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ENGINEERING GEOLOGY OF THE KANGAROO CREEK DAM

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Letter of Transmittal

Geological Survey Office,
Department of Mines,
Adelaide,
South Australia 5000

11*th December,* 1972

Sir,

I submit a report by J. P. Trudinger, Assistant Senior Geologist, on the Engineering Geology of the Kangaroo Creek Dam.

The need for a second damsite in the Torrens Gorge was recognized by the Engineering and Water Supply Department many years ago. The first geological investigations in the vicinity of the present dam were carried out by the Mines Department in 1949. *Investigations recommenced ten years later and led to final selection of the present dam site, based on geological factors. A critical examination of this site in the mid* 1960*'s led to design changes from a concrete arch dam to the concrete faced rockfill embankment structure, construction of which was completed in* 1969.

Because of geological features of the site, and of their interrelationship to engineering design, officers of the Engineering Geology Section of the Department of Mines were associated with the project from about 1964. *During construction, the author was stationed at the site as project geologist, to advise on day to day problems, and to record the geology of the site as construction progressed.*

As a result, the recording of this project has been the most thorough ever attempted in South Australia.

In the successful completion of the dam, due credit must be given to the design and construction engineers of the E. & W.S. Department and to the construction contractors. However, their task must have been assisted very materially by the continuous availability of a trained engineering geologist throughout the construction period. The present report will be a valuable reference for future engineering projects of a similar nature, involving difficult site conditions.

Approval is sought to publish this report as a Bulletin of the Geological Survey of South Australia.

BRUCE P. WEBB, Government Geologist

To the Honourable D. A. Dunstan, M.P.,
Minister of Development and Mines.

D. A. DUNSTAN, Minister of Development and Mines

Frontispiece—
Kangaroo Creek Dam, looking west with City of Adelaide in background.

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ENGINEERING GEOLOGY OF THE KANGAROO CREEK DAM

SUMMARY

The Kangaroo Creek Dam, located in the Mount Lofty Ranges 14 miles northeast of Adelaide, has been constructed to provide additional storage for the Adelaide water supply.

Geological factors led to the selection of the Kangaroo Creek Dam site instead of two other localities. Consideration of the effects of geological weaknesses resulted in rejection of the initial concrete arch dam design in favour of a concrete faced rockfill embankment. The bulk of the embankment was constructed of weak schistose rock from the excavation for the spillway which was located on the left bank.

The results of systematic geological investigations carried out throughout the construction of the project were recorded to provide a comprehensive factual account of geological conditions at the site. The detailed understanding which emerged from these investigations of the nature and properties of the rock substances and rock masses at the site, contributed to the solution of many of the day to day problems of construction and led to several significant modifications of the design of some features of the project.

The presence of clay-filled seams in the left bank resulted in numerous minor stability problems in the excavation batters, and necessitated numerous foundation treatments beneath the concrete structures.

Consideration of the mechanism of past slide movements on the left bank suggested that further movements would probably occur under operating conditions. Measures were undertaken to protect several project features from damage which could result from rock slides.

The major problems involved in the excavation and placement of the rockfill materials were attributable to the weak, anisotropic nature of the schistose materials at the site and to the complex weathering profiles.

High densities and shear strengths were developed in the embankment as a result of the high degree of compaction which was achieved by breakdown of the weak, schistose materials. Stronger materials placed beneath the upstream face and at the base of the embankment ensure that these zones have adequate permeability to accommodate any leakage from the reservoir.

Consideration of the effects of geological factors on the efficiency and applicability of the various construction techniques led to several improvements during construction, and suggested other ways in which the techniques could be improved for future projects.

INTRODUCTION

The Kangaroo Creek Dam is located in the River Torrens Gorge, 14 miles northeast of Adelaide (Fig. 1). The dam has been constructed as part of the metropolitan water supply scheme, to store water pumped from the River Murray *via* the Mannum-Adelaide Pipeline, as well as increasing the annual yield of the River Torrens through 112 square miles of catchment area. At full supply level (800 feet above sea level) the reservoir will contain 6,000 million gallons of water.

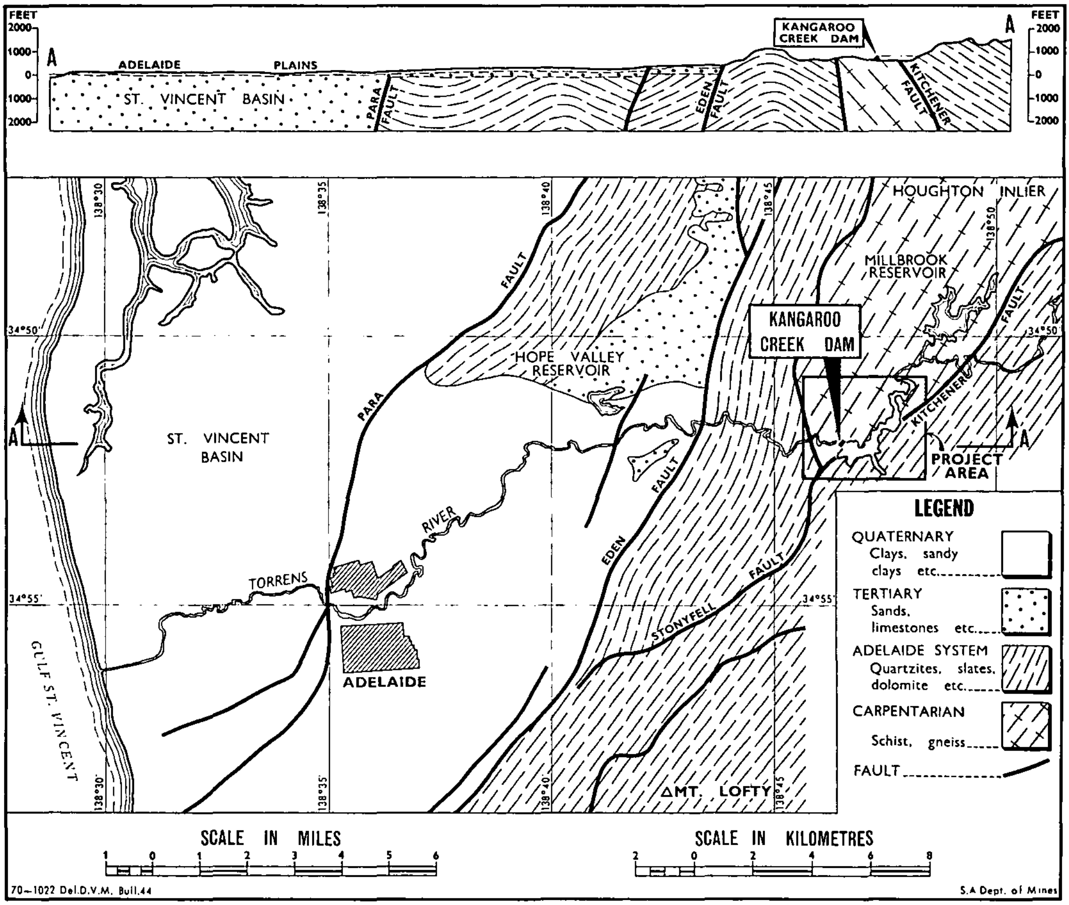

Fig. 1. Regional geology; simplified plan and section.

The Kangaroo Creek Dam project included a 200 feet high rockfill dam and an 840 feet long spillway, constructed by private contractors on behalf of the Engineering and Water Supply Department.

The author has been associated with geological investigations for the project since 1964. Throughout the main construction period (February,

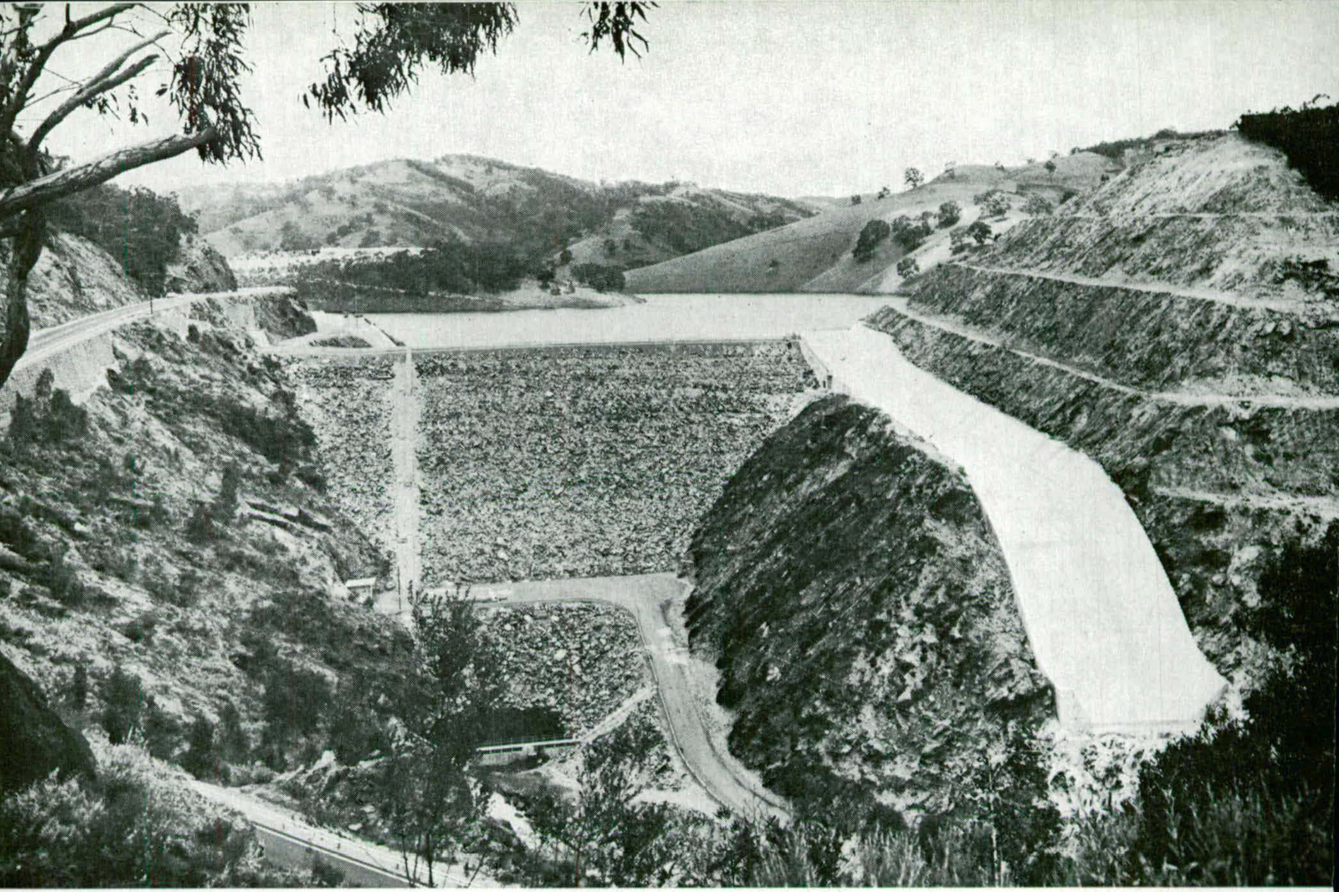

Fig. 2. View of rockfill dam and spillway near completion. Reservoir in background is partly filled.

1968 to October, 1969) the author was stationed at the site as resident geologist in the construction team. During this period the main objectives of the resident geologist were:—

1. To assist and advise the resident engineer on geological factors affecting construction.
2. To keep a complete record of the geological conditions revealed during construction.
3. To compare the actual geological conditions with those predicted at the design stage.

This report is divided into four sections:—

1. Background. The relevant geological factors are described and the history of the project is summarized.
2. Site geology. The detailed geology of the site—the physical properties, weathering effects and structural features of soil and rock substances are described.
3. Construction. The geological conditions revealed during construction are recorded, mainly in the form of detailed geological drawings.

4. Evaluation. The main geological problems encountered during construction are described and discussed. The effects of geological factors on the efficiency and value of various construction techniques are examined, and some suggestions are made to improve the techniques. The role of the engineering geologist as part of the construction team is also discussed.

ACKNOWLEDGEMENTS

The geological work for this project was carried out under the direction of Mr. D. H. Stapledon (Supervising Geologist of the Engineering Geology Section, South Australian Department of Mines) who himself carried out most of the pre-design stage investigations. Other geologists associated with the project included J. A. C. Painter and W. R. P. Boucaut. The author wishes to record his gratitude to these and other officers of the Department of Mines and to those officers of the Engineering and Water Supply Department who were associated with the project.

Part 1

BACKGROUND

HISTORY OF PROJECT

Site selection studies

Preliminary investigations for a dam on the River Torrens in the vicinity of Kangaroo Creek, were carried out as early as 1949 (Miles, 1950). During the period 1959 to 1960, geological investigations were made at two sites; one (Site No. 1) near the confluence of Kangaroo Creek and the River Torrens, and the other 900 yards downstream, near Batchelor's Bridge (Site No. 2). These investigations were made to determine the most suitable site for the construtcion of a 140 feet high concrete dam.

(1) *Dam Site No.* 1. The investigations consisted of geological mapping and the drilling of 13 diamond drill holes. This work showed that the site comprised a U-shaped valley, the floor of which was 200 feet wide and underlain by up to 20 feet of alluvium. Rock exposed in the abutments

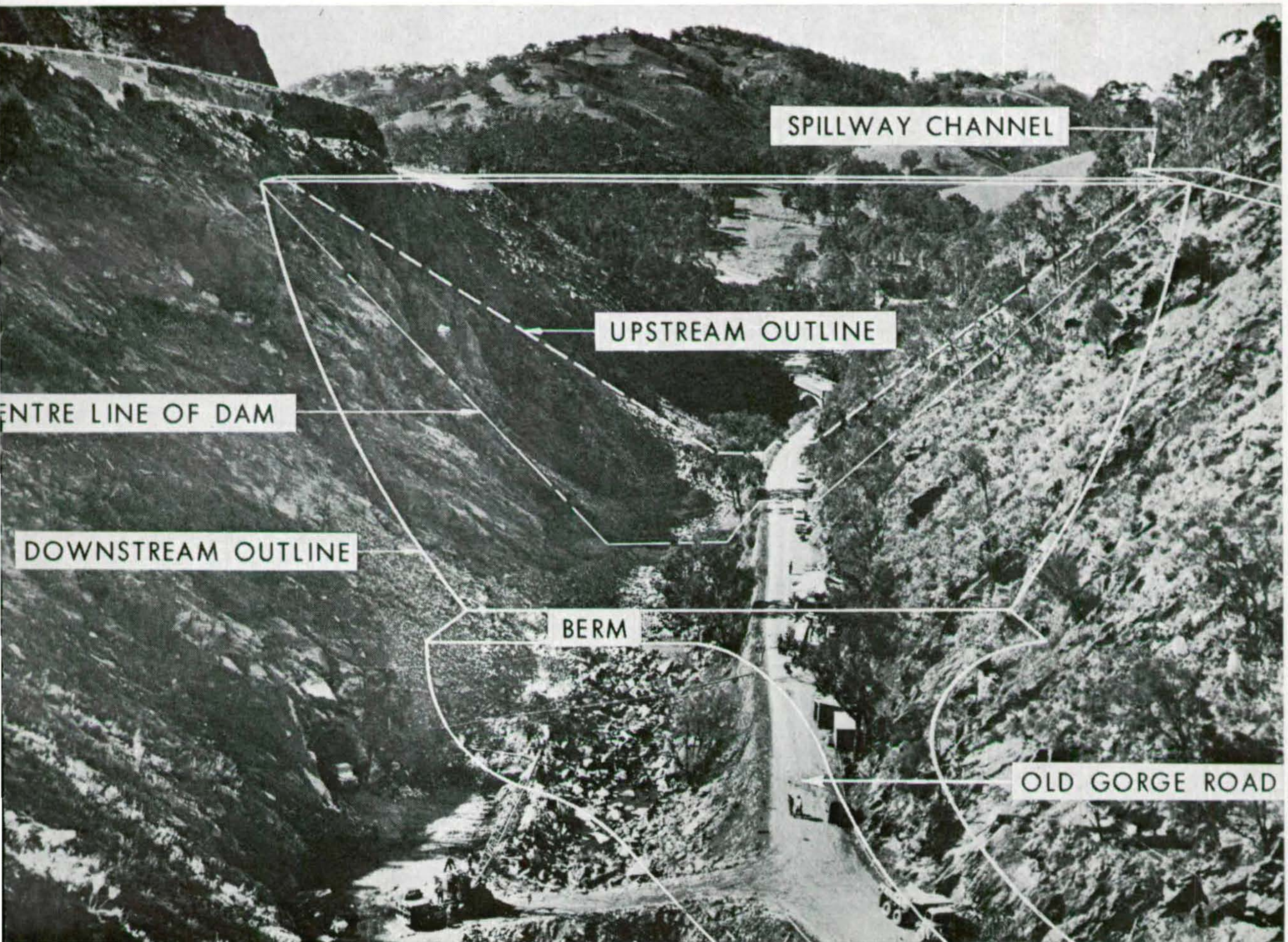

Fig. 3. Site before construction; outline of proposed dam is shown.

were dolomitic sandstones and slates, with numerous solution cavities along joints in the calcareous rocks. The results of these investigations are described by Miles (1950; 1951) and Hillwood (1959).

(2) *Dam Site No.* 2. A reconnaissance geological examination of this site indicated that the rock consisted of non-calcareous, metamorphic rocks (schists and gneisses) of Precambrian age.

Due mainly to the well known difficulties associated with dam construction on calcareous foundations, Site No. 1 was rejected in favour of Site No. 2, which also had distinct topographic advantages with its narrow valley and steeper walls, and a larger potential storage volume.

Feasibility stage investigations

The steep, narrow profile of the valley and the apparently high proportion of strong rock outcrop suggested that Dam Site No. 2 was ideal for a concrete arch dam. During 1960 an exploration programme consisting of sufficient geological mapping to enable "bedrock maps" to be prepared, and diamond drilling, was carried out at this site to determine its suitability for construction of a 140 feet high dam (Hillwood, 1960).

Nine cored holes were drilled, eight to test the foundation conditions, along the line of the proposed dam and one to check the possibility of leakage from the reservoir in a gully to the southeast of the dam site. The holes were water pressure tested at 10 feet intervals.

The diamond drilling was carried out using 'NX' core barrels, and one or more losses of up to two feet of core occurred in most of the holes.

As a result of these investigations it was decided to proceed with the design of a concrete arch dam to be constructed at Site No. 2.

Re-assessment and detailed investigation for a concrete arch dam

In 1963 it was decided to change the design of the dam to that of a 200 feet high, double curvature concrete arch. During the following year a systematic geological investigation was carried out, including detailed outcrop mapping on a scale of one inch to 20 feet and a new programme of diamond drilling using 'NMLC' coring equipment.

A detailed account of this investigation has been given by Stapledon (1966).

The investigation revealed the presence in the left abutment of numerous zones of weakness, some up to two feet wide and containing seams of soil materials. These zones were correlated with the core losses in the first diamond drilling programme. The nature of the seam materials and the results of surface mapping indicated that many of the seams were related to small slide movements.

