 161' | 3020' | 282° M | 40° |
| D.D.3 | 195' | 2927' | 068° M | 60° |
| D.D.4 | 200' | 3127' | 047° M | 45° |
| D.D.5 | 280' | 3215' | - | 90° |
| D.D.7 | 141' | 2937' | 057° M | 30° |
| D.D.8 | 158' | 3050' | - | 90° |
| D.D.9 | 201' | 3129' | 277° M | 65° |
| D.D.10 | 165' | 3131' | 063° M | 45° |
| D.D.11 | 421' | 3303' | - | 90° |
| D.D.12 | 176' | 3211' | 097° M | 50° |
| D.D.13 | 115' | 3045' | - | 90° |
| D.D.15 | 44' | 3127' | - | 90° |
| D.D.16 | 44' | 2985' | - | 90° |
| D.D.17 | 31' | 3011' | - | 90° |
| D.D.24 | 71' | 2925' | - | 90° |
| D.D.26 | 100' | 2991' | 126° M | 60° |
| D.D.27 | 160' | 2920' | 126° M | 0° |
| D.D.29 | 110' | 3000' | 210° M | 55° |
| D.D.30 | 88' | 2950' | 030° M | 30° |
| D.D.31 | 130' | 3023' | - | 90° |
| D.D.38 | 53' | 2993' | - | 90° |
| D.D.39 | 75' | 3010' | - | 90° |
| D.D.40 | 40' | 3020' | - | 90° |
| C.F.1 | 150' | 3091' | - | 90° |

To accompany Record 1969/111

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PLATE 5
CORIN DAM - GEOLOGICAL SECTION OF DAMSITE
LINE OF SECTION IS 1,000 N DAM GRID LINE
40 0 40 80 120 160 200 240 280
FEET
WEST
EAST
DAM CREST LEVEL - 3155 FEET
TOP WATER LEVEL - 3137 FEET
Concrete-lined spillway channel
Concrete buttress
P-2
?
D.D.5
D.D.4
D.D.2
P-1
D.D.1
D.D.7
D.D.8
D.D.9
D.D.10
D.D.12
Fault A
Fault B
Fault C
Fault D
Fault E
Fault F
Anticlinal axis
Synclinal axis
Anticlinal axis
Diversion tunnel
3200'
3100'
3000'
2900'
3240'
3220'
3200'
3180'
3160'
3140'
3120'
3100'
3080'
3060'
3040'
3020'
3000'
2980'
2960'
2940'
2920'
2900'
2880'
2860'
2840'
Laminated siltstone
Interbedded sandstone and siltstone
Thickly-bedded sandstone and quartzite
Well-bedded quartzite (4" to 30" beds) with interbeds of quartzite containing numerous silty laminae (2" to 20" beds)
Fault breccia
The attitude of bedding is indicated by the rock-type symbol.
Fault, position accurate
Fault, position approximate
Shear zone
D.D.2
Diamond drill hole, projected into plane of the section
The lithological data from diamond drill holes are projected into the plane of the section along prevailing geological strike
Several drill holes intersect plunging fold axes or fault planes with strikes different to bedding strike; this accounts for the breaks in the drill hole information, indicated by broken lines.
Dips of bedding measured from drill core are represented as apparent dips in the plane of the section; these were calculated using strike directions measured at the surface near the drill holes
To accompany Record 1969/111
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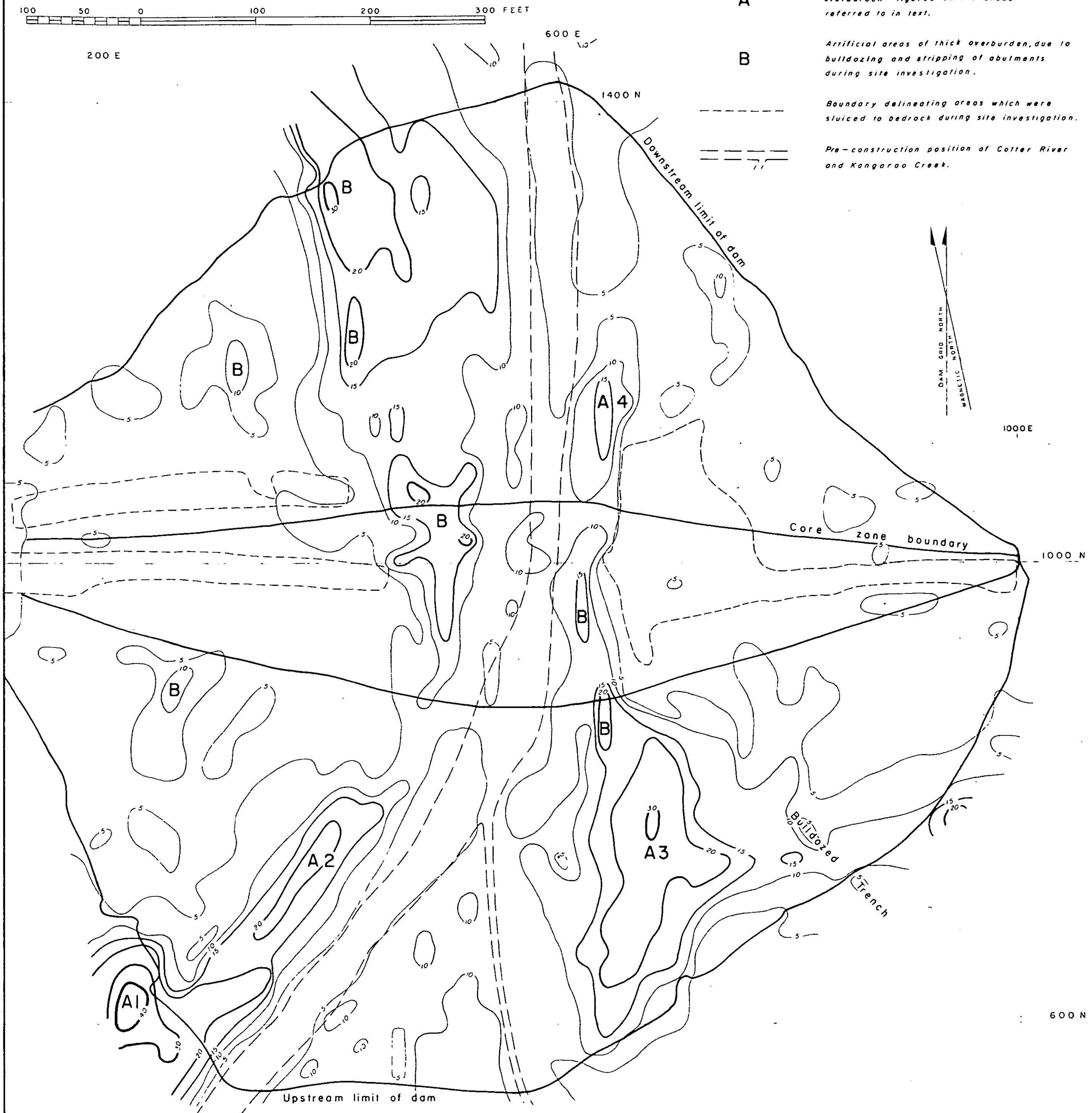
# CORIN DAM
## DISTRIBUTION OF OVERBURDEN REMOVED DURING CONSTRUCTION