To further assess the extent and nature of these foundation weaknesses, two exploratory adits were excavated in the left abutment. The adits confirmed the disturbed nature of the rock mass.

This exploration led to considerable doubt as to the ability of the foundations to withstand the wide variation of stresses which would be transmitted through a concrete arch structure. The area of investigation was then extended to determine whether better sites were available within 600 feet in a downstream direction. However, similar conditions were revealed for each of the new sites investigated.

It was therefore decided to consider other types of dams.

Design stage investigations for a concrete-faced rockfill dam

It was evident from the investigations carried out for the concrete arch proposals that the site was suitable for the construction of any one of several types of rockfill dam. The proximity of deposits of usable rock and the absence of nearby deposits of suitable core materials, resulted in the acceptance of a concrete-faced rockfill design.

The investigations for various arch sites had included most of the site chosen for the rockfill dam, considerably reducing the design stage investigation requirements. The following supplementary investigations were carried out:—

1. Extension of the detailed geological mapping on a scale one inch to 40 feet.
2. Excavation of two hand trenches in the grout cap area and three bulldozer trenches in the spillway area.
3. Drilling of five diamond holes totalling 780 feet in a proposed quarry area downstream of the site.
4. Testing of rock materials. Comprehensive reports of factual geological data were compiled for prospective contractors. (Painter and Trudinger, 1967; Trudinger, 1967a; Trudinger, 1967b.)

ADOPTED DESIGN OF PROJECT

The purpose of the project is to impound a storage of 6,000 million gallons of water and to release this water as required into the Adelaide metropolitan reticulation system.

The project may be divided into four main parts:—

Dam embankment.
Spillway.
Outlet and diversion works.
Quarries.

The layout of the main project features is shown in Fig. 4.

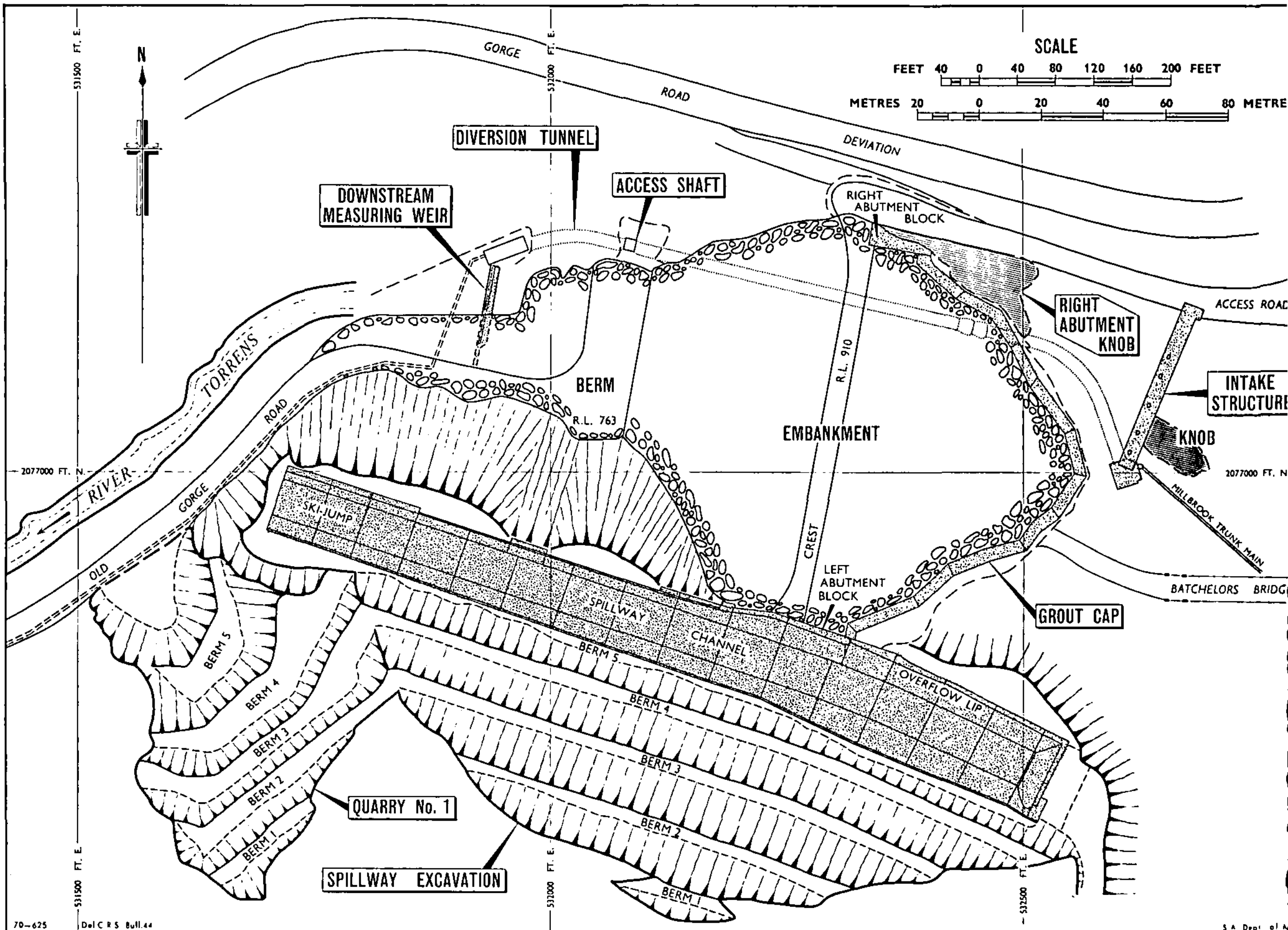

Fig. 4. Site plan.

Dam embankment

The 200 feet high dam consists of a zoned embankment of trapezoidal cross-section, composed of rockfill, placed and compacted in layers. A concrete membrane on the upstream face renders the dam watertight and is connected to the foundation rock around the upstream perimeter of the structure, by a concrete grout cap. A 50 feet wide berm extends one quarter of the way up the downstream face of the embankment.

Spillway

Provision for discharge of overflow water is by means of an 880 feet long concrete-lined, side-channel spillway with a 320 feet long overflow lip at the upstream end and terminating at the downstream end in a ski-jump.

The safe discharge capacity of the spillway is 24,000 cusecs which represents a one in one thousand year flood.

Outlet and diversion works

An 870 feet long diversion tunnel beneath the right abutment was used for diversion of the River Torrens during construction. The intake structure consists of a concrete tower inclined against the right bank immediately upstream of the diversion tunnel. Water from the reservoir is released by valves in the intake structure to flow along outlet pipes in the diversion tunnel, up the access shaft, through a screen chamber located on the downstream berm of the embankment into the Kangaroo Creek Trunk Main and thence to the metropolitan distribution system. A separate piped outlet which leads through the tunnel to two hollow jet valves discharging into the River Torrens, provides water for riparian rights.

Quarries

The rockfill for the embankment was obtained from six main quarries, five of which were excavated specifically for the project (Fig. 5) and the other, an existing commercial quarry.

1. *Spillway excavation.* The bulk of the material for the embankment was obtained from the excavation for the spillway. This excavation consists of a series of five 50 feet high benches separated by 15 feet wide berms, and a channel up to 40 feet deep and 40 feet wide.

2. *Quarry No.* 1. This quarry is located immediately downstream of the spillway. Material from it was used in the lower parts of the embankment and in the downstream berm. The excavation consists of six benches, 50 and 75 feet in height, with berms 15 to 40 feet wide.

3. *Quarry No.* 2. This quarry is located 1,000 feet southeast of the upstream end of the spillway. Material from it was used in the upper part of the embankment. The excavation consists of a single, three-sided bench up to 70 feet high.

4. *Zone* 1 *Quarry.* This quarry is located one and a quarter miles northeast of the dam. Material from it was used in the outer zone of the embankment, beneath the upstream face. The excavation consists of a single three-sided bench up to 50 feet high.

5. *Road spoil—contract variation No.* 7. Loose spoil materials from the lower right bank of the River Torrens, a half a mile downstream of the site, were used in the lower part of the embankment.

6. *Riverview Quarry.* This privately owned quarry situated three miles downstream of the dam, provided some of the material used in the upstream toe of the embankment. The material was obtained from several parts of the quarry complex.

CONSULTANTS

During the latter part of the investigation for a concrete arch dam, expert opinion was sought from geological consultants D. G. Moye (then Chief Geologist, Snowy Mountains Hydro-electric Authority) and Professor E. A. Rudd (then Professor of Economic Geology, University of Adelaide). In his capacity as consultant to the Engineering Geology Section, South Australian Department of Mines, Professor Rudd retained an interest in the project throughout all stages.

In designing the concrete faced rockfill dam, the Engineering and Water Supply Department obtained advice from American consultant J. Barry Cooke. Mr. Cooke made regular six-monthly visits to the site during construction, and was consulted on many of the problems which occurred during this stage.

REGIONAL GEOLOGY

Physiography

The dam and reservoir are located on the western side of the Mount Lofty Ranges (Fig. 1) which form part of a major mountain chain extending from the southern coast to the northern border of South Australia. In the vicinity of Adelaide the ranges are bounded to the west by coastal plains of the St. Vincent Basin, and to the east by plains of the Murray Basin.

The ranges consist of a series of ridges and dissected plateaus trending north to northeast and tilted to the southeast. Several rivers are deeply entrenched in the western part of the ranges and follow a westerly direction across the trend of the ranges, before discharging onto the plains.

The River Torrens, on which the Kangaroo Creek Dam is situated, has its head-waters 15 miles east of the site. From the dam to the edge of the plains (a distance of three miles) the river flows in a narrow steep-sided gorge 600 to 800 feet deep, and the river grade averages 100 feet per mile. Across the coastal plain, a distance of 15 miles, the grade averages 15 feet per mile and the river meanders in an alluviated terraced valley up to 50 feet deep and 1,300 to 5,000 feet wide.

Stratigraphy and structure

The geology of the region has been mapped by Sprigg, Whittle and Campana (1951) on the 1 : 63,360 Adelaide map sheet, and has been described in Parkin (1969). The geological succession is summarized in Table 1.

TABLE 1

GEOLOGICAL SUCCESSION

Age	Description of materials	Environment of deposition	Location
Quaternary	Clays, gravels and sands Clays, silts, sands	Fluvial and lacustrine. Marine (shallow) and estuarine	St. Vincent Basin
Tertiary............	Lateritic sandstone .... Limestones, sandstones, marls, sands, etc. Sands, clays and lignites	Terrestrial........... Marine (shallow) .... Lacustrine	Mount Lofty Ranges St. Vincent Basin
Precambrian (Adelaide System)	Slates, quartzites, dolomites, phyllites and tillites	Marine (geosynclinal)	Mount Lofty Ranges (also underlying St. Vincent Basin)
Older Precambrian (Barossian)	Schists and gneisses	Unknown	Mount Lofty Ranges

The stratigraphy and structure are shown in simplified form in plan and section on Fig. 1. The older Precambrian rocks, in which the dam is located occur within the Houghton Inlier, which is 20 miles long and up to five miles wide, and extends northeast from Castambul. The sediments of the Adelaide System are folded into a series of broad folds trending in a northeasterly direction.

Most of the major faults, including those bounding the Houghton Inlier, were formed in Palaeozoic times. During and since the Tertiary period differential block movements occurred along some of the Palaeozoic faults, forming the Mount Lofty Ranges as a series of fault blocks tilted towards the southeast with the St. Vincent Gulf to the west.

The relationship of the present topography to these fault blocks, is discussed by Sprigg (1945). Continuing relative movements along these faults since Tertiary time have given rise in the St. Vincent Basin to several periods of transgression with marine deposition, and regression with erosion and terrestrial deposition.

Seismicity

The mountain chain which includes the Mount Lofty Ranges is one of the most seismically active areas on the Australian continent, the major part of which is considered to be seismically quiet (Doyle, Everingham and Sutton, 1968). Since systematic recording of shocks in South Australia commenced in 1962, three to four minor shocks per month have been recorded. A map showing locations of the main seismic events prepared by Sutton and White (1968) shows that most shocks have been concentrated along the western margin of the Mount Lofty and Flinders Ranges, and it is considered that they represent small movements along the faults which formed the ranges and adjacent gulf. Most of the shocks recorded have been of small magnitude (Local Magnitude (M_L) less than three). However several moderate to severe shocks have been recorded.

An earthquake of magnitude 6 on the Local Magnitude Scale, which is equivalent to Intensity 8 of Modified Mercalli Scale, occurred near Adelaide in 1954 and is believed to have been caused by small movements at shallow depth along the Eden Fault (Fig. 1) which passes about three miles west of the Kangaroo Creek Dam. This shock caused widespread minor damage in the Adelaide metropolitan area (Kerr Grant, 1956). Several major faults occur closer to the dam and one of these, the Kitchener Fault, passes within 600 feet of it (Fig. 5). This fault was formed in Palaeozoic times and does not appear to have been reactivated during Tertiary and Recent times. A prominent local lineament which crosses the Kitchener Fault near the dam site (Fig. 5) may be the surface expression of a younger fault, along which future movements could possibly occur.

GEOLOGY OF PROJECT AREA

The project area, shown in plan in Fig. 5, includes the dam site, spillway, quarry, reservoir and disposal areas.

Various aspects of the geology of this area have been described and discussed by Spry (1951), Webb (1953), Talbot (1963) and Williams (1967).

Topography

The area consists of a dissected plateau in which the Torrens River valley is incised up to 800 feet deep, and 3,000 to 6,000 feet wide.

Between Castambul and Batchelor's Bridge (Fig. 5) the River Torrens has formed a steep-sided, rocky gorge with slopes ranging from 27 degrees (1 on 2) to 90 degrees (vertical). The valley floor ranges from 40 to 80 feet wide and is mainly occupied by the river channel.

Upstream of Batchelor's Bridge (Frontispiece), valley slopes are mainly soil-covered and moderate to gentle (about 20 to 30 degrees) except on the undercut side of the river meanders where slopes are up to 55 degrees (1 on 0.7). Several alluvial terraces occur, 10 to 70 feet above river bed level. The river meanders over the valley floor which is up to 400 feet wide.

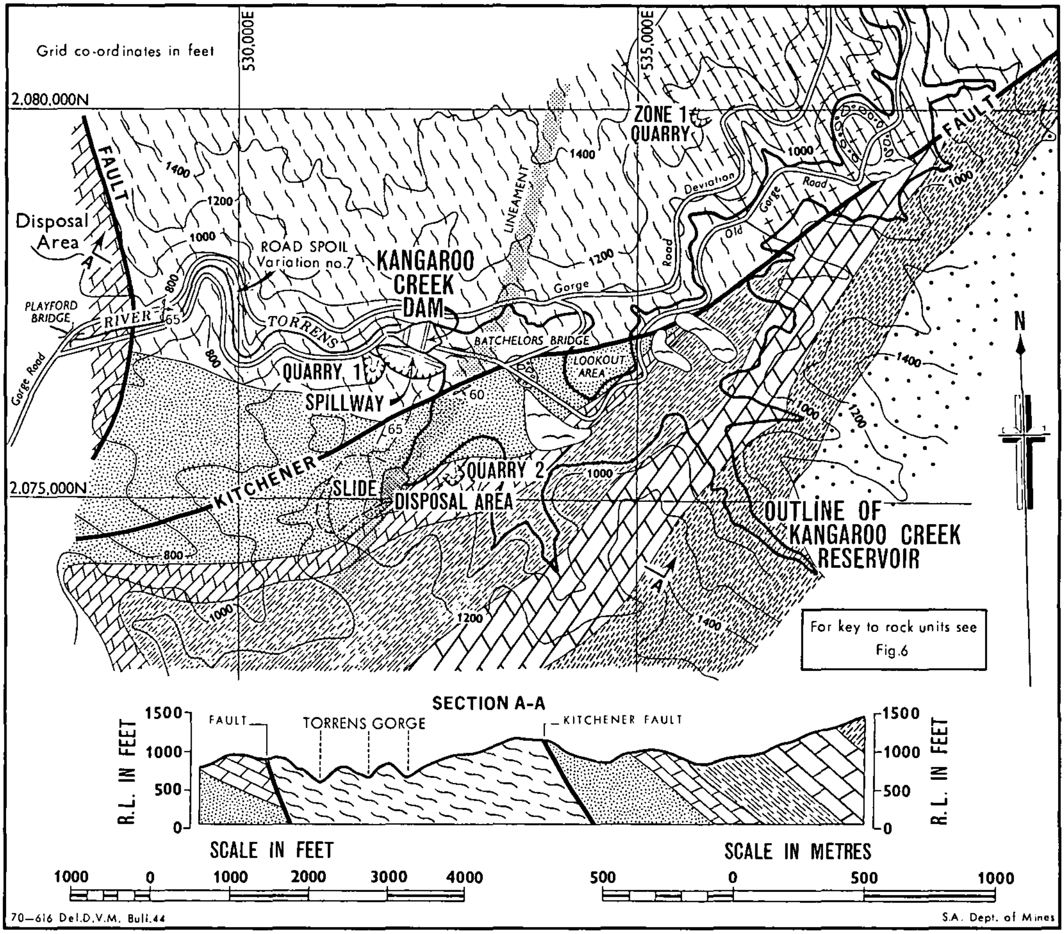

Fig. 5. Geology of project area; plan and section.

AGE	ROCK UNITS	ESTIMATED THICKNESS METRES	GRAPHIC SECTION	LOCATION IN PROJECT AREA	LITHOLOGY
QUATERNARY	Torrens River Alluvium	Up to 5		Reservoir Area	**Sand,** Silty. Gravel and boulders adjacent to river channel
DISCONFORMITY					
QUATERNARY	Torrens River Alluvium	Up to 10		Reservoir Area	**Silt** with bands of sand and gravel
UNCONFORMITY					
PRECAMBRIAN—ADELAIDE SYSTEM, TORRENSIAN SERIES	STONYFELL QUARTZITE	>300		Reservoir Area	**Quartzite,** medium grained, arkosic interbedded with **Siltstone,** sandy in part
PRECAMBRIAN—ADELAIDE SYSTEM, TORRENSIAN SERIES	LOWER PHYLLITES	350		Reservoir Area	**Siltstone,** sandy in part, calcareous in part, prominent cleavage
PRECAMBRIAN—ADELAIDE SYSTEM, TORRENSIAN SERIES	MONTACUTE DOLOMITE	300		Reservoir Area	**Siltstone,** sandy in part, calcareous in part, blue-grey with bands and lenses of **Dolomite,** Fine grained, argillaceous in part, and minor **Quartzite**
PRECAMBRIAN—ADELAIDE SYSTEM, TORRENSIAN SERIES	SILTSTONE AND QUARTZITE (unnamed)	200		Reservoir Area	**Siltstone,** bands up to 50 metres thick, interbedded with **Quartzite** bands up to 5 metres thick, medium grained, arkosic
PRECAMBRIAN—ADELAIDE SYSTEM, TORRENSIAN SERIES	CASTAMBUL DOLOMITE	120		Reservoir Area; Quarry No. 2	**Dolomite,** medium grained. Contains up to 20% quartz grains. White, cream and pink
PRECAMBRIAN—ADELAIDE SYSTEM, TORRENSIAN SERIES	ALDGATE SANDSTONE	>200		Disposal Area Reservoir Area	**Quartzitic sandstone,** feldspathic, micaceous in part, crossbedded in part, contains heavy mineral banding and **Siltstone,** sandy in part, prominent cleavage Conglomeratic at base. Calcareous towards top.
UNCONFORMITY – FAULTED IN PART					
PRE-TORRENSIAN	BAROSSA COMPLEX			Dam Site Spillway Quarry No. 1	**Schist,** quartz-sericite-feldspar-chlorite, mainly fine grained, prominent foliation flakey **Gneiss,** quartz-feldspar-sericite, medium to course grained, massive and minor masses of **Granitic gneiss**
PRE-TORRENSIAN	BAROSSA COMPLEX			Zone I Quarry Reservoir Area	**Gneiss,** quartz-feldspar, medium grained and **Gneiss,** quartz-feldspar-biotite, medium grained and minor **Schist,** mainly chlorite

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Fig. 6. Geological succession and descriptions of thickness, lithology and location of rock units in project area.

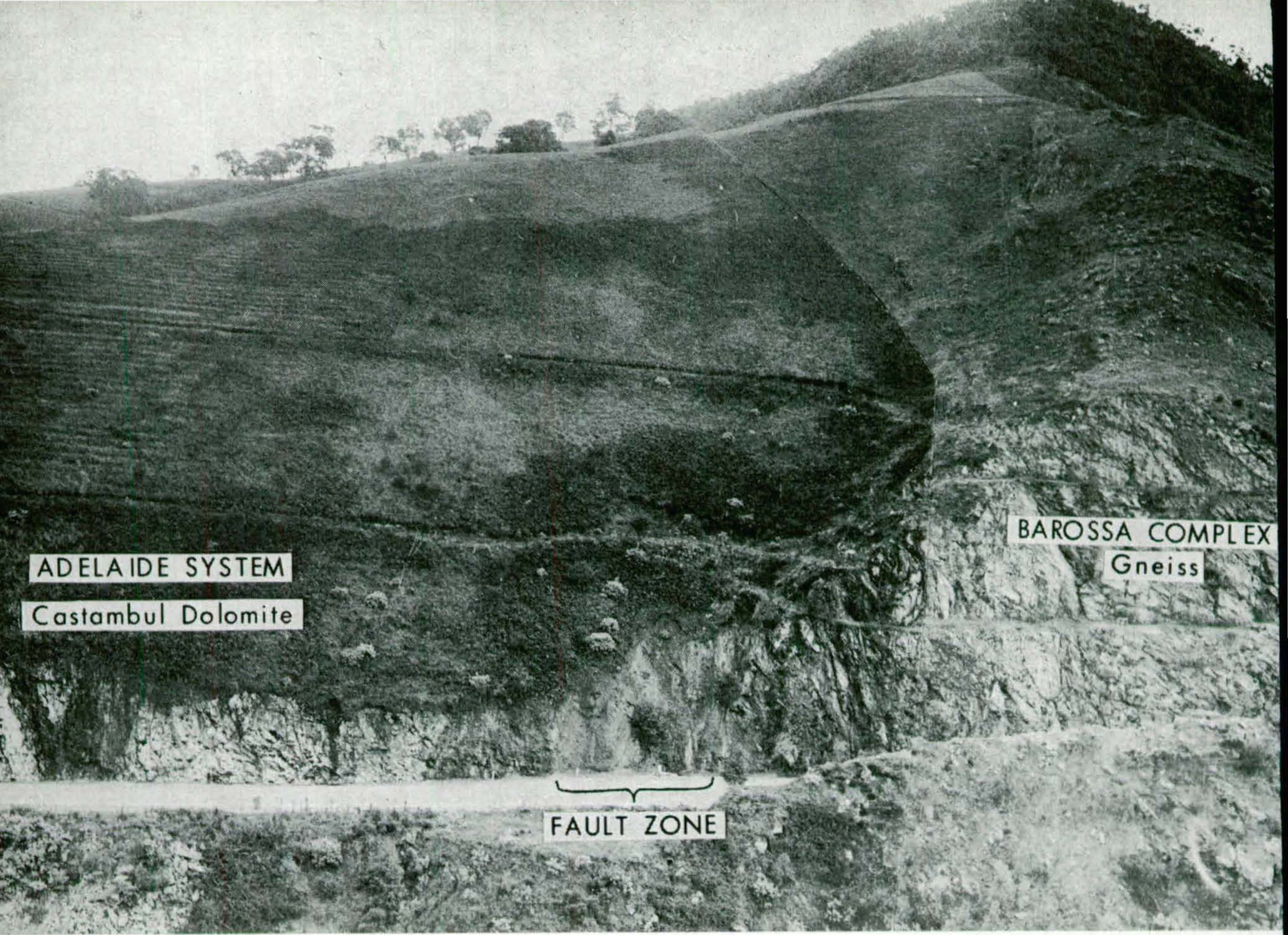


Fig. 7. Prominent fault separating Adelaide System rocks from Barossa Complex rocks; Gorge Road deviation excavation, 4,000 feet downstream from Kangaroo Creek Dam.

Stratigraphy and structure

The distribution of rock units and major geological structures is shown in plan and section on Fig. 5. The geological succession and descriptions of thickness, lithology and location of rock units, are shown in Fig. 6.

The project area may be divided into two main parts:—

1. *The area to the north and west of the Kitchener Fault* and bounded on the western side near Castambul by another prominent regional fault (Fig. 7). This area is an anticlinal core known as the Houghton Inlier. The inlier contains rocks of the Barossa Complex, except in the extreme south-west corner where Barossa Complex rocks are unconformably overlain by basal Adelaide System sediments.

2. *The area to the south of the Kitchener Fault,* occupied by Adelaide System rocks.

Barossa Complex rocks

These rocks apparently originated as sediments, however, metamorphism has destroyed almost all sedimentary features. Talbot (1963) concludes that there were at least three periods of metamorphism. Evidence of an initial high grade of metamorphism of the upper amphibolite facies has been found in relic minerals which occur in parts of the complex. However most of the minerals are consistent with a lower metamorphic grade of the lower greenschist facies produced by retrograde metamorphism.

Within the Barossa Complex in the project area two divisions have been respectively described by Spry (1951) as augen gneisses and schists, and granulites; and by Talbot (1963) as mica gneisses and quartz-feldspar gneisses.

1. *Augen gneisses and schists.* These rocks have been almost completely altered by retrogression so that no relic minerals remain. The dominant foliation shown as S1 on Fig. 10, is the one produced by the later periods of metamorphism. However relics of an earlier foliation shown as S2 on Fig. 10, occur in places.

2. *Granulites.* These rocks have been less severely affected by retrogression, and contain relic minerals such as garnet and andalusite. The original foliation shown as S2 on Fig. 10 is quite prominent and in many places dominates the later S1 foliation.

Spry (1951) has shown that the S2 foliation occurs in the form of tight isoclinal folds in the augen gneisses and broad folds in the granulites. The S1 foliation is fairly constant throughout the area, and dips slightly north of east at 40 to 70 degrees.

Adelaide System rocks

These rocks originated from sediments derived from Barossa Complex rocks which were deposited in the Adelaide Geosyncline. Although the rocks have been considerably affected by metamorphic processes such as recrystallization, many of the original sedimentary features such as bedding remain. The metamorphic grade is that of the lower greenschist facies, similar to that of the Barossa Complex rocks.

The argillaceous rocks contain a prominent cleavage or foliation which is near-parallel to the bedding, and near the site area is also near-parallel to the main S1 foliation in the Barossa Complex rocks.

River Torrens alluvium

These deposits which occur within the River Torrens valley may be divided into two types:—

1. Contemporary river channel and flood plain deposits, consisting of silty sands, gravels and boulders flanking the river channel, which occur up to 10 feet above the normal river bed level.

2. Older valley deposits, consisting of cobbles, gravel, sands and silts, which occur as remnants of terraces up to 70 feet above present river bed level.

Summary of main geological events

1. Deposition of Barossa Complex sediments.

2. Severe regional metamorphism, faulting and folding (Carpentarian Orogeny).

3. Retrograde regional metamorphism.

4. Prolonged erosion of Barossa Complex rocks and deposition of Adelaide System sediments in the Adelaide Geosyncline.

5. Regional metamorphism, folding and faulting (Palaeozoic Orogeny).

6. Prolonged erosion of Barossa Complex and Adelaide System rocks, resulting in peneplanation.

7. Laterization—development of lateritic crust on peneplain. Faulting—mainly by block fault movements along Palaeozoic faults (Tertiary Orogeny).

8. Erosion of Tertiary, Adelaide System and Barossa Complex rocks, development of existing drainage system, deposition of valley sediments and development of surface soils and weathering profiles.

Part 2
SITE GEOLOGY

DETAILED GEOLOGY OF SITE

Topography

In the vicinity of the dam the channel of the River Torrens follows an almost straight course in a westerly direction. The natural river bed gradient is approximately 1 on 35.

The valley in this area is markedly asymmetric. On the right bank the valley wall rises up to 750 feet above river level. The ground surface in the lower 250 feet consists mainly of rock outcrops with slopes mainly greater than 40 degrees and in places vertical to overhanging. The upper part of the right bank valley wall is bisected by a prominent gully which extends for almost one mile in a northerly direction. Slopes in this area range from 30 to 45 degrees with a general flattening towards the top.

The left bank valley wall rises up to 500 feet above river level and consists of rocky ridges up to 100 feet wide trending upslope at right angles to the river direction, and separated by soil-filled gullies, also up to 100 feet wide. Slopes are mainly 30 to 45 degrees, flattening to 20 degrees towards the top of the valley.

Soil substances

The skeletal soils in the site area are typical of the soils developed on steep slopes in a temperate climate with moderately high rainfall. Two horizons occur in most of the area and overly bedrock at relatively shallow depth (mainly less than four feet). The physical and mineralogical characteristics of these soils are given in Table 2. In the gully areas, transported soils occur in slide deposits up to 20 feet thick. These soils consist of rock fragments in a matrix of red-brown clay similar to the "B" horizon soil which overlies bedrock.

TABLE 2

CHARACTERISTICS OF NEAR-SURFACE SOIL SUBSTANCES

Type of substance	Unified Classification	Fines	Atterberg Limits of fines	Mineralogy of clay fraction
		per cent		
Topsoil	Sand, excess silty fines—(SM)	23 to 39	Liquid limit:—Approximately 20 Plastic limit:—Not applicable to some. Up to 15 in others Plasticity index:—Not applicable in some. Up to 6 in others	Illite
"B" horizon	Clay soil, high plasticity, gravelly and sandy in places.—(CH)	42 to 80	Liquid limit:—20 to 30 ..... Plastic limit:—15 to 18 ..... Plasticity index:—6 to 10....	Illite, trace of kaolin
Slide debris..	Gravel, excess clayey fines—(GC) to clay soil, high plasticity—(CH)	39 to 63	Liquid limit:—Approximately 25 Plastic limit:—15 .... Plasticity index:—9 to 13....	Illite, trace of kaolin

The occurrence of soil in narrow seams within the rock mass is described on pages 58 to 62.

Rock substances

Type and distribution

Numerous varieties of rock substance* could be disinguished in the site area on the basis of detailed petrological differences. However for simplicity they have been classsified into three main types—schist, gneiss and granitic gneiss. Table 3 summarizes the main petrological characteristics of the fresh rock substances. Detailed petrological descriptions of thin sections are given in Appendix 2.

TABLE 3

PETROLOGY OF FRESH ROCK SUBSTANCES IN SITE AREA

Rock substance	Mineralogy		Grain sizes	Texture	
	Mineral	Percentage range		Macroscopic	Microscopic
Schist	Quartz..........	10 to 30	As porphyroblasts up to 2 mm and in groundmass less than 0.3 mm	Material contains closely-spaced, slightly wavy foliation planes. Porphyroblasts occur isolated and in elongated patches	Material is lepidoblastic with elongated porphyroblasts around which foliae are bent. Alignment of porphyroblasts gives discontinuous banding in some places
	Muscovite (sericite)	40 to 60	Less than 0.2 mm (groundmass) .		
	Feldspar	Up to 20........	Mainly as porphyroblasts up to 2 mm.		
	Chlorite	5 to 20..........	As elongated flaky aggregates up to 20 mm long and 2 mm wide		
	Pyrite (and other sulphides)	Up to 5	Individual crystals up to 1 mm diameter		
	Hematite........	Up to 5			
	Zircon .... Tourmaline	Accessory			

Gneiss	Quartz	15 to 60	As elongated fractured porphyroblasts, up to 20 mm long. In groundmass, less than 0.5 mm	Alternating coarse-grained bands up to 2 mm wide consisting mainly of quartz and feldspar, and bands up to 25 mm wide of fine-grained foliated chlorite and sericite	Coarse bands have granular texture with crystals elongated along the foliation. Fine-grained bands are lepidoblastic with some porphyroblasts. Foliae bend around porphyroblasts and projection in coarser bands
	Feldspar	15 to 30	Mainly as elongated porphyroblasts, up to 20 mm long		
	Sericite	20 to 30	In groundmass, less than 0.3 mm		
	Chlorite	10 to 30	In fibrous aggregates up to 1 mm long		
	Pyrite (and other sulphides)	Up to 5	Individual crystals up to 2 mm diameter		
	Hematite	Up to 5			
	Tourmaline, Zircon	Accessory			
Granitic gneiss	Quartz	Up to 40	Crystals up to 10 mm diameter	Pegmatitic, coarse-grained, granular with few narrow stringers of schistose material	Coarse-grained, granular, with a small amount of lepidoblastic groundmass
	Feldspar	Up to 50	Crystals up to 20 mm diameter		
	Chlorite	Up to 20	Fibrous aggregates—individual fibres less than 0.2 mm wide		
	Sericite	Less than 10	Less than 0.2		
	Tourmaline	Up to 2	Individual crystals up to 3 mm diameter		
	Pyrite, Hematite	Accessory			

* The terms "soil substance", "rock substance", "soil mass" and "rock mass" are used in the sense defined in Standards Association of Australia—Australian Standard Code of Recommended Practice for Site Investigations (in press, 1972).

The three rock types occur in alternating layers and lenses up to 150 feet wide, but usually 20 to 40 feet wide. The more prominent boundaries between rock types are generally parallel to the S1 foliation direction. However there is considerable variation in rock type which is considered to be due to layering in the S2 foliation direction. Schist and gneiss commonly grade into one another, however the granitic gneiss usually occurs as relatively discrete masses.

The distribution of rock substances underlying the embankment and the spillway excavation, shown in Figs. 24 and 53 respectively, are typical of their distribution throughout the site. The right bank is underlain mainly by siliceous gneiss and granitic gneiss, which are considerably stronger than the flaky sericitic schists and highly feldspathic gneisses which underly the left bank.

Chemical weathering

This is the decomposition of rock-forming minerals due to the chemical action of groundwater. The rock materials are virtually impermeable and ground moisture penetrates the rock along joints, faults and microfractured areas. The near-surface zones in which stress relief has caused numerous microfractures to open up, is particularly susceptible. Dixon (1969) has shown that as chemical weathering proceeds, the intensity of microfracturing increases, presumably due to the volume changes caused by chemical reactions at grain boundaries. This has the following physical effects on the rock substance:—

1. Decrease in unconfined compressive strength.
2. Marked decrease in modulus of elasticity.
3. Increase in porosity.
4. Decrease in coefficient of restitution.

Classification of weathering products

Various attempts have been made to devise a classification to define the relative degrees of weathering for a particular rock type. Moye (1955) devised a classification (Table 4) for weathering products of granitic rocks.

The main disadvantage with Moye's classification was that it was not readily applicable to the weaker rocks such as schists. This disadvantage has been overcome in the case of the schist at Kangaroo Creek by changes in the definition from a basis of actual strength to one of proportionate strength. The classification as devised by Moye was found to be readily applicable to the weathering products of gneiss and granitic gneiss.

Recently, Dixon (1969) has devised a classification of decomposition products based on type of decomposition and the strength of the rock substance. However, as this classification does not indicate degree of decomposition, the author considers that it should be supplemented by terms such as those used by Moye. It is preferable however, that the terms are defined quantitatively, and it is suggested that wherever possible, classification should be based on the strength of the fresh rock substance.

TABLE 4

WEATHERING PRODUCTS OF ROCK—MOYE (1955)

Term	Abbreviation	Definition
Fresh..................	Fr	The rock shows no discolouration, loss of strength or any other effect due to weathering
Slightly weathered.......	SW	The rock is slightly discoloured but not noticeably weaker than the fresh rock
Moderately weathered ...	MW	The rock is discoloured and noticeably weakened, but 2in. diameter drill cores cannot be broken up by hand across the rock fabric
Highly weathered........	HW	The rock is discoloured and weakened to such an extent that 2in. diameter cores can be readily broken up by hand across the rock fabric
Completely weathered ...	CW	The rock is discoloured and entirely changed to soil, but the original rock fabric is mostly preserved. The properties of the soil depend upon the composition and structure of the parent rock

Tables 5 and 6 show the classifications of weathering products of schistose and gneissic rocks respectively, related to Dixon's classification and various physical properties which can be determined by simple field tests. The petrological features of the weathering products are also related.

TABLE 5

Classification of Weathering Products of Schistose Rock Substances

Modification of Moye's classification (1955)		Dixon's classification (1969)		Percentage of strength of fresh rock substance	Rebound number	Petrological characteristics
Term	Description	Terms	Unconfined compressive strength			
Fresh (Fr)	The rock shows no discolouration, loss of strength, or any other effect due to weathering.	Fresh	5,000 ↑	100	28 to 32	Minerals show no sign of decomposition, due to weathering. No limonite present. Any micro-fractures present are filled with sericite and are due to metamorphic cataclasis, not weathering
Slightly weathered (SW)	The rock is slightly discoloured, but but not noticeably weaker than the fresh rock. Chips of rock less than 0.5in. thickness can be broken up by hand, across the fabric	Medium strong	↓ 3,000	More than 75	23 to 28	Feldspars are partly weathered to sericite. Chlorite is partly weathered to limonite. Pyrite is slightly weathered at grain boundaries and hematite shows no traces of weathering. Few open or limonite-filled microfractures are present

Moderately weathered (MW)	The rock is discoloured and noticeably weakened, and 2in. diameter cores can be broken up by hand across the fabric	Chemically weathered	Weak	3,000 ↑ ↓	40 to 75	20 to 23	A major proportion of original feldspar crystals consist of sericite. Chlorite is mainly weathered to limonite. Pyrite and hematite are partly weathered to limonite and goethite
Highly weathered (HW)	The rock is discoloured and weakened to such an extent that chips less than 0.5in. thick are pulverized with a light hammer blow			1,000 / 1,000 ↑	15 to 40	Less than 12	Only traces of original feldspars remain—in the centre of crystal grains. Chlorite, pyrite and hematite are absent—weathered to limonite and goethite. Large crystals contain numerous irregular fractures. Numerous microfractures also occur in ground mass, some open, some filled with limonite.
Completely weathered (CW)	The rock is discoloured and entirely changed to soil, but the original rock fabric is mostly preserved. The properties of the soil depend upon the composition and structure of the parent rock		Very weak		Less than 15	Not measurable with Schmidt hammer	No feldspar present although outlines of original crystals remain. Limonite forms up to 30 per cent of material occurring in numerous microfractures penetrating porphyroblasts and groundmass

TABLE 6

Classification of Weathering Products of Gneissic* Rock Substances

Moye's classification (1955)		Dixon's classification (1969)		Percentage of strength of fresh rock substance	Rebound number	Petrological characteristics
Term	Description	Terms	Unconfined compressive strength			
Fresh (Fr)	The rock shows no discolouration, loss of strength, or any other effect due to weathering	Fresh Strong	12,000 ↑ ↓ 10,000	100	34 to 42	Minerals show no sign of decomposition due to weathering. No limonite present. Any microfractures present are filled with sericite and are due to metamorphic cataclasis not weathering.
Slightly weathered (SW)	The rock is slightly discoloured but not noticeably weaker than the fresh rock		10,000 ↑	More than 75	27 to 34	Feldspars are partly weathered to sericite. Chlorite is partly weathered to limonite. Pyrite is partly weathered to limonite and hematite shows slight weathering at grain boundaries. Few microfractures in fine-grained bands, tight, some limonite coated.