To accompany Record 1969/111

PLATE 7

# CORIN DAM – GEOLOGY OF VALLEY FLOOR CORE ZONE FOUNDATIONS


Thinly bedded (2"-30") quartzite containing numerous silty laminae.

Thickly bedded (>20") sandstone and quartzite.

Thinly interbedded sandstone and siltstone.

Laminated siltstone.

Where possible, the above symbols are used to illustrate the trace of bedding in the excavation.

Strike and dip of bedding

Strike of vertical bedding

Strike and dip of jointing

Trace of joint, showing strike and/or dip if known

Trace of vertical joint

Trace of anticlinal axis

Trace of synclinal axis

Trace of fault, showing dip if known. Thickness of seams of clay and crushed rock (cr. rk.) are indicated if significant

Fault zone

Areal extent of prominent joint plane, showing strike and dip

Areal extent of prominent fault plane

Contour of final rock excavation, in feet

To accompany Record 1969/111

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# CORIN DAM – PLAN SHOWING SURFACE TREATMENT OF CORE ZONE FOUNDATIONS

PLATE 8


To accompany Record 1969/111

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# CORIN DAM

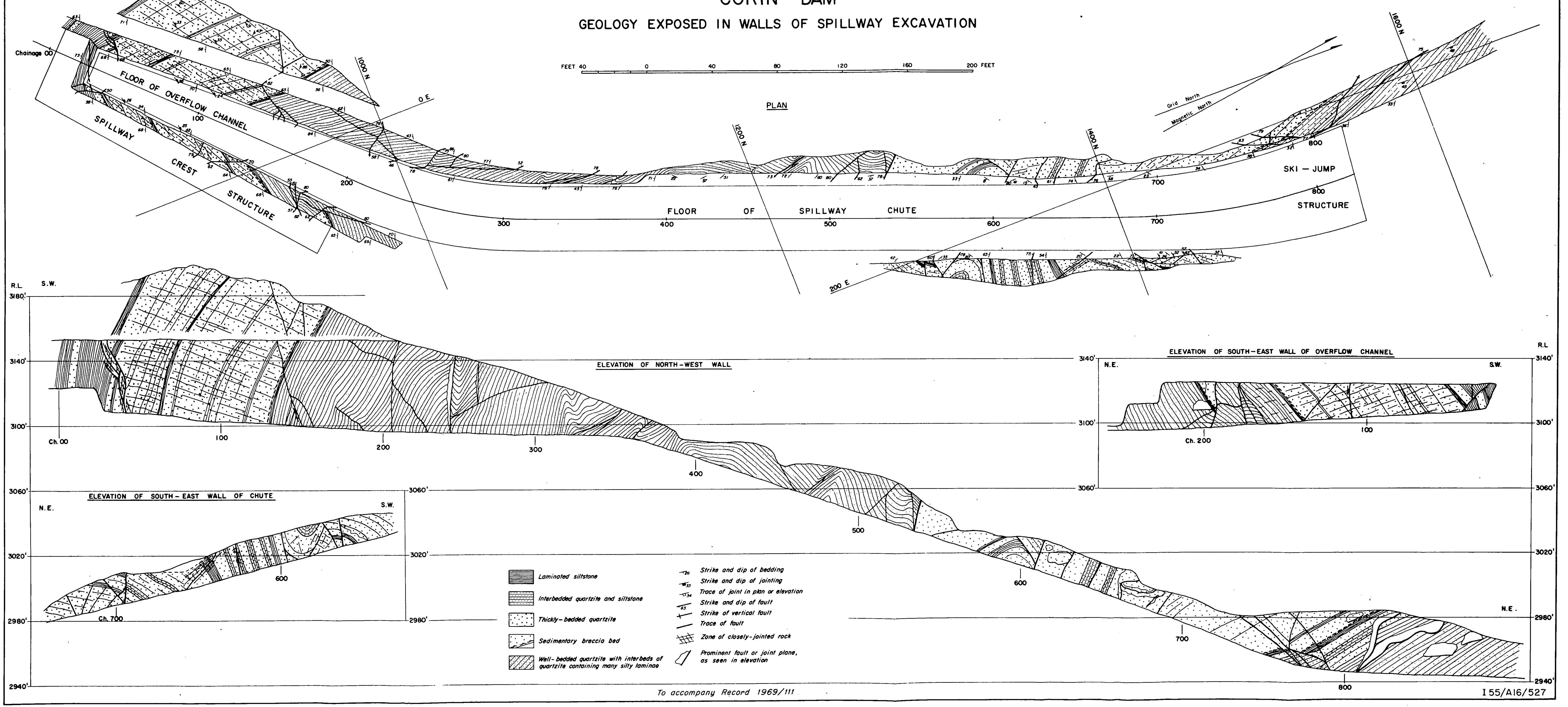
## GEOLOGICAL PLAN OF SPILLWAY EXCAVATION


To accompany Record 1969/111

# CORIN DAM

## GEOLOGY EXPOSED IN WALLS OF SPILLWAY EXCAVATION

PLATE 10

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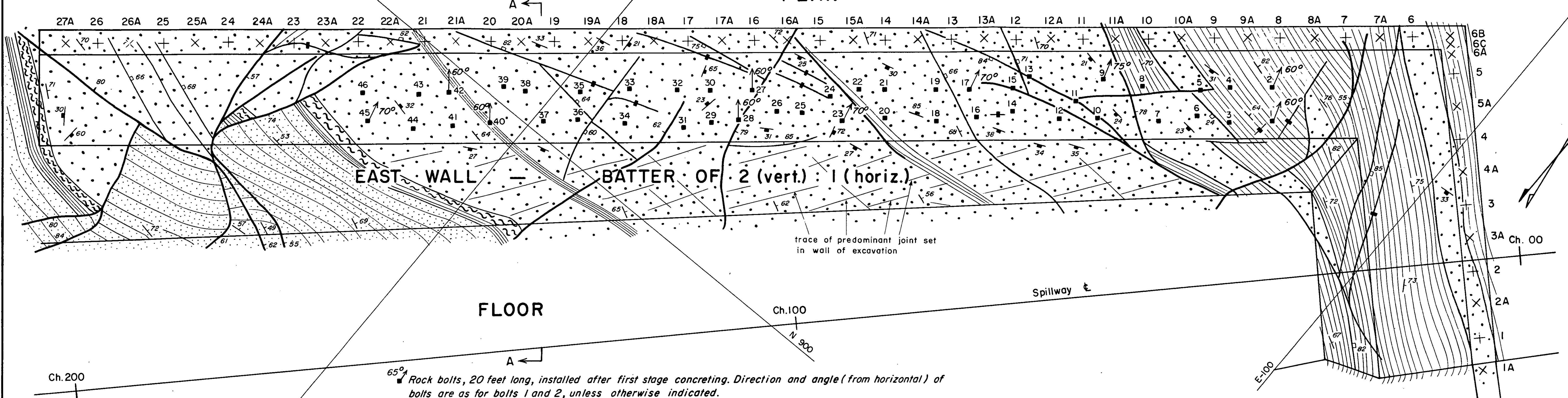
# CORIN DAM — ENGINEERING GEOLOGY OF SPILLWAY CREST

## PLAN

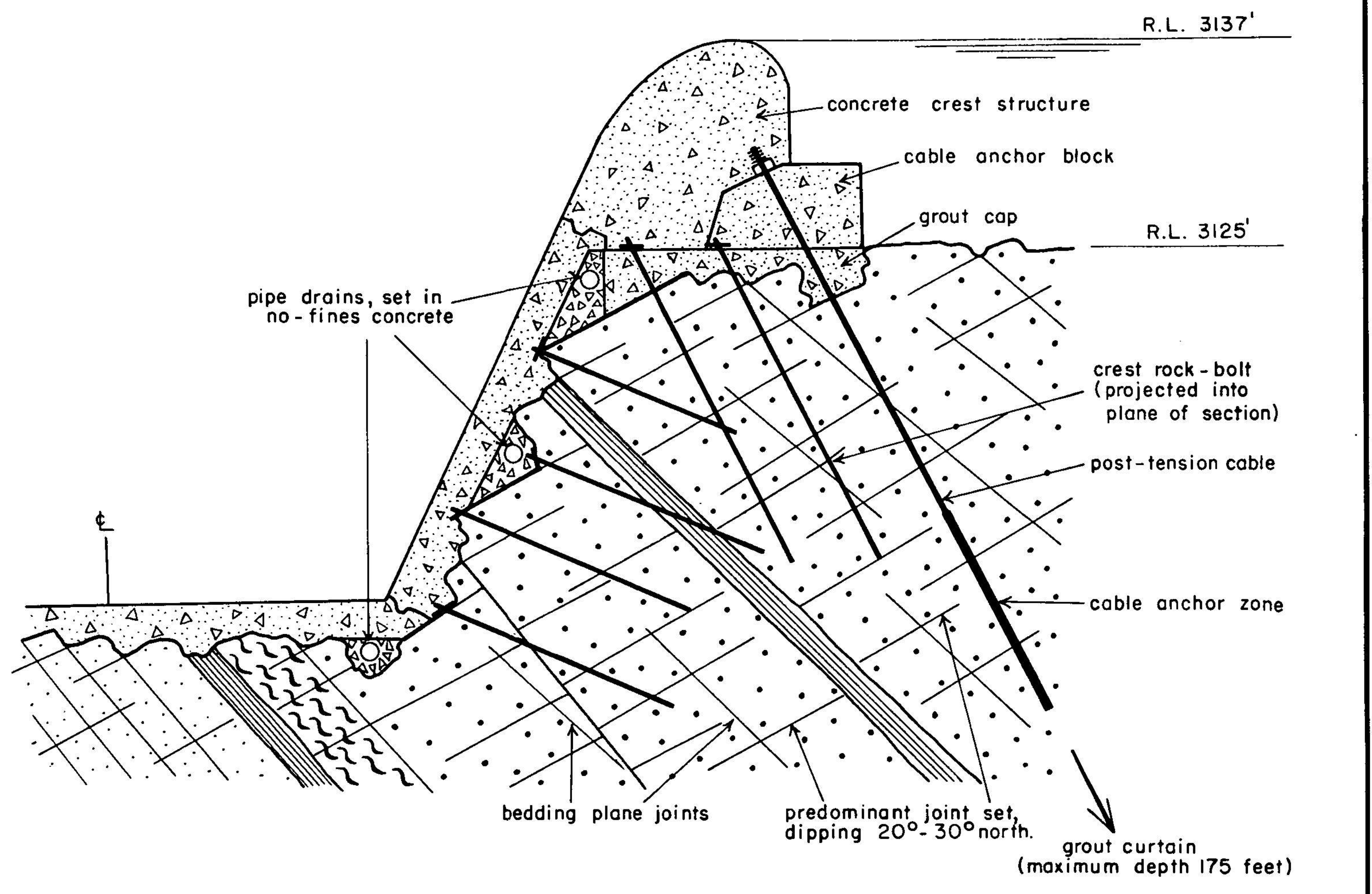



65° Rock bolts, 20 feet long, installed after first stage concreting. Direction and angle (from horizontal) of bolts are as for bolts 1 and 2, unless otherwise indicated.

+ Post-tension cable hole, 35 feet long; used as a quaternary curtain grout hole prior to cable installation. Holes 1 to 5 are vertical; holes 6 to 26 dip at 65° in an easterly direction, perpendicular to the crest. (see section)

× Curtain grout hole. Attitude of holes are same as corresponding cable hole, except that 6A, 6C and 6B dip at 82°, 77°, and 72° respectively.