Moderately weathered (MW)	The rock is discoloured and noticeably weakened, but 2in. diameter drill cores cannot be broken up by hand across the rock fabric	Chemically weathered	Medium strong	↓ 3,000	40 to 75	20 to 27	A major proportion of original feldspar grains has been weathered to sericite. Only traces remain of original chlorite, which has been weathered to limonite. Pyrite and hematite are mainly weathered to limonite and hematite. Microfractures are present throughout, mainly closed.
Highly weathered (HW)	The rock is discoloured and weakened to such an extent that 2in. diameter cores can be readily broken up by hand across the rock fabric	Chemically weathered	Weak	3,000 ↑ ↓ 1,000	15 to 40	10 to 20	Feldspar is absent except for occasional remnants. Chlorite, pyrite and hematite are absent, weathered to limonite and goethite. Numerous micro-fractures throughout, some filled with limonite, some open to 1 mm.
Completely weathered (CW)	The rock is discoloured and entirely changed to soil, but the original rock fabric is mostly preserved. The properties of the soil depend upon the composition and structure of the parent rock		Very weak	1,000 ↑	Less than 15	Less than 10	Feldspar absent. Limonite occurring in numerous patches and micro-fractures constitutes up to 30 per cent of the material. Numerous micro-fractures, including many open to 1 mm. Quartz grains surrounded by limonite

* This classification was applied to materials described as gneiss and granitic gneiss.

Once all this information is related for a particular rock type, then in most cases only rock type and its degree of weathering are needed to define the properties of any rock substance. This system has been used successfully throughout the construction stage geological investigations for the Kangaroo Creek project, and would be equally applicable to any project in which rock substances range from fresh to completely weathered.

Chemical weathering profiles

The depth and degree of chemical weathering in the rock at the dam site are extremely variable, as they are often related more to the susceptibility of the rock type and to the pattern of joints and faults than to the depth below the ground surface. Small variations in relative proportions of those minerals which are comparatively susceptible to chemical weathering, such as pyrite, hematite and feldspars, are commonly reflected by marked variations in the degree of chemical weathering.

An idealized profile of the degree of weathering for a particular rock type is shown in Fig. 8.

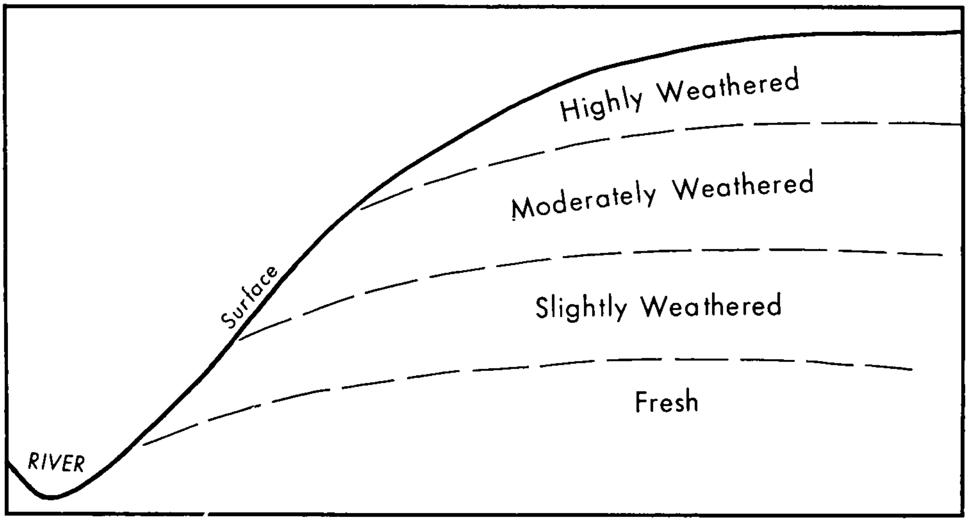


Fig. 8. Profile showing degree of weathering for a particular rock type.

Actual profiles such as those shown in Fig. 53 are far more complex, due to variations in rock type and the influence of joints and faults.

Physical properties

The results of unconfined compression tests on dry cores of unweathered samples of the three main rock types, are summarized in Table 7.

TABLE 7

UNCONFINED COMPRESSIVE STRENGTHS OF ROCK SUBSTANCES

Rock type	No. of samples	Unconfined compression strength (lb/sq./in.)		Modulus of Elasticity (lb/sq.in. x 10^6)	
		Average	Range	Average	Range
Schist	11	3,650	3,040	2.5	4.4
Gneiss...........	9	10,660	7,660	7.3	8.0
Granitic gneiss ...	4	11,300	8,800	5.9	0.5

A series of unconfined compression tests are also carried out on paired samples of schist. One of each pair was tested dry, the other after soaking in water for one or two weeks. The results are plotted in Fig. 9 and show a distinct relation between the unconfined compressive strength and the angle β (the angle between the foliation direction and the horizontal).

These results are consistent with the results of the comprehensive series of strength tests on similar foliated rocks, carried out by Donath (1961).

The results also show a marked decrease in strength due to saturation. Values of unconfined compressive strength for the saturated samples ranged from 35 to 70 per cent of those of the dry samples.

The results of other laboratory tests on fresh and partly weathered rock samples are given by Painter and Trudinger (1967) and Trudinger (1967c). These results are summarized in Table 8.

TABLE 8

LABORATORY TESTS ON ROCK SUBSTANCES

Tvpe of Substance		Physical Properties				
Rock type	Degree of weathering	Specific gravity (s.s.d.)†	Absorption (per cent)	Porosity (per cent)	Los Angeles abrasion (per cent)	Sodium sulphate soundness loss per cent (5 cycles)
Gneiss	Fresh	2.69 to 2.80	0.6 to 2.6	Not known	36	0.7 to 5.7
	Slightly weathered	2.64 to 2.68	2.1 to 2.9	Not known	Not known	13.5 to 21.9
	Moderately weathered	2.6	2.3 to 3.4	Not known	Not known	44
Schist	Fresh	2.68 to 2.80	0.7 to 1.2	4.8	43	0.2 to 2.7
	Slightly weathered	2.68 to 2.70	1.1 to 2.7	5.2 to 6.1	Not known	38
	Moderately weathered	2.65	1.8 to 3.0	8.0	Not known	62
Dolomite ..	Fresh	2.81	1.0	Not known	Not known	1.2
	Slightly weathered	2.68	2.7	Not known	Not known	2.6
	Moderately weathered	2.50	6.8	Not known	Not known	14.9

† Saturated and surface dry.

NOTE: Where a single value is given, only one of the particular test was made for the type of rock substance. Where more than one test was made, a range of values is given.

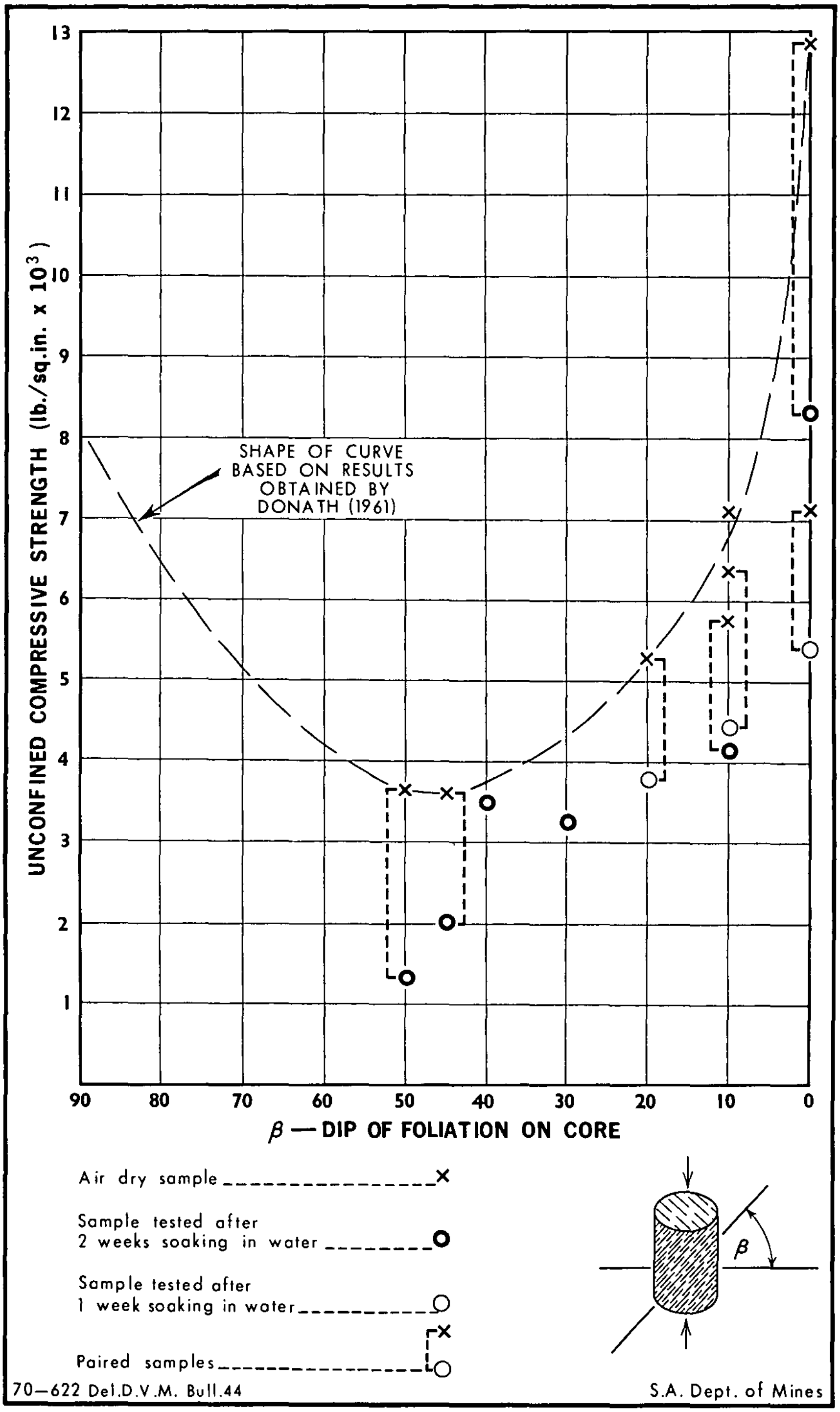


Fig. 9. Strength of schist materials; effects of anisotropy and saturation.

The results of a series of Schmidt rebound hammer tests carried out on the excavation surfaces at Kangaroo Creek Dam, are included in Appendix 4. The theory of the test is described by Dixon (1969). In Tables 5 and 6 the rebound numbers obtained from the Schmidt hammer tests are related to the degrees of weathering of the rock substance.

Durability

Three types of tests were carried out to determine the relative durability of different types of rock substance. The aim of these tests was to induce rapid weathering by extreme physico-chemical and temperature effects in the belief that the results could be related to the weatherability of the rock under more normal conditions.

The test results are given in Appendix 5. Two of the tests (sulphate soundness test and D.M.R. weathering test) are particularly harsh in relation to natural conditions. The results show that considerable breakdown of most samples occurred under test conditions. However, the temperature and weak salt solution tests caused almost no breakdown, even after 50 cycles, and it is considered that the effects of this test are more indicative of weathering effects under natural conditions.

The rapid deterioration of sulphide-bearing rock substances is discussed on pages 150 to 157.

Rock masses

Structural features

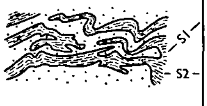
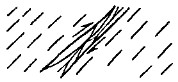
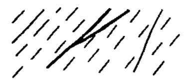
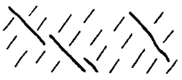
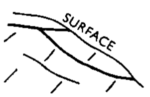
The rock mass is weakened to varying degrees by geological structural features of the types shown and described in Fig. 10. Those features which show a constant range of orientations within the site area are shown relative to the main engineering features of the project, in Fig. 14. In the following text the terms "spacing" and "extent" are used as defined by John (1962).

*Foliations**

Two separate foliations are present in the site rocks.

1. S1 *foliation* is present to some extent in all site rock materials. In most rocks it is the major textural feature of the rock fabric, to which it contributes marked anisotropy. The orientation is constant within a relatively small range throughout the site area.

* The term "foliation" is used to include laminar banding of minerals as well as alignment of tabular minerals, in the sense of Turner and Weiss (1963).

STRUCTURAL FEATURE	DESCRIPTION	NOTES ON ORIGIN
 FOLIATION – S1	In SCHIST – occurs as wavy laminar arrangement of fine grained sericite and chlorite. In GNEISS – occurs as layering of parallel bands up to 1cm. diameter of coarse grained quartz and feldspar with alternating bands of foliated, fine grained sericite, quartz and chlorite.	Alignment of tabular minerals accompanied by metamorphic differentiation – effects of Post-Adelaide system low grade regional metamorphism.
 FOLIATION – S2	Layering formed of alternating parallel bands 1mm. to 10cm. wide with different mineral assemblages e.g. quartz and feldspar with sericite, quartz and biotite (commonly distorted or completely obscured by S1 foliation).	Metamorphic differentiation during high grade regional metamorphism – Pre-Adelaide system.
 SHEARED ZONE	Zone of near-parallel, closely-spaced joints which intersect one another and divide the rock into narrow platy blocks. Joint faces are commonly slickensided. Zone commonly contains one or more crushed seams. Total width from 10cm to 2m	Small displacements along a number of joints, due to shearing.
 CRUSHED SEAM	Near-planar seam of mechanically disintegrated rock, commonly composed of sand and gravel-sized rock fragments in a silty matrix. Total width from 1mm to 20cm	Displacement along single plane causing crushing of irregularities, due to shearing.
 JOINTS Sets 1 and 3	Near-planar partings across which there is zero tensile strength. Surfaces commonly slickensided. Set 1 joints parallel or near-parallel to S1 foliation. Limonite coated. (For detailed physical description see TABLE 9)	Rock failure in shear.
 JOINTS Sets 2 and 4	Near planar partings. Surfaces rough. Commonly *en echelon*. Limonite coated. (For detailed physical description see TABLE 9)	Rock failure in tension.
 MINERAL VEIN	Tabular and irregular masses of quartz, feldspar, and chlorite. Often contain vughs which may be partly clay-filled.	Hydrothermal intrusion and alteration.
 KINK BAND	Band up to 10cm wide, within which there is sharp angular deflection of the foliation (S1). Commonly contain joints along axial planes.	Migration of axial surfaces by internal shearing.
 SHEET JOINT	Curved irregular partings, near parallel to ground surface. Gaping or filled with transported soil. Surfaces commonly limonite coated. Generally tighter at greater depths.	Rock failure in tension due to re-distribution of overburden stresses.

70–619b Del.D.V.M. Bull.44 Adapted from Stapledon (1966) S.A. Dept. of Mines

Fig. 10. Structural features present in rock masses within the Barossa Complex.

2. S2 *foliation* is restricted in occurrence, having been obliterated in most places by the more recent S1 foliation. The variation of orientation from place to place suggests similar folding, although there is an overall dip towards the southeast at shallow angles (20 to 40 degrees). The S2 foliation has been distorted by the S1 foliation by varying degrees in different places, as shown in Fig. 11.

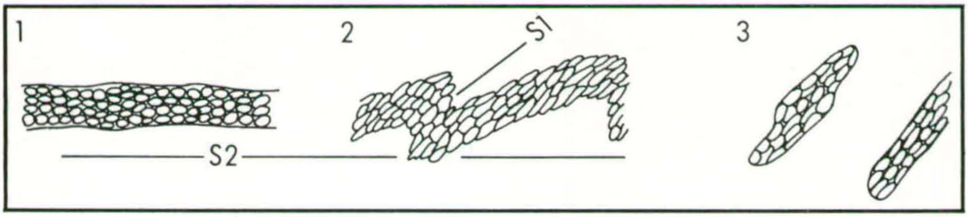


Fig. 11. Distortion of S2 foliation by S1 foliation.

Sheared and crushed zones

These features occur within or near to the orientation range of the S1 foliation. They may be divided into two main groups which probably represent conjugate shear directions.

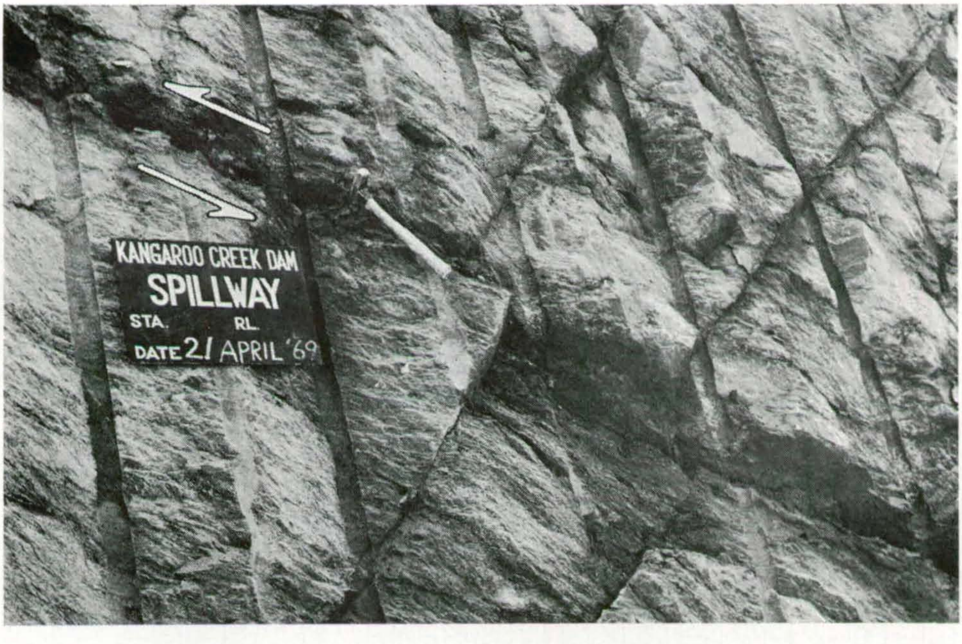


Fig. 12. Sheared, partly crushed zone exposed in excavation for spillway block LB; Set (2) joints indicate displacement of 10 cm across seam.