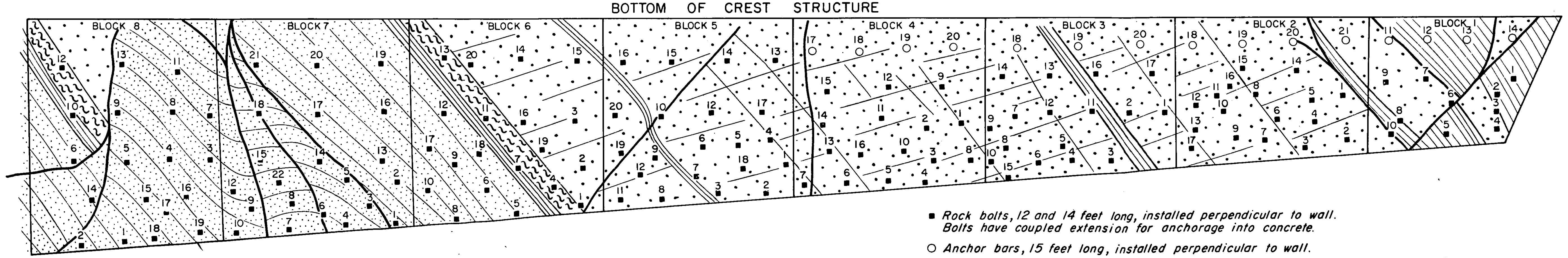
## TYPICAL SECTION (AA) LOOKING DOWNSTREAM





## DEVELOPED ELEVATION OF EAST WALL

BOTTOM OF CREST STRUCTURE




■ Rock bolts, 12 and 14 feet long, installed perpendicular to wall. Bolts have coupled extension for anchorage into concrete.

○ Anchor bars, 15 feet long, installed perpendicular to wall.

Laminated siltstone

Thickly-bedded sandstone and quartzite

Sedimentary breccia bed

Well-bedded quartzite with thin interbeds of quartzite containing numerous silty laminae

67 Strike and dip of bedding

38 Strike and dip of jointing

Strike of vertical joint

35 Trace of joint, showing true dip

66 Trace of fault, showing true dip

Engineering information based on C.D.W plans. System of numbering for rock bolt holes, grout holes and cable holes refers to C.D.W working drawings.
Details of foundation drainage layout are omitted for the sake of clarity.

To accompany Record 1969/111

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# CORIN DAM

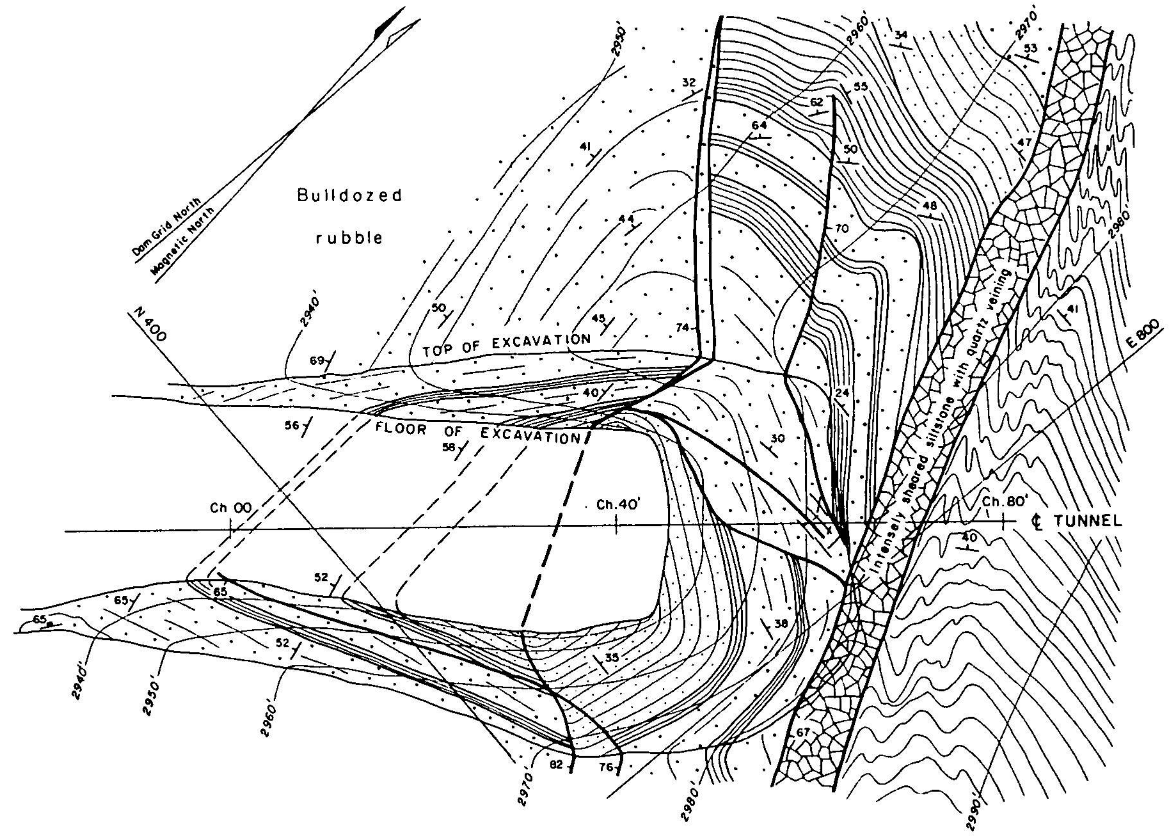
## GEOLOGY OF INLET PORTAL EXCAVATION

FEET 20 0 20 40 60 80 100 FEET

- Silicified sandstone; quartzite
- Interbedded sandstone and siltstone
- Laminated siltstone
- Fault zone
- Closely-jointed rock
- Strike and dip of bedding
- Strike and dip of jointing
- Trace of fault, position accurate, showing dip if known
- Trace of fault, position approximate
- Position of rock bolt
- Contour of rock surface

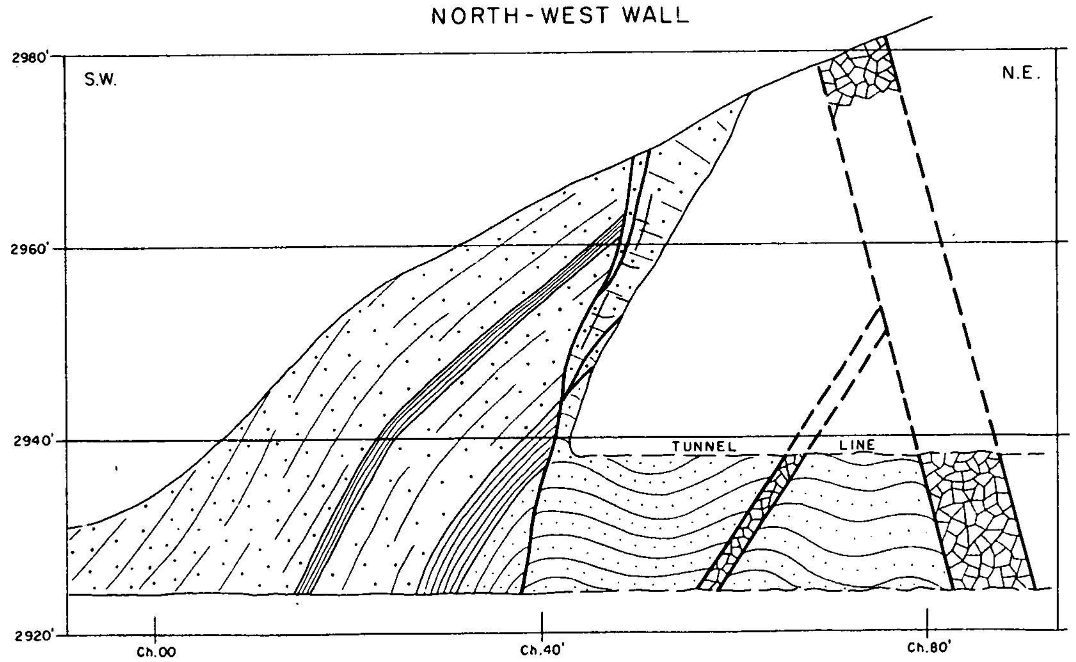
Note: Where possible, the trace of bedding in plan and elevations is indicated by rock type symbols