Sheared and crushed zones intersect the rock mass throughout the site, spaced less than 100 feet apart and extending for more than 300 feet. In some places (Fig. 12) displacements of joints along sheared zones, indicate that relatively recent overthrust movements have occurred along these structures. It is considered that these displacements were caused by Tertiary fault movements which re-activated pre-existing (Palaeozoic) weaknesses.

Two types of crushed seams and zones are present throughout the rock mass (Fig. 20).

1. Zones of rock disintegrated to silt size. These occur as seams less than one centimetre thick formed by crushing of the rock substance due to translational movement.

2. Zones of partly disintegrated rock. These occur as zones up to 30 centimetres wide*, containing rock fragments up to 20 centimetres in length which have been affected by localized translational and rotational movements.

In many places crushed zones and seams occur within sheared zones, in which they indicate the planes along which relatively high displacements have occurred.

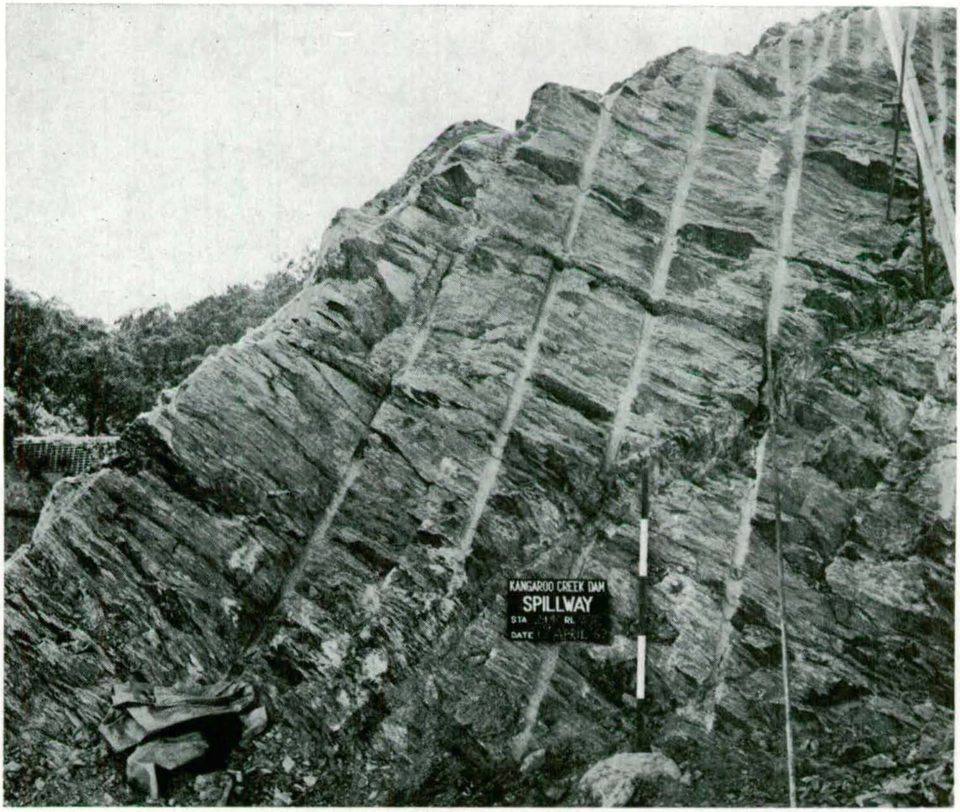


Fig. 13. Steeply dipping kink band exposed in initial excavation for spillway block LE; the zone also contains a seam of infilled clay 8 cm wide.

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* Throughout the project, the Metric system of measurement was used to describe the thicknesses of geological structures, and the British system was used for all other measurements. For convenience the same procedures have been followed in this text.

Kink bands

These features consist of monoclinal folds in the foliation, with axial planes usually less than 50 centimetres apart (Fig. 13). The zone between the axial planes, which is considerably disturbed and commonly contains numerous openings or mineral veins, may constitute a significant weakness within a rock mass.

Kink bands were intersected at intervals of 20 to 300 feet at the site. In the spillway channel foundations, three prominent kink bands dipping steeply towards the north, form the back of past slide masses. In other parts of the site there are kink bands with different orientations and with dips less than 20 degrees from the horizontal.

Tectonic joints

Tectonic joints occur in four main sets as shown in Figs. 14 and 15. The main characteristics of each set are shown in Table 9.

TABLE 9

JOINT CHARACTERISTICS

Joint set	Spacing *	Extent †	Description of surface texture			Field relationships
			Large scale	Small scale	Coatings	
1	15 cm to 5 metres (0.5 to 15ft.) mainly 1 metre	More than 30 metres (100ft.)	Slightly wavy. Amplitude 15-60 cm. Wave length more than 20 metres. Lay of waves—approximately 40° towards the east	Smooth to slightly scaly, or slickensided (grooved). Lay of slickensides —approximately 40° toward the east. Depth of irregularities mainly less than 1 mm	Limonite. Sometimes white silt (ML)	Commonly intersect and divide. Rarely fade out
2	15 cm to 10 metres (0.5 to 35ft.) mainly 3 metres	3 to 15 metres (10 to 50ft.)	Mainly planar. May be curved towards fringes. Some are slightly wavy with amplitude up to 15 cm and wave length more than 3 metres	Hackly to ridged. Depth of irregularities 0.5 to 3 mm. Lay of ridges—45° towards northeast (i.e. approximately 40° from direction of dip)	Limonite	Commonly occur *en echelon*. Fade out or terminate against other discontinuities
3	1 to 12 metres (3 to 40ft.) mainly approximately 8 metres	15 to 25 metres (50 to 80ft.)	Planar to slightly curved	Smooth. Occasionally slickensided. Lay of slickensides dips 20° to 40° east. Depth of irregularities mainly less than 1 mm	Limonite or chlorite	Some show plumose structures towards fringe
4	30 cm to 15 metres (1 to 50ft.) mainly 5 metres	3 to 15 metres (10 to 50ft.)	Mainly planar, may be slightly curved towards fringes. Some are slightly wavy with amplitude up to 15 cm. Wave length more than 3 metres	Hackly to ridged. Depth of irregularities 0.5 to 3 mm. Lay of ridges—25° towards the north (i.e. approximately 60° from direction of dip)	Limonite	Commonly occur *en echelon*. Fade out or terminate against other discontinuities

* Spacing is measured normal to the joint planes.

† The ranges given refer to all directions within the joint plane. No widespread tendency for elongation of joint planes in any particular direction, was evident from field observations.

Set (1) *joints* are within or near to the S1 foliation direction and, like the sheared and crushed zones the distribution of Set (1) joints shows a dumb-bell-shaped maxima, the lobes of which probably correspond to conjugate shear directions.

Set (3) *joints* are relatively infrequent. The limited number of measurements of these joints is partly due to the fact that most of the site excavations were elongated near-parallel to the Set (3) strike direction (Terzaghi, 1965).

The spacing of joints, particularly Set (1) joints, varies in different rock types. Generally the joint spacing is much broader in the gneissic rocks. In the granitic gneiss, joints tend to be discontinuous and irregular.

Sheet Joints

These joints, which are also known as exfoliation or Bankung joints, are believed to be formed as part of the mechanical weathering process. They occur in a zone within 30 feet of the surface of the rock, in areas where tectonic joints are either too broadly spaced or unsuitably orientated to relieve the tensile stresses developed in the valley walls (Stapledon, 1966). They are approximately parallel to the ground surface, and in many places several parallel joints occur in the near-surface zone, spaced two to 15 feet apart (Figs. 69 and 70). Usually the outermost sheet joints extend further and are more open.

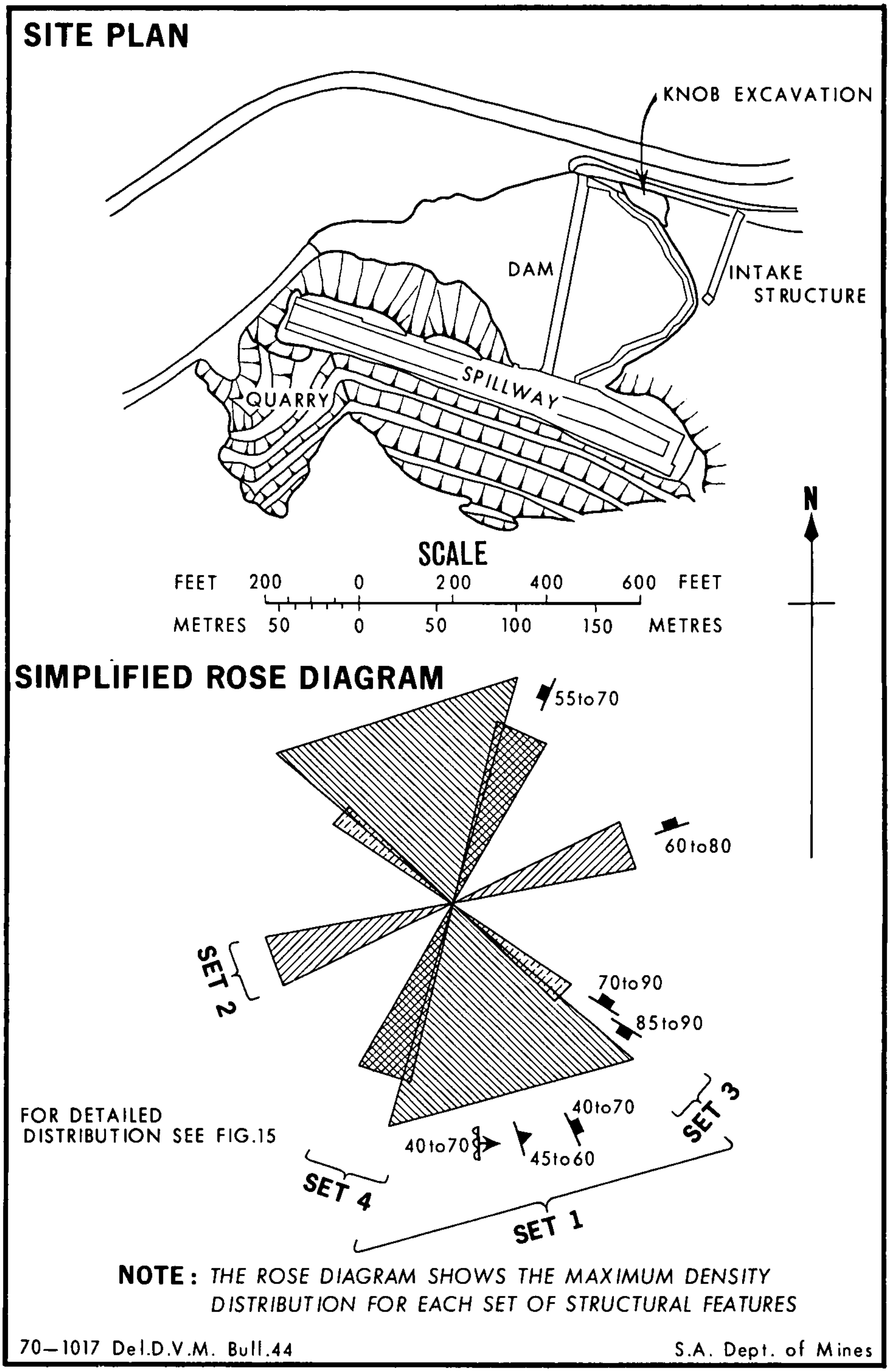

Fig. 14. Orientation diagram of structural features.

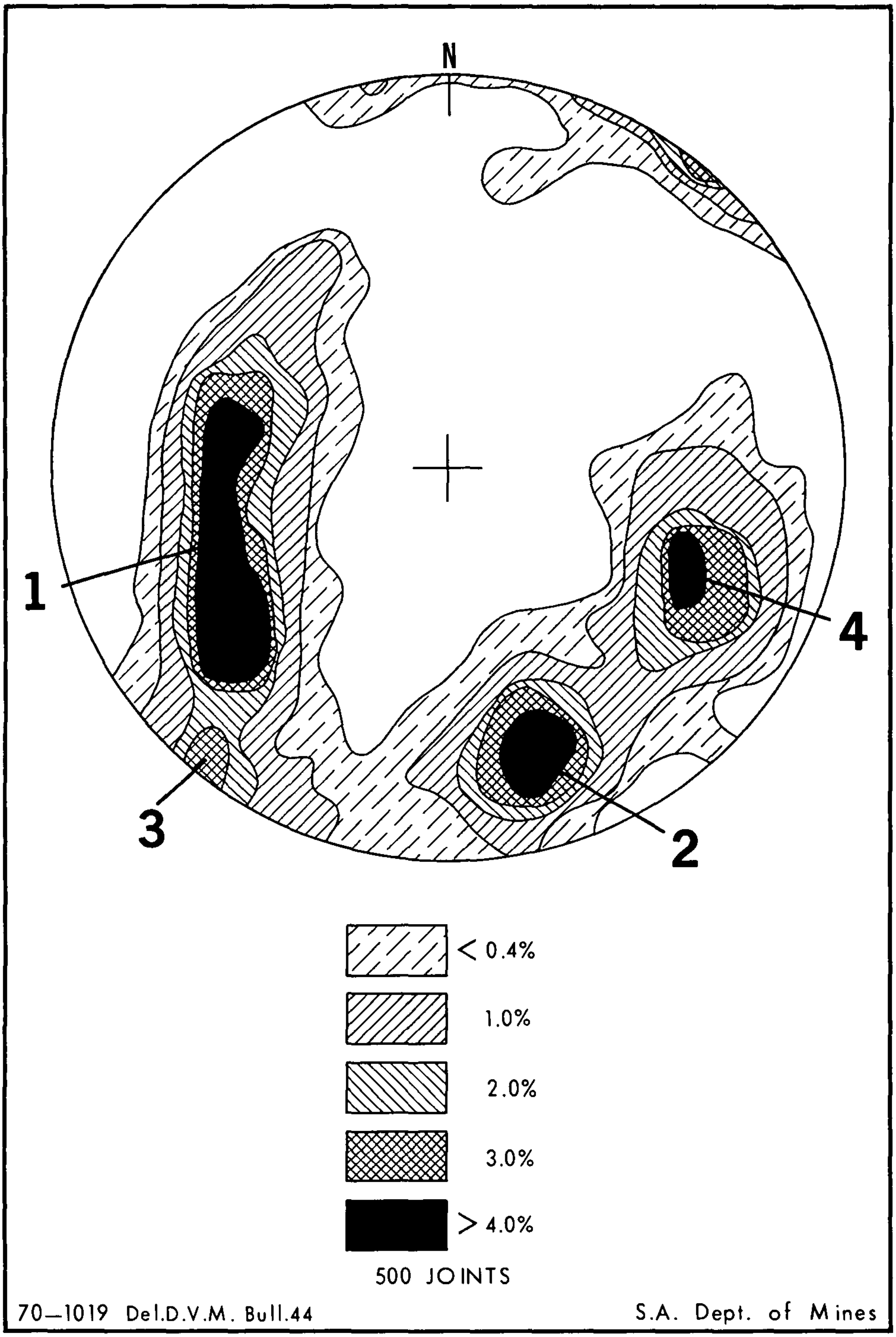

Fig. 15. Stereographic projection of joint poles.

Unit blocks

The concept of unit blocks is discussed by John (1962). A unit block is defined as a homogenous rock unit surrounded by a system of structural features intersecting a rock mass.

In the site area unit block sizes range from 10 by 30 by 30 centimetres up to 300 by 600 by 600 centimetres. The most common sizes of the unit blocks in different rock types are given in Table 10.

TABLE 10

SIZES OF UNIT BLOCKS

Rock type	Unit block sizes
Schist	10 x 30 x 30 cm to 50 x 200 x 300 cm
Gneiss	20 x 30 x 50 cm to 200 x 400 x 400 cm
Granitic gneiss	Greater than 30 x 50 x 50 cm. (Unit blocks usually incomplete due to discontinuous nature of joints).

Mechanical weathering

This is the macroscopic physical disintegration of the near-surface rock mass, and is believed to be initiated by the redistribution of stresses that follows the removal of overburden by erosion (Hast, 1967). Once fine cracks develop, the process is continued by the roots of vegetation which grow down along them, and in some cases by deposition of expansive clays. The rock mass may be further disturbed by downslope creep or slip movements. In closely jointed rock masses and where tectonic weaknesses occur near-parallel to the ground surface, mechanical weathering results mainly in small outward movements by relaxation of existing joint blocks. In more broadly-jointed rock masses, sheet joints are formed approximately parallel to the ground surface (see page 53).

The mechanical weathering profile may be classified into four zones:—

1. Transported material consisting of soil and loose rock fragments.
2. Zone of severe mechanical weathering effects. The rock mass contains numerous gaping or soil-filled joints exceeding one centimetre in width.
3. Zone of minor mechanical weathering effects. The rock mass contains few gaping or soil-filled joints and those that do occur are mainly less than one centimetre in width.
4. Zone unaffected by mechanical weathering.

Mechanical weathering profiles

Typical profiles of right and left banks are shown diagrammatically in Fig. 16.

On the right bank the main effect of mechanical weathering has been the development of numerous sheet joints. On the left bank tectonic (Set 1) joints near-parallel to the ground surface have been opened up, wedge-shaped masses of rock have been loosened by block-slide movements and in the upper parts flaky, chemically weathered rock has been highly disturbed by downslope creep. Some sheet joints occur in the lowermost 70 feet of the valley wall.

Transported material occurs only in isolated patches on the right bank whereas on the left bank it occurs in extensive areas up to 30 feet thick. The zone of severe mechanical weathering effects is about 20 feet thick in the right bank and between 30 and 70 feet on the left bank. Minor mechanical weathering effects extend to depths of 50 feet in the right bank and to more than 100 feet on the left bank.

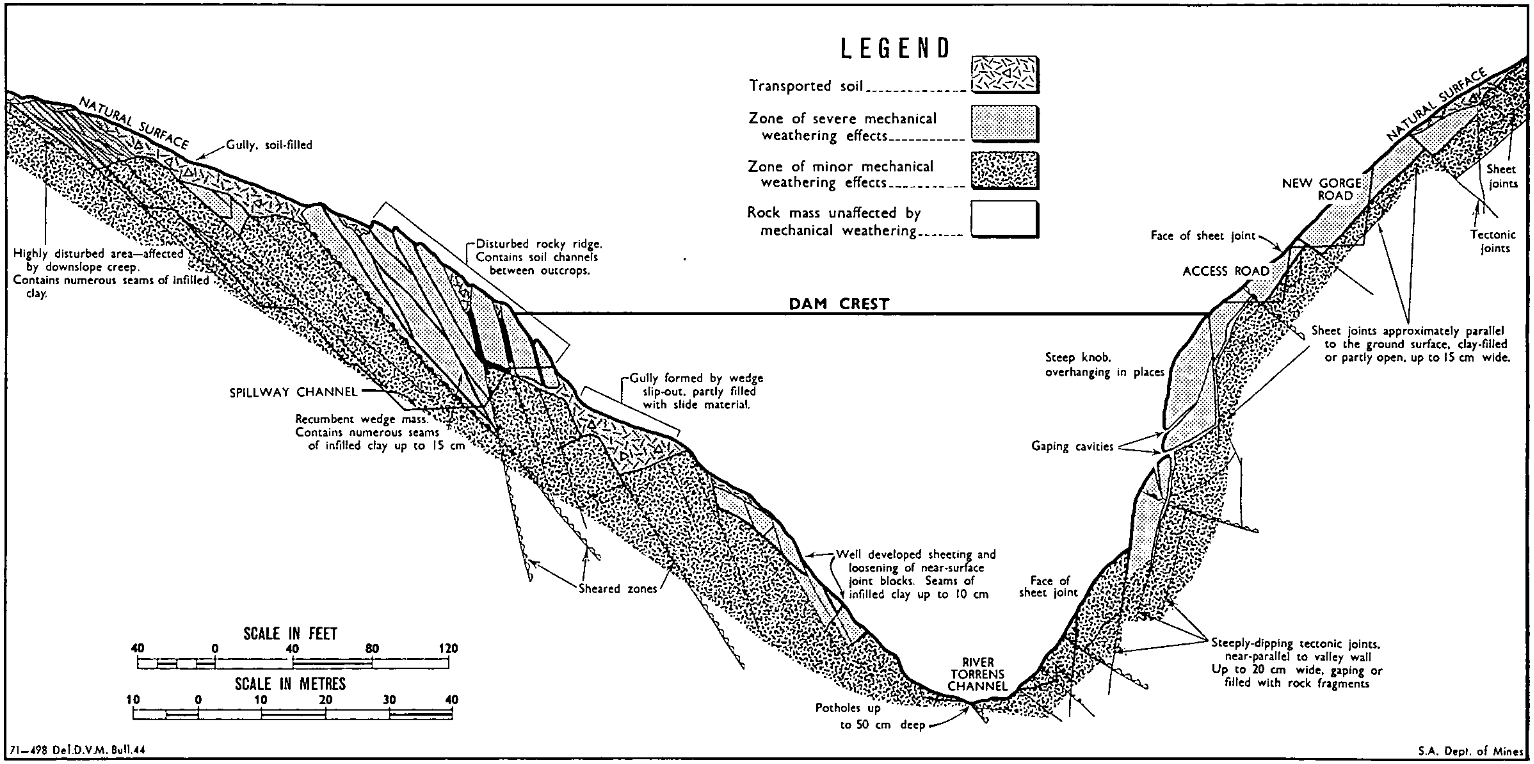

Fig. 16. Diagrammatic mechanical weathering profiles of right and left abutments.

Beneath the river channel the zone of minor mechanical weathering effects is about 20 feet deep. The surface contains numerous waterworn potholes up to one and a half feet deep and up to three feet in diameter.

The role of mechanical weathering in the development of site topography

Right bank. The natural surface of the lower part of the valley wall consists mainly of the faces of sheet joints, exposed due to the removal of overlying rock by sliding. In several areas steep bluffs of rock occur, in places where the slide masses have terminated along steeply-dipping joints (see Figs. 67, 69 and 70).

In the upper part of the valley superficial transported soils up to five feet thick occur over much of the surface. In many places these soils occur directly on the faces of sheet joints.

Left bank. The surface of the valley wall consists of alternating rocky ridges and soil-filled depressions from 30 to 120 feet in width, and trending upslope.

The rocky ridges consisting of disturbed and partly detached joint blocks, form part of large block slides which extend to considerable depths. The nature of these slides is discussed on pages 141 to 150.

Many of the depressions occur where V-shaped gullies formed by the sliding of wedge masses of rock have been partly filled with transported soil materials from above. In other more recently formed gullies the slide planes are still exposed. Figs. 17, 18 and 19 show different stages of the failure of wedge masses on the left bank.

Soil seams

The rock mass in the site area is intersected by a wide variety of seams, some of which are illustrated in Fig. 20. These seams constitute the most significant weaknesses in the rock mass and hence have been the subject of considerable study. Table 11 describes the main types of seams, their composition, properties and origin. Field classification of seams using the criteria given in Table 11, has proved extremely valuable in assessing the stability of excavations, quality of foundations, results of grouting *etc.*

Fig. 17. Seam of infilled clay in a Set (2) joint which forms one side of a semi-detached wedge of rock.

Fig. 18. Soil mass of rock fragments and clay, filling a V-shaped gully formed by sliding of wedge mass bounded by Set (1) and Set (2) joints; central part of spillway excavation.

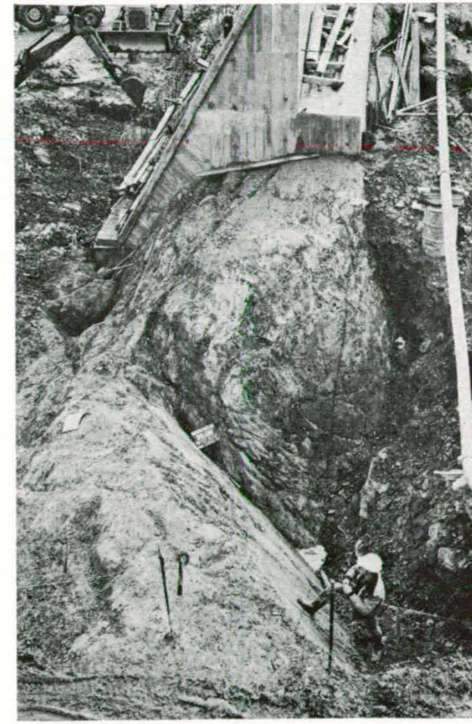

Fig. 19. V-shaped gully formed by sliding along Set (1) and Set (2) seams; spillway channel, north wall—foundation for block LH.

TABLE 11

Classification of Soil Seams in Rock Masses

	Crushed seams		Decomposed seams		Infilled seams		Composite seams Numerous types (see Fig. 20)
	Breccia	Rock flour	Weathered	Altered	Red-brown	Grey-green	
Grading......	Gravel in a matrix of silty sand (GM)	Mainly silt, some sand (MH, ML, SM)	Mainly fine sand and silt (ML, SM)	Usually medium to coarse sand (SP, SM)	Clay soil, high plasticity, with up to 60 per cent of sand and gravel (CH)	Clay soil, high plasticity, with up to 40 per cent sand and gravel (CH, SC)	Mixtures of clay and silt
Mineralogy ..	Gravel consists of schist or gneiss fragments. Silty sand is sericite, quartz and illite	Sericite, quartz, illite, smectite	Mainly sericite and illite (approx. 50 per cent) with quartz and limonite		Sand and gravel are quartz and rock fragments. Clay is smectite, illite and kaolin	Sand and gravel are quartz and rock fragments. Smectite, illite, kaolin	Mixtures of illite, sericite, quartz, smectite and rock fragments
Physical properties	Low dry strength	Low dry strength	Low density. Medium dry strength. Plasticity index approximately 5		High dry strength. Plasticity index 30 to 70	High dry strength. Plasticity index 20 to 40	Combination of properties for other types
Orientation ..	Near set (1) direction		Mainly along set (1) direction	Irregular	Along all joint directions........		Mainly along set (1) direction
Extent	Commonly more than 50 metres	Commonly more than 50 metres	Up to 20 metres	Up to 20 metres	Up to 100 metres	Less than 20 metres	Up to more than 30 metres
Range of thickness	Up to 50 cm ..	Up to 1 cm....	Up to 100 cm..	Up to 100 cm..	Up to 10 cm ..	Up to 40 cm ..	Up to 100 cm
Mode of formation	Partial crushing of asperities on plane of tectonic shearing	Complete crushing of asperities on plane of tectonic shearing	Weathering of highly susceptible minerals	Alteration by hydrothermal solutions, *etc.*	Precipitation in open joints of clay from groundwater in oxidizing environment	Precipitation in open joints of clay from groundwater in reducing environment	Combination of 2 or more other types

Seams of infilled clay are widespread throughout the site, but particularly on the left bank (Figs. 12, 13 and 17) where they are up to 40 centimetres in width. The appearance and composition of the clay suggest that it is derived from the clayey "B" horizon soils (Table 2). The following mechanism is postulated to explain the formation of infilled seams:—

1. Openings between adjacent unit blocks in the rock mass are formed by mechanical weathering processes.

2. Clayey material from surface soils is washed into and deposited in the openings which after numerous cycles of infilling and drying, are eventually filled with clay. A similar mechanism is postulated by Casagrande (1951) to explain the occurrence of clay infilling voids in openwork gravels.

3. Subsequent saturation may cause the clay to expand, thus enlarging the separation between unit blocks and allowing further infilling to occur after drying out.

Some of the seams are laminated, which confirms this progressive mechanism of formation. Other seams contain randomly orientated rock fragments which occupy almost the entire width of the seam, indicating that the separation of joint blocks in these cases, was not progressive.

Permeability

The permeability of the rock substances in the site area is negligible. Within the rock mass it is not strictly correct to consider a mass permeability, as the flow of water is restricted to localized paths along selected structural features. This is due to the fact that open-jointed areas are rare even within the zones of severe mechanical weathering, as these areas have been subjected to widespread infilling with clayey soil materials. For this reason the flow of water through the rock mass is slow throughout most of the site.

Only three areas of widespread open-jointing were encountered during investigation and construction. These were the spillway ski-jump north wall, left abutment block and right abutment block foundations—see Figs. 40, 49 and 36 respectively. In other parts of the site the structural defects are either tightly closed or mainly filled by soil materials. Permeability tests in these areas showed leakages mainly less than three lugeons with occasional higher leakages, up to 40 lugeons.

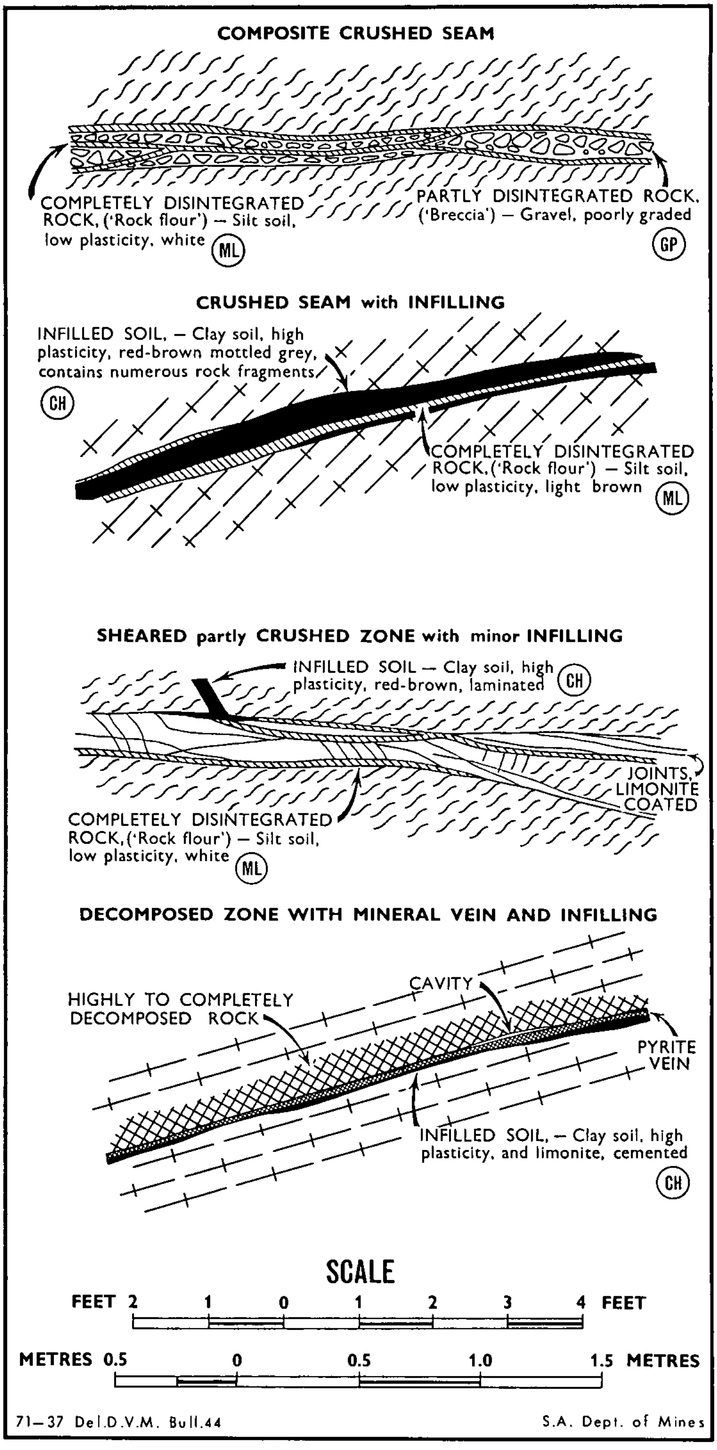

Fig. 20. Varieties of seams intersecting rock mass in site area.

Groundwater

With the steep valley slopes in the site area, much of the water from heavy falls of rain reaches the river by surface run-off. In gully areas percolation is restricted by clayey subsoil materials and the water soaks downward on the upper surface of the clay to emerge as seepages near the base of the valley.

Water entering the rock mass follows localized narrow paths along selected structural features. The percolation rate is slow and the amount of water absorbed into the rock is relatively small.

After periods of dry weather the free water table is near the level of the River Torrens (Fig. 21), rising slightly away from the valley. After moderately wet periods there is a zone of saturation near the surface. It is only after a period of prolonged wet weather that this zone of saturation extends to the water table. At such times the water table is near-parallel and close to the ground surface (Fig. 21).

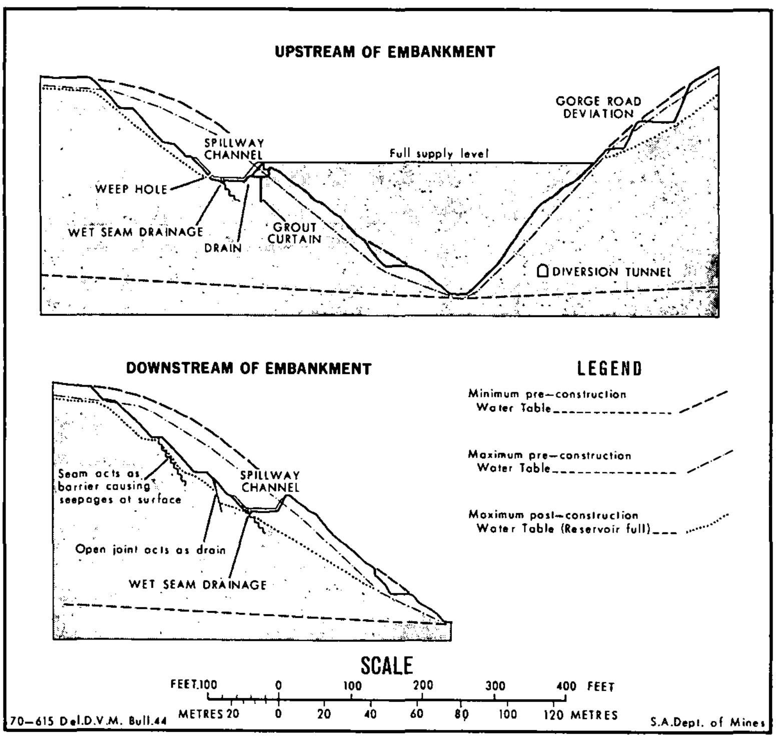

Fig. 21. Groundwater situation (idealized) at site area.

Part 3
CONSTRUCTION

FOUNDATIONS

Embankment

Requirements of foundations

The main consideration applied to rockfill embankment foundations is that they should be at least as strong as the embankment material itself.

The criteria for acceptance of the foundations were that they be:—

1. Mainly rock—so that rock to rock contact develops between rockfill and foundation.
2. Intact or almost intact—so that the foundation does not settle significantly under the loads applied by the embankment.
3. Durable.
4. Non-erodible.

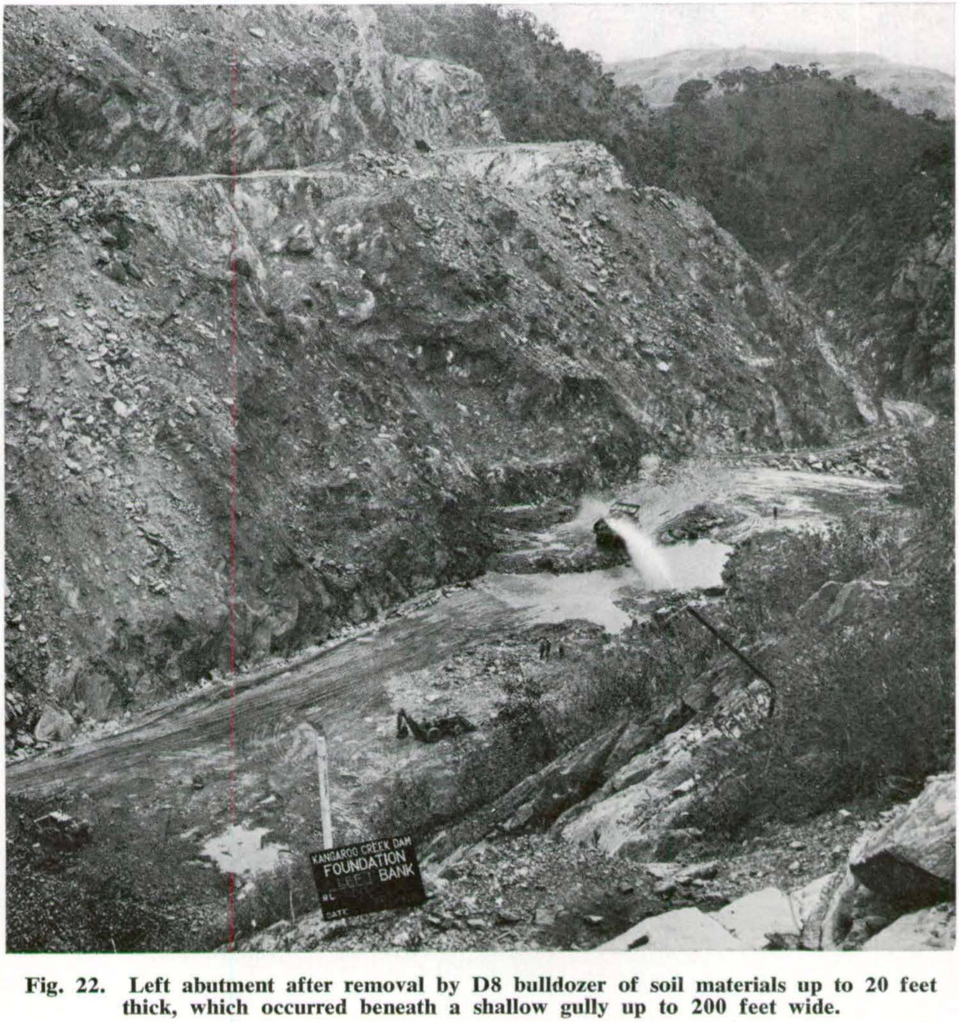

Fig. 22. Left abutment after removal by D8 bulldozer of soil materials up to 20 feet thick, which occurred beneath a shallow gully up to 200 feet wide.