## PLAN OF PORTAL EXCAVATION

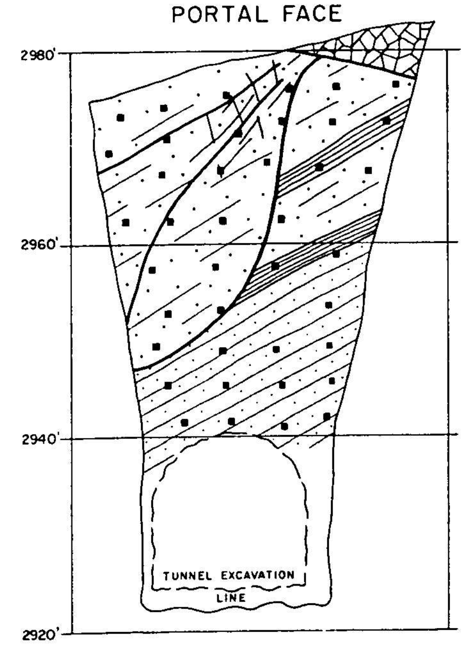



## ELEVATIONS OF THE WALLS AND FACE OF THE PORTAL EXCAVATION

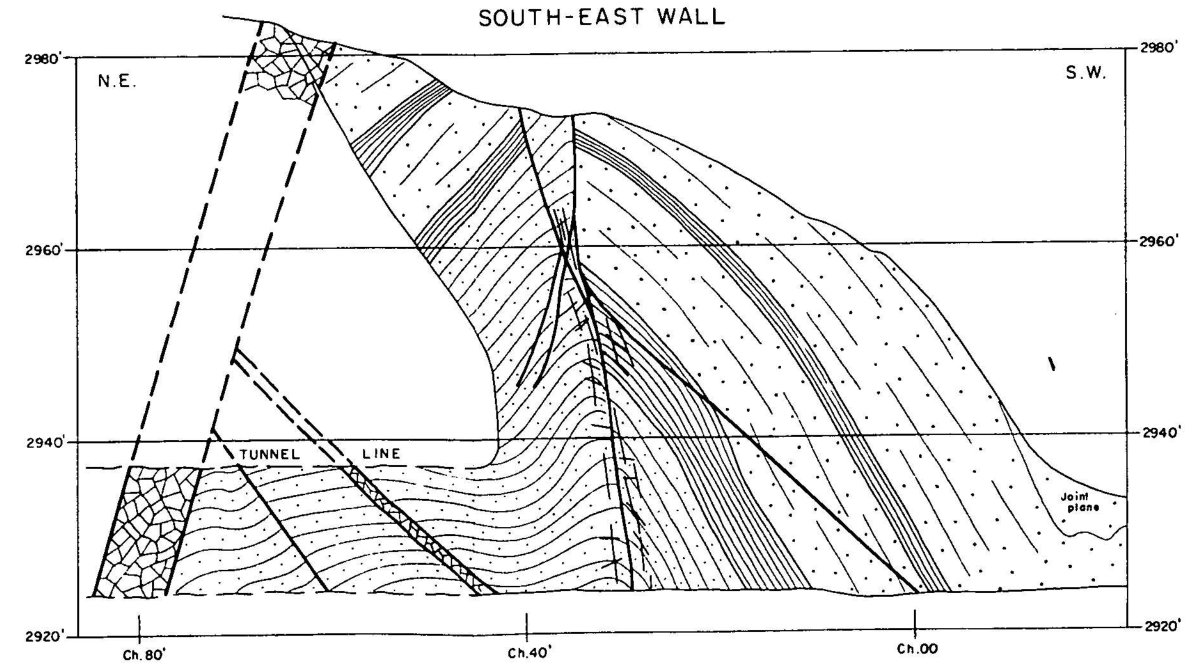
### NORTH-WEST WALL




### PORTAL FACE




### SOUTH-EAST WALL

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