In order to achieve acceptable foundations it was necessary to remove:—

1. Topsoil, spoil, tree stumps etc.
2. Completely and highly weathered rock except where it occurs in localized zones, too narrow to be excavated.
3. Masses of highly disturbed rock containing a high proportion of soil materials.

The specifications required the progressive removal of loose materials as the embankment advanced upwards. Loose materials were defined as those which could be readily removed by small excavating machines operating from the surface of the embankment.

Method of excavation

A small back-hoe loader machine was used to remove soil materials for approximately five feet above the embankment surface. This method proved satisfactory in most places. In those places where slopes were steep and loose material exceeded four feet in thickness, the excavation commonly tended to become unstable. In the central part of the left abutment where loose material was up to 20 feet thick, it was removed by Caterpillar D8 bulldozers (Figs. 22 and 25). Final clean-up was then carried out as specified.

The lower part of the right abutment, and the river channel were covered by up to 20 feet of road spoil, which was removed using front-end loaders, followed by more detailed cleaning of the remaining pockets of soil using back-hoe machines (Fig. 23).

Fig. 23. Foundation clean-up in downstream part of river bed.

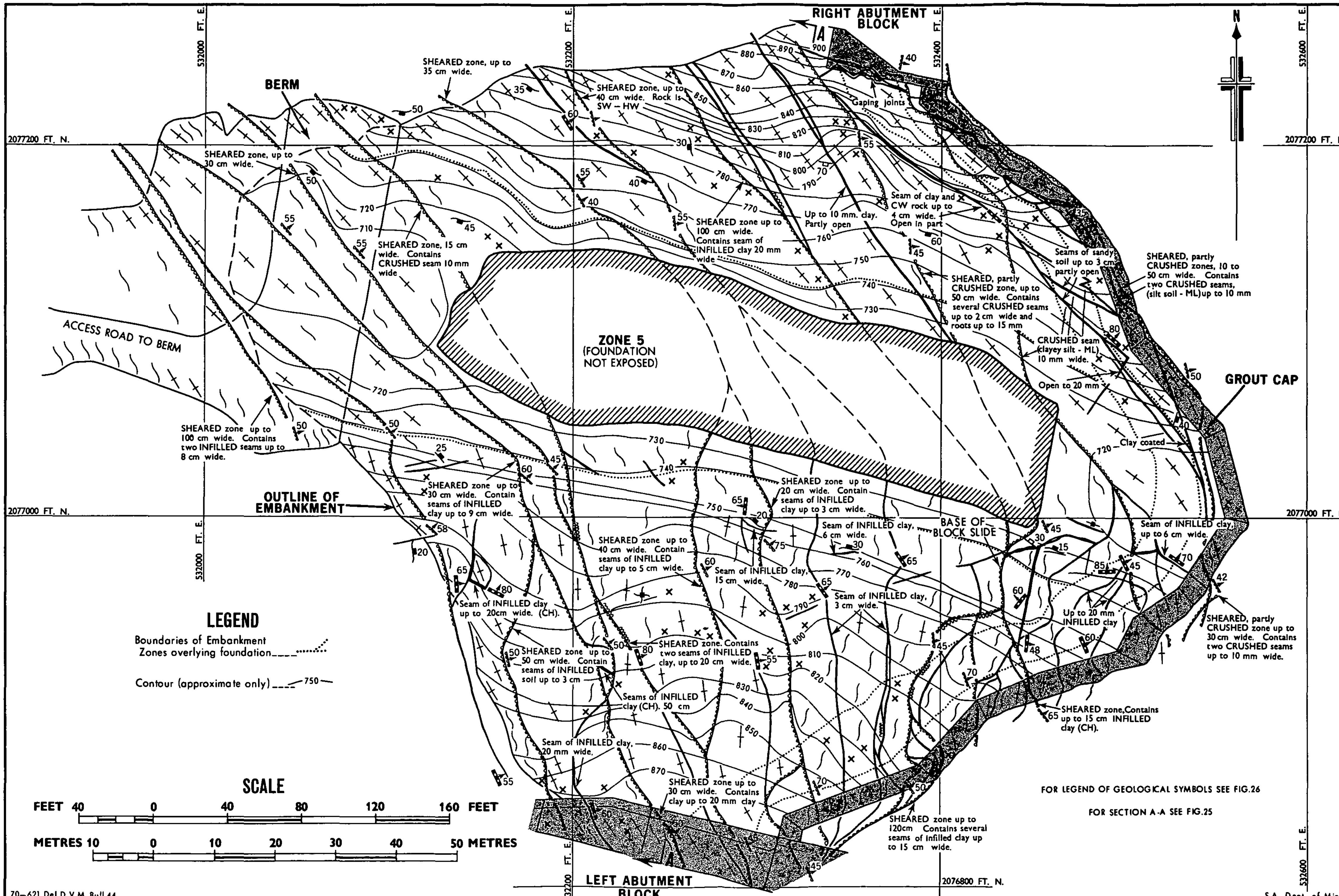

Fig.24. Embankment foundation; geological plan.

Geological investigations

After each horizontal strip of foundation had been excavated, it was inspected by the resident geologist before placement of rockfill. At vertical intervals of 20 feet, the five feet high strip of foundation above the embankment was logged in plan on a scale of one inch to 20 feet. A geological plan of the foundation showing the major geological weaknesses (Fig. 24), was compiled from these strip plans.

Foundation conditions

The geology of the foundations under all zones except zone 5, is shown in plan in Fig. 24 and in cross-section in Fig. 25.

Right bank. The rock substance in the foundations consists mainly of fresh to slightly weathered gneiss with minor amounts of fresh schist and bands of slightly to moderately weathered granitic gneiss up to 10 feet wide.

The rock mass is relatively intact and appears to be undisturbed except for the presence of numerous sheet joints and Set (1) joints containing up to five centimetres of highly plastic clay. Sheared zones up to 100 centimetres wide, some containing crushed seams up to two centimetres wide, are spaced from five feet to 80 feet apart.

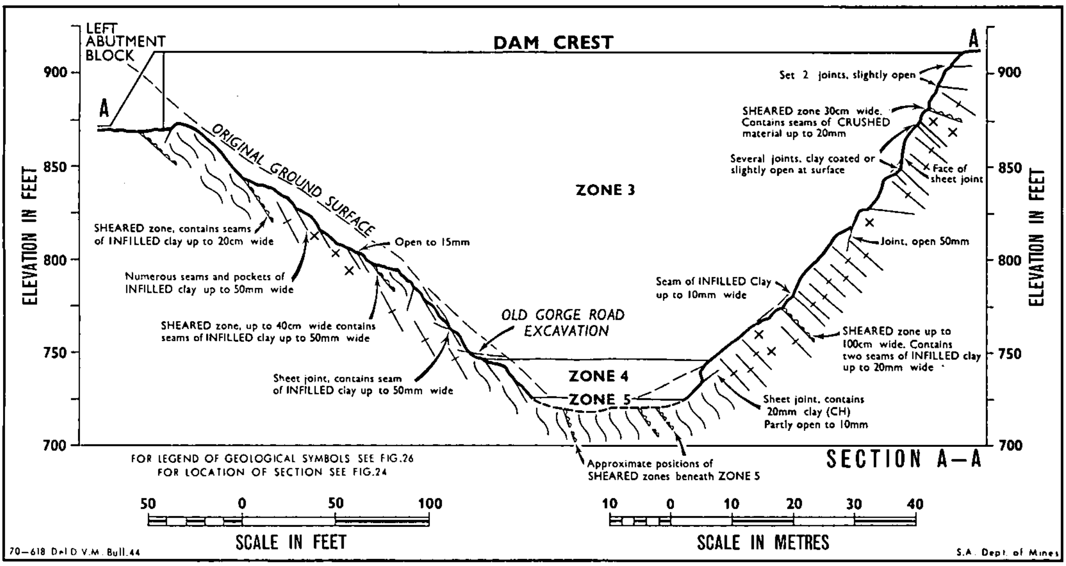

Fig. 25. Embankment foundation; geological section along dam axis.

River bed. The rock foundations were not exposed in the central part of the river bed where existing spoil materials were compacted to form zone 5 of the embankment. However the foundation geology may be substantially inferred from the exposures on either side (Fig. 24). Upstream and downstream of zone 5 the foundations consist of mainly fresh schist and gneiss with several sheared, partly crushed zones 10 to 50 feet apart. The surface contains numerous potholes ranging from 20 to 100 centimetres in diameter and up to 40 centimetres in depth. Channels up to 50 centimetres deep, occur along the traces of the sheared zones and several smaller channels occur along prominent joints.

Left bank. The foundations consist of slightly weathered schist with bands of slightly to moderately weathered gneiss up to 40 feet wide, and minor moderately weathered granitic gneiss.

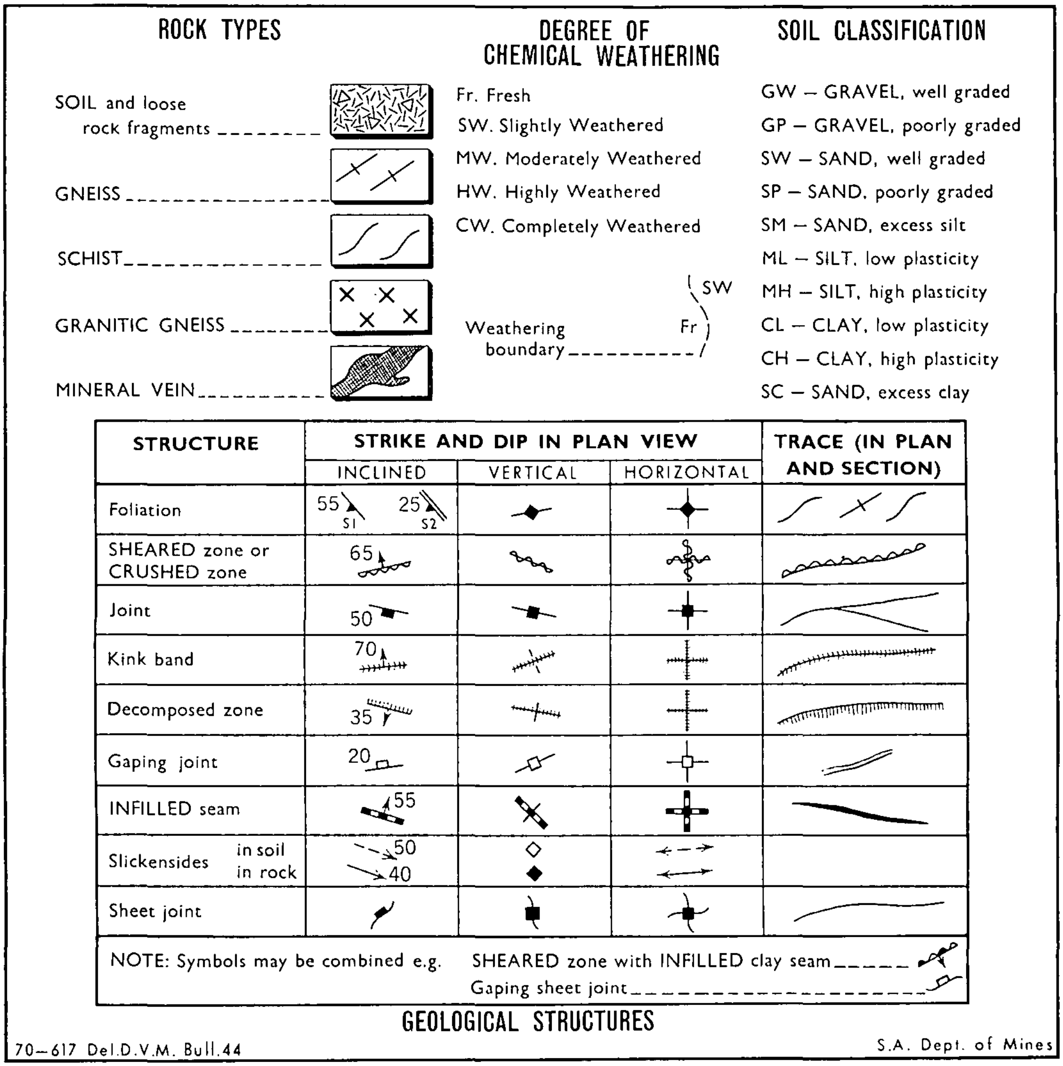

Fig. 26. Legend of geological symbols and abbreviations used in Figures.

Two main gullies which occur in the foundation appear to have been formed by the removal by sliding of wedge-shaped rock masses.

Sheared zones up to 210 centimetres wide occur spaced up to 80 feet apart. Most of the rock mass has been affected by small scale sliding movements which have resulted in the formation of numerous seams of infilled clay, some up to 50 centimetres wide (Fig. 24).

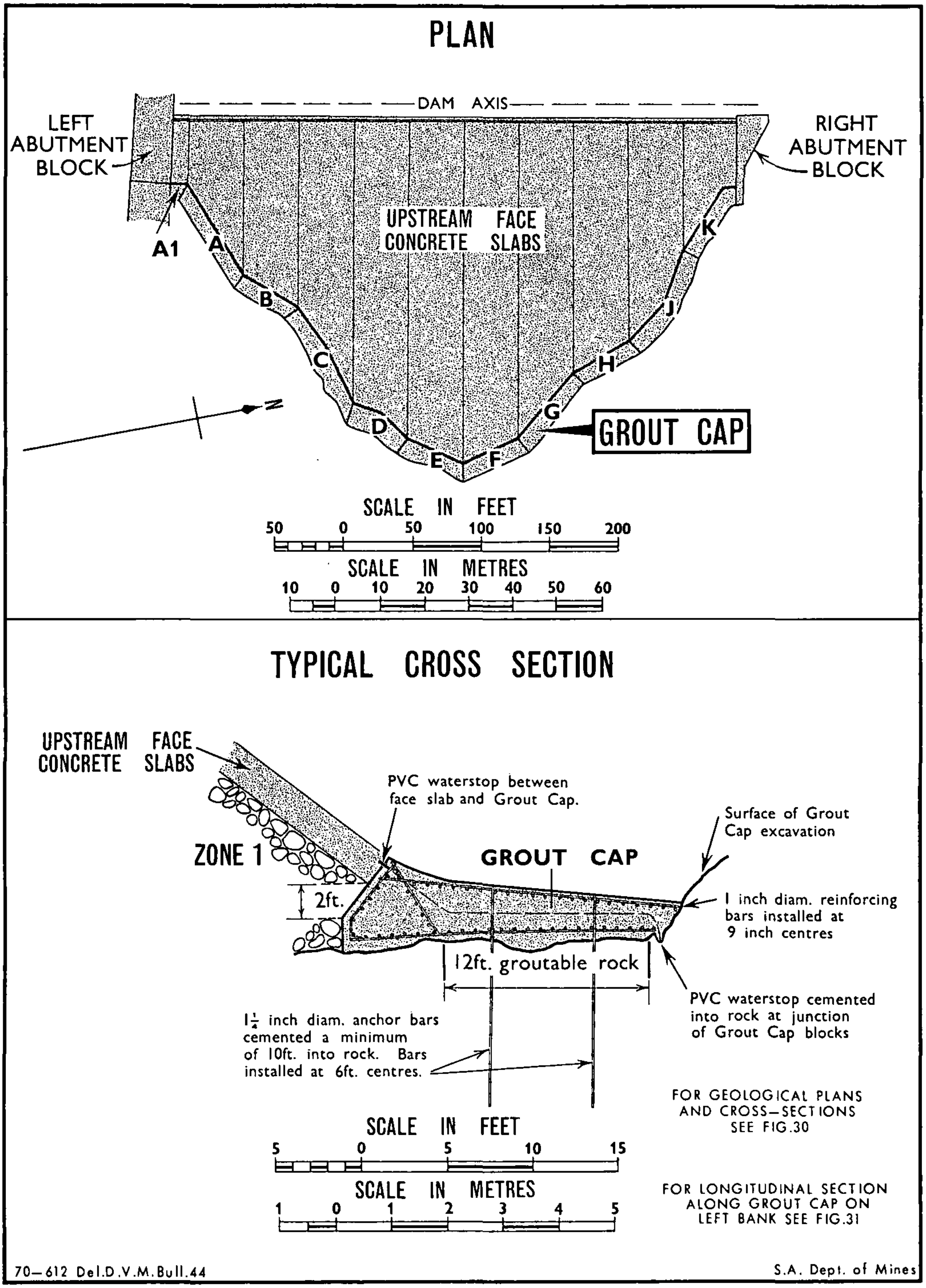

Fig. 27. Grout cap; layout and detail of design.

Grout Cap

Design

The grout cap consists of a series of 11 concrete blocks marked A to K on Fig. 27, which connect the concrete slabs of the upstream face to the foundation rock at the upstream periphery of the dam embankment (Figs. 27 and 28). The blocks are reinforced by two layers of steel mesh tied into the rock mass by a pattern of grouted anchor bars (Figs. 27 and 29). Individual blocks are connected to each other and to their respective face slabs by nine inch wide PVC waterstops.

Fig. 28. Construction of grout cap on right bank. Blocks F, G and K are complete. Pouring of block H in progress.

The specifications stipulated that the grout cap be founded on groutable rock over a minimum horizontal width of 12 feet, and that the concrete have a minimum thickness of T with $T = \frac{h \times 12}{200} + 12$ inches.

"h" is the vertical distance in feet up to the dam crest at reduced level 910.00.

A major function of the grout cap was to facilitate the formation of a grout curtain in the upper part of the foundation. Grout pipes were installed prior to pouring of each block.

Method of excavation

Right bank. Excavation commenced in the lower part of the valley with removal of spoil from the Gorge Road deviation by a dragline machine. Further hand clean-up of loose material was followed by close-pattern drilling and blasting, and hand clean-up of the blasted rock. A batter was formed by pre-splitting, with holes at angles ranging from 76 degrees (1 on 0.25) to vertical, and spaced about two feet apart. The depth of excavation below the natural surface ranged from 12 feet in block G, to 25 feet in block K.

The total quantity of material removed from the excavation was approximately 3,500 cubic yards of which an estimated 90 per cent consisted of rock materials.