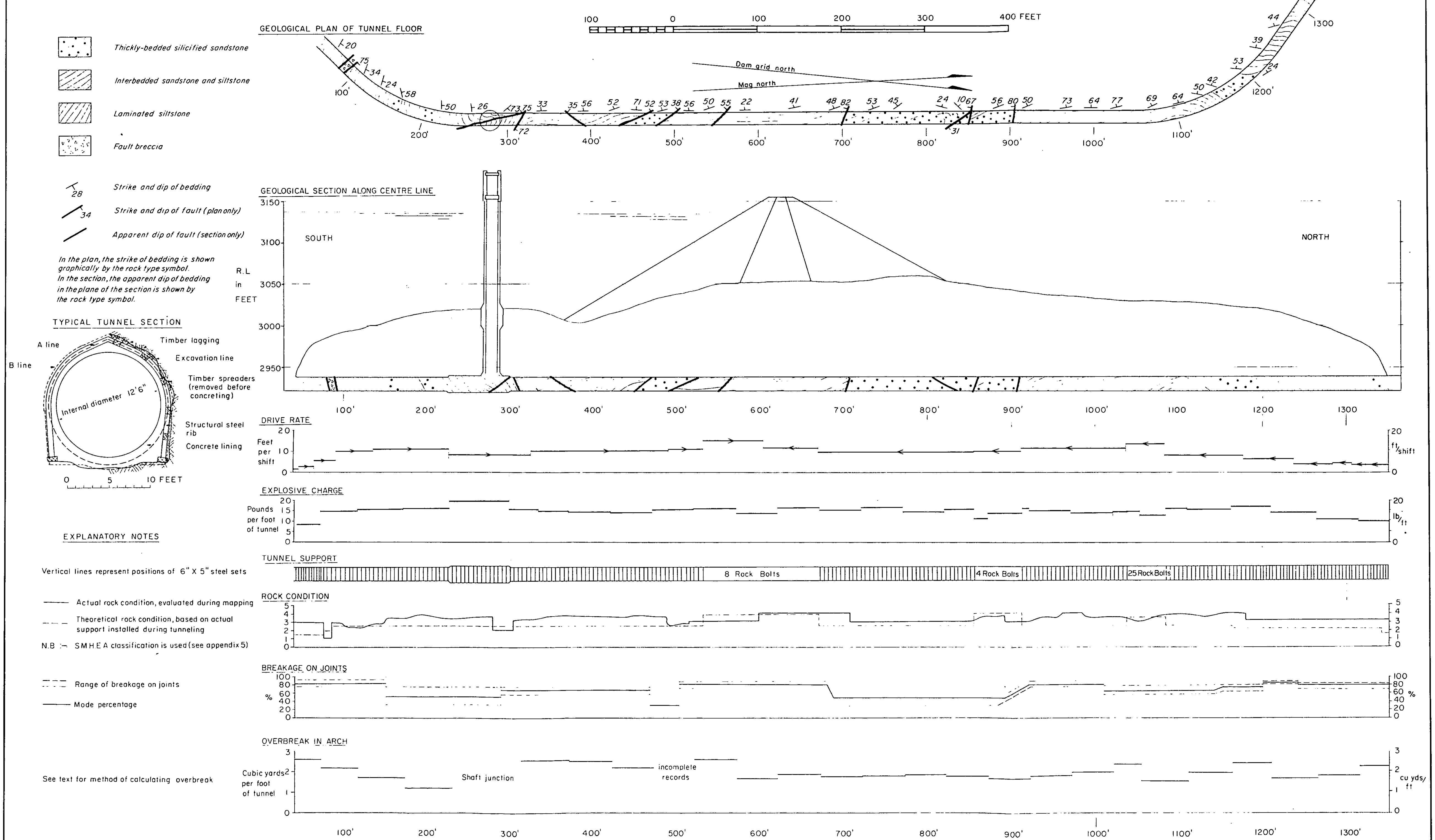
*To accompany Record 1969/111*

PLATE 13

# CORIN DAM – ENGINEERING GEOLOGY OF DIVERSION TUNNEL

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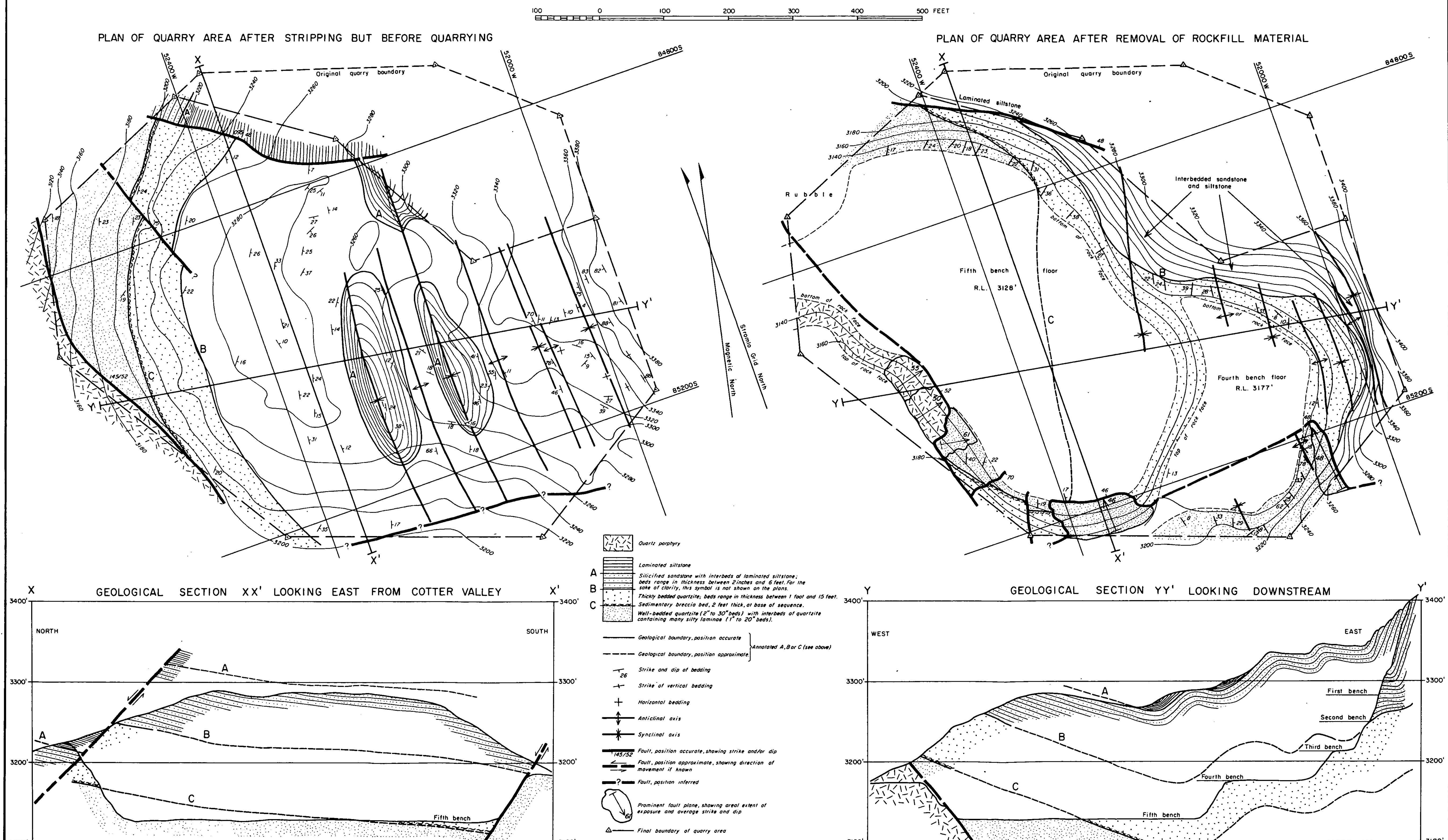
# CORIN DAM — GEOLOGY AND TOPOGRAPHY OF ROCKFILL QUARRY

PLATE 14




To accompany Record 1969/111

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PLATE 15

# CORIN DAM — "A" HOLE CURTAIN GROUTING


*To accompany Record 1969/111*

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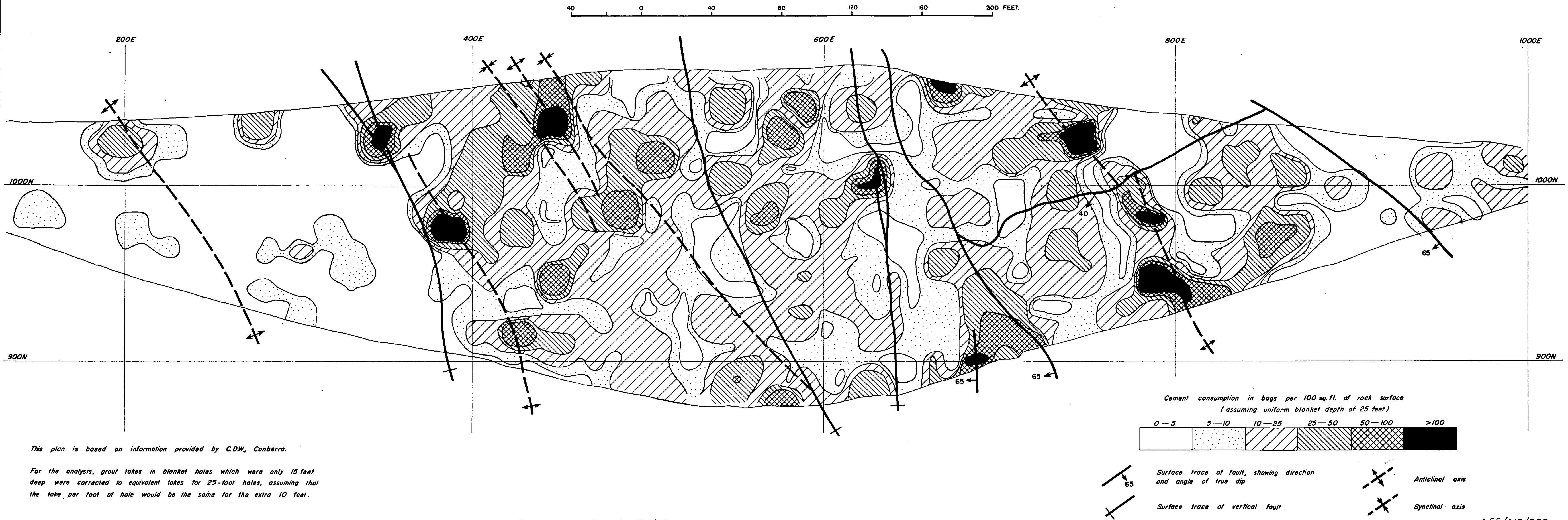
# CORIN DAM 'B' HOLE BLANKET GROUTING

## ANALYSIS OF GROUT CONSUMPTION, SHOWING CORRELATION OF HIGH TAKES WITH GEOLOGICAL FEATURES



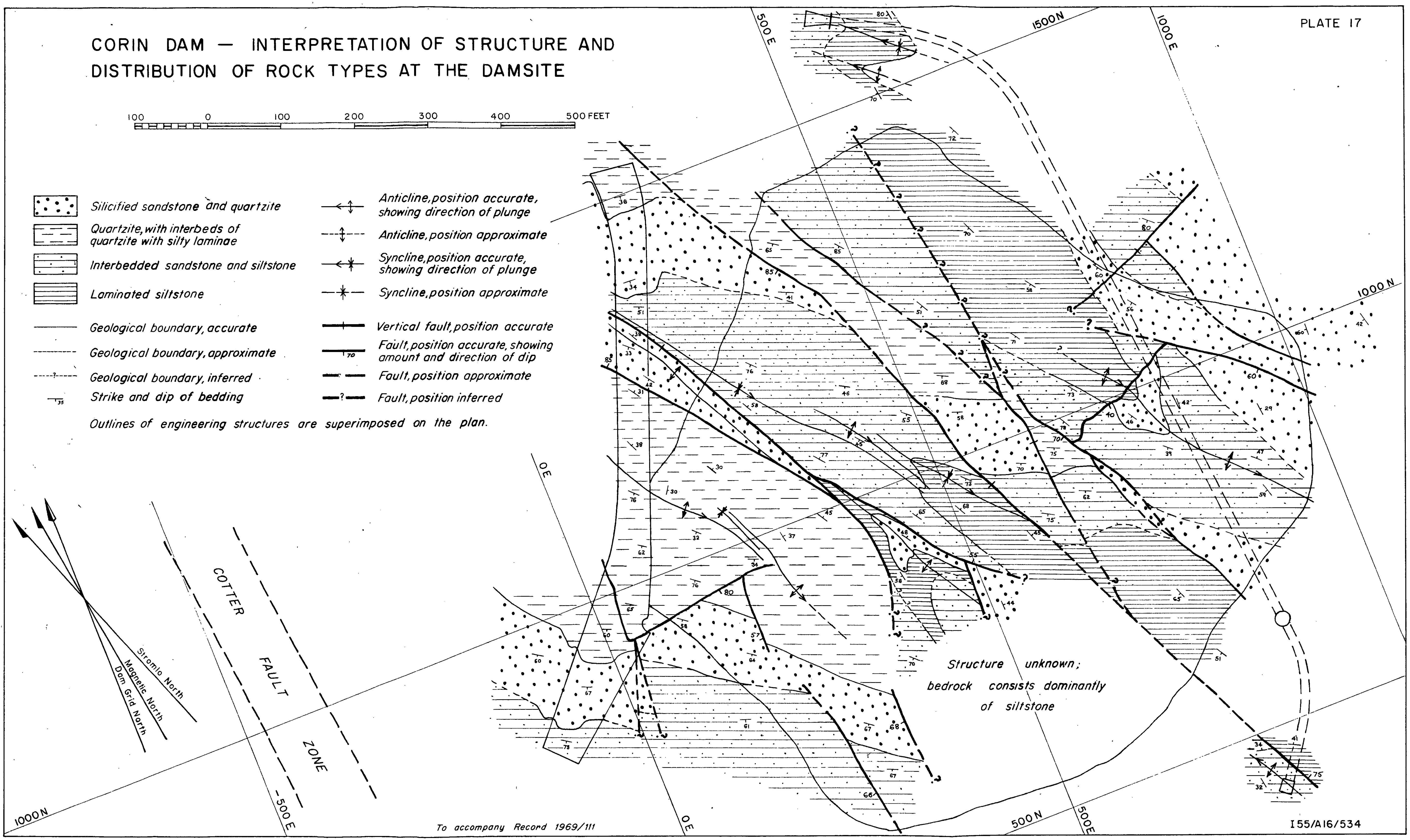

*This plan is based on information provided by C.D.W., Canberra.*

*For the analysis, grout takes in blanket holes which were only 15 feet deep were corrected to equivalent takes for 25-foot holes, assuming that the take per foot of hole would be the same for the extra 10 feet.*

*To accompany Record 1969/111*

I 55/A16/533

# CORIN DAM — INTERPRETATION OF STRUCTURE AND DISTRIBUTION OF ROCK TYPES AT THE DAMSITE



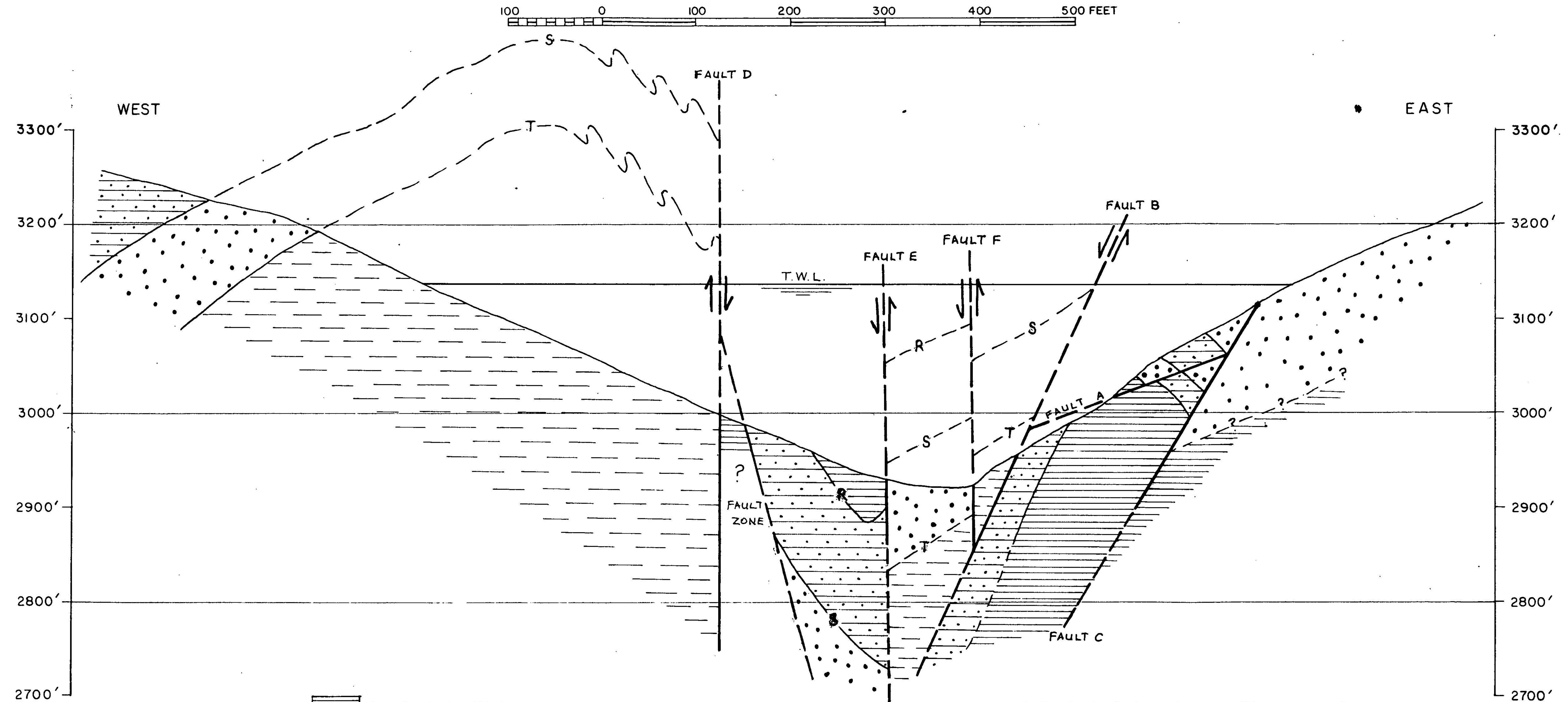

To accompany Record 1969/111

I55/A16/534

PLATE 18

# CORIN DAM

## GEOLOGICAL SECTION ALONG 1000 N SHOWING INTERPRETATION OF STRUCTURE




*Laminated siltstone*

*Interbedded sandstone and siltstone*

*Thickly bedded silicified sandstone ; quartzite*

*Quartzite with interbeds containing silty laminae*

*Correlated geological boundaries are annotated R, S & T.*

*Geological boundary, position accurate*

*Geological boundary, position approximate*

*Geological boundary, position inferred*

*Fault, position accurate*

*Fault, position approximate*

*Annotated faults refer to descriptions in text.*

*To accompany Record 1969/111*

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