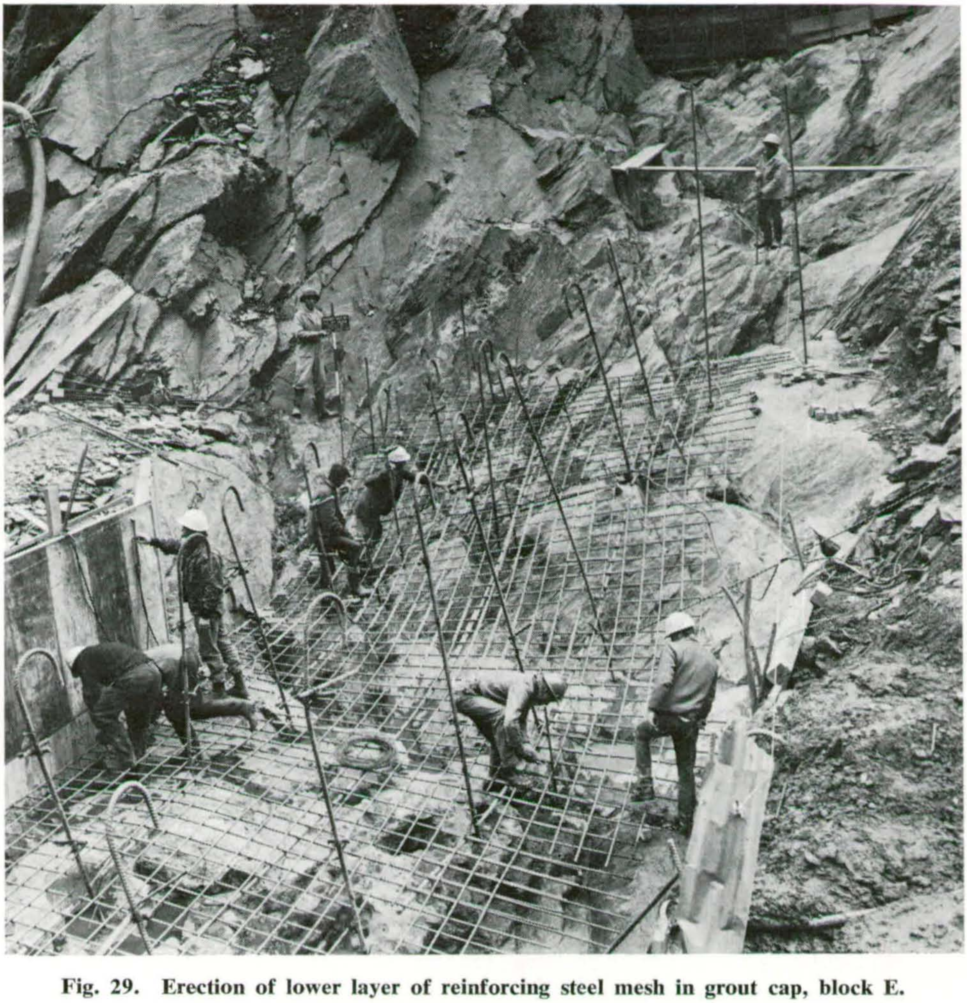

Fig. 29. Erection of lower layer of reinforcing steel mesh in grout cap, block E.

River bed. Gorge Road deviation spoil and river gravel were removed from the river channel by dragline, followed by hand clean-up. No rock excavation was carried out except for a small area adjacent to the left bank.

Left bank. Fill materials beneath the Old Gorge Road were removed by D8 bulldozer and back-hoe shovel. Some close-pattern drilling and blasting was then carried out in the area.

From the Old Gorge Road excavation up to RL815 through a jagged rocky ridge, excavation was carried out in several steps by close-pattern drilling and blasting.

Above RL815 excavation of the soil-filled gully was initially by hand methods, and later by Caterpillar D8 bulldozer. This was followed by close-pattern drilling and blasting of the rock mass, removal of blasted material by bulldozer and hand clean-up. This process was repeated until acceptable foundations were exposed.

For most of its length the batter was pre-split at an angle of 76 degrees (1 on 0.25). The depth of the excavation below the natural ground surface at the toe of the batter ranged from three feet in block D to almost 30 feet in block B. The total quantity of material removed from the excavation was approximately 5,500 cubic yards of which an estimated 70 per cent consisted of rock materials.

Requirements of foundations

As recommended by the consultants, Moye and Rudd (1966), the criteria for acceptance of foundations for the grout cap were that they be:—

1. Strong.
2. Durable—there should be no risk of rock substances disintegrating in water.
3. Impervious, or capable of being made so by dental treatment or grouting.
4. Non-erodible—the grouting will not be 100 per cent effective; if leaks develop through rock joints the adjacent rock should be capable of withstanding erosion.
5. Mechanically stable, that is, not detached from the rock mass.

In order to achieve such foundations it was necessary to remove:—

1. Topsoil and organic matter including roots, except for occasional roots penetrating deeply into otherwise acceptable rock.
2. Completely, highly and moderately weathered rock, except where it occurred in narrow zones.
3. Detached or loosened pieces of rock, including pieces loosened by blasting, except where such loose rock would be surrounded by concrete.
4. Rock containing seams of infilled soil.

In practice it was not possible to achieve this standard of foundation in some areas, even after considerable excavation. In such places extra foundation treatments were undertaken to reinforce the rock mass and to improve the bond between concrete and rock.

Geological investigations

The following procedure was adopted to determine the suitability of each section of grout cap foundation:—

1. Immediately after excavation and preliminary clean-up, the section was inspected by the resident geologist and the feature engineer.
2. If further major excavation was obviously required it was immediately specified.
3. If no further major excavation appeared necessary, the foundation was geologically logged in plan and section on a scale of one inch to 10 feet. The results of this logging are included in Fig. 30. In some areas further clean-up was required to delineate the extent of particular geological weaknesses.
4. A factual geological report was prepared and submitted, along with recommendations for further foundation treatments, to a meeting of the Foundation Committee*. This committee either passed the foundation or specified further excavation.
5. Anchor bar holes were probed with a "scraping rod"† to locate sub-surface defects, and in particular to reveal any major defects not exposed in the foundations. The results of these probes were used to determine modifications to the pattern and depth of anchor bar holes, and to interpret the results of grouting operations. In several areas the probes revealed thick seams, resulting in further excavation.

Foundation conditions

Right bank (blocks G, H, J and K—Fig.30). The foundation rock is mainly fresh to slightly weathered gneiss. Slightly weathered granitic gneiss occurs over 30 per cent of the foundation area in blocks H and J. Fresh schist occurs in minor amounts in the upper parts of block K.

The foundation surface for a large area under blocks G, H and J is formed by the face of a narrow sheared zone containing a crushed seam up to two centimetres wide, which is approximately parallel to the embankment

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* The Foundation Committee consisted of the following members:—Engineer for Construction, Engineer for Design, Engineer for Water Supply, Design Engineer—Dams Section (all Engineering and Water Supply Department), and Senior Geologist, Engineering Geology Section (Department of Mines).

† The scraping rod consists of a three-eighths of an inch diameter solid metal rod of circular cross-section with a pointed projection at one end. Weaknesses are detected by scraping the point against the side of the hole.

upstream face. Joints and thin weathered seams which branch out from the sheared zone, also form prominent faces in the foundation. This zone of joints also underlies block K (section KA, KB and KC, Fig. 30) and was penetrated by most of the anchor bars installed in this area.

Disturbed rock containing numerous gaping joints, some open to 10 centimetres (four inches), forms the foundation in the upper part of the excavation (block K and a small part of block J). The origin of the gaping joints is discussed in the section on the right abutment block (pages 83 and 84).

The remainder of the foundation was relatively intact with narrow gaping joints open less than one centimetre, and a few narrow seams of infilled soil less than one centimetre wide. Adjacent to block H on the downstream side, the rock contains several gaping joints and infilled seams up to 15 centimetres wide. However this disturbed zone lies almost completely outside the grout cap foundations.

River bed (blocks E and F—Fig. 30). The foundation rock is mainly fresh gneiss with minor amounts of fresh schist and granitic gneiss. A zone up to three feet wide containing moderately weathered pyritic gneiss, occurs in the central part of block E.

Much of the foundation surface of the lower part of block F and most of block E consists of potholes from 25 to 50 centimetres in diameter and up to 40 centimetres in depth. A prominent sheared zone up to 100 centimetres wide containing two narrow crushed seams, occurs in block E. This zone was intersected by anchor bar holes drilled in the northern part of the block. Several narrow seams up to 15 millimetres wide, of infilled sand and clay occur in the foundations, and the scraping of anchor bar holes suggests that one or more narrow, near-horizontal sheet joints occur within 10 feet of the rock surface.

Left bank (blocks A, B, C and D—Figs. 30 and 31). The foundation rock is mainly slightly weathered schist. Bands of slightly weathered gneiss occur in the lower part of block D and the upper part of block A. Granitic gneiss ranging from moderately to slightly weathered, comprises 40 per cent of the foundation for block B, but does not occur in the other blocks.

The final surface of the foundations for blocks A, C and D was step-shaped, with Set (1) joints and seams dipping slightly steeper than the overall slope, forming the back of each step (Fig. 31). The foundation for block B was approximately horizontal.

The foundation rock mass contains numerous seams of highly plastic infilled clay up to 20 centimetres wide (Figs. 30 and 31). The origin of these seams is discussed in the section on the stability of the left abutment (pages 141 to 150).

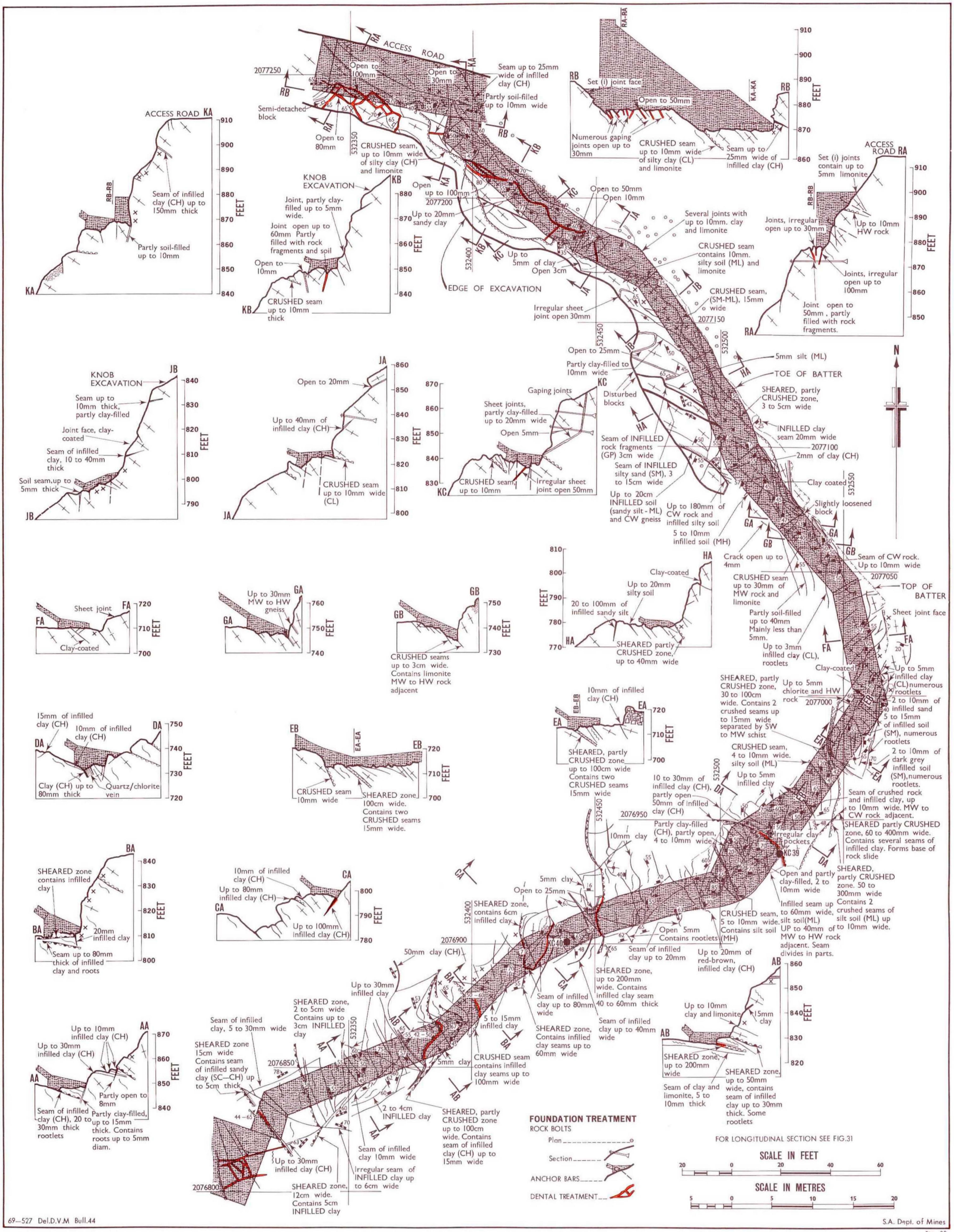


Fig.30 Grout cap,left and right bank,plan and sections.

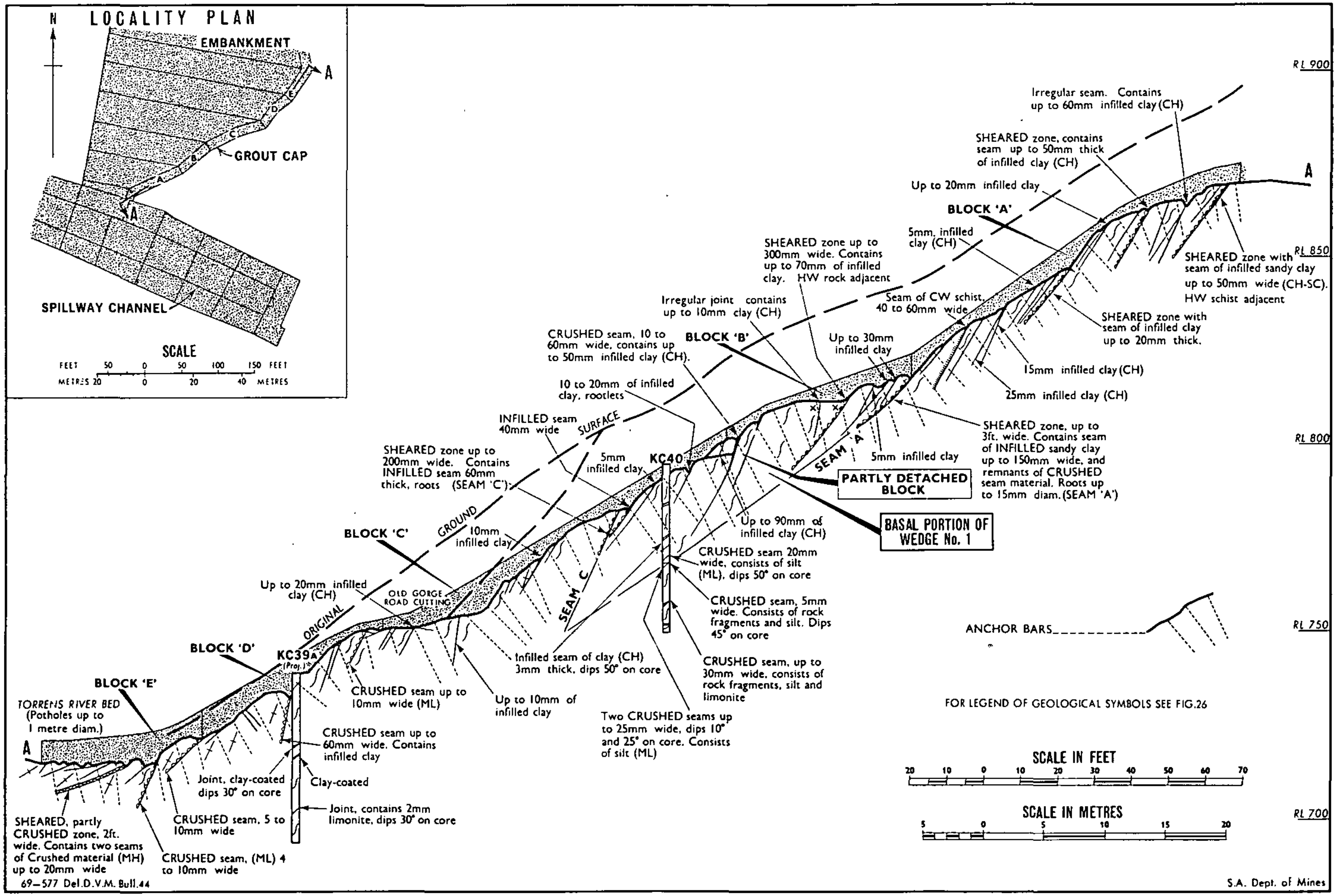


Fig. 31. Section along grout cap—left bank.

Foundation treatments

In several areas the rock mass in the foundation did not meet the required criteria for strength and durability and it was apparent that these conditions extended to considerable depth.

Two types of remedial treatment were carried out in these areas and are shown in Fig. 30.

Dental treatment was carried out where soil seams more than 50 millimetres in width intersected the foundation surface. The purpose of this treatment was to guarantee an effective bond between the concrete and the foundation rock and to avoid water seepages along the interface which could result in erosion of the seam materials.

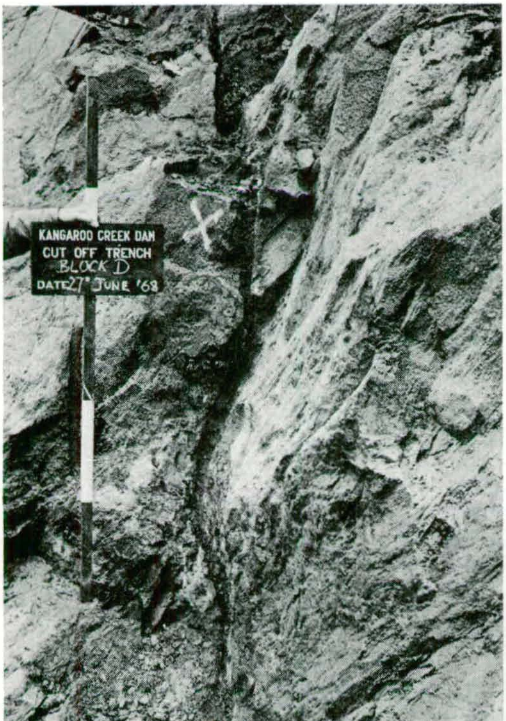


Fig. 32. Foundation for grout cap with infilled clay seam 5 cm wide before dental treatment.

Fig. 33. Foundation for grout cap with infilled clay seam 5 cm wide after dental treatment.

The procedure adopted was as follows:—

1. The seam and adjacent rock was excavated using jack-hammers, to form a channel of U-shaped cross-section in the foundations and for one or two feet on the upstream side (Figs. 32 and 34).

2. The excavation was throughly cleaned by air/water jet.

3. The excavation was filled by well-vibrated concrete (Fig. 33). The concrete was poured before the seam materials could dry out below their natural moisture content.

This type of treatment was applied to seven seams all on the left bank, for which a total of approximately 40 cubic feet of concrete was used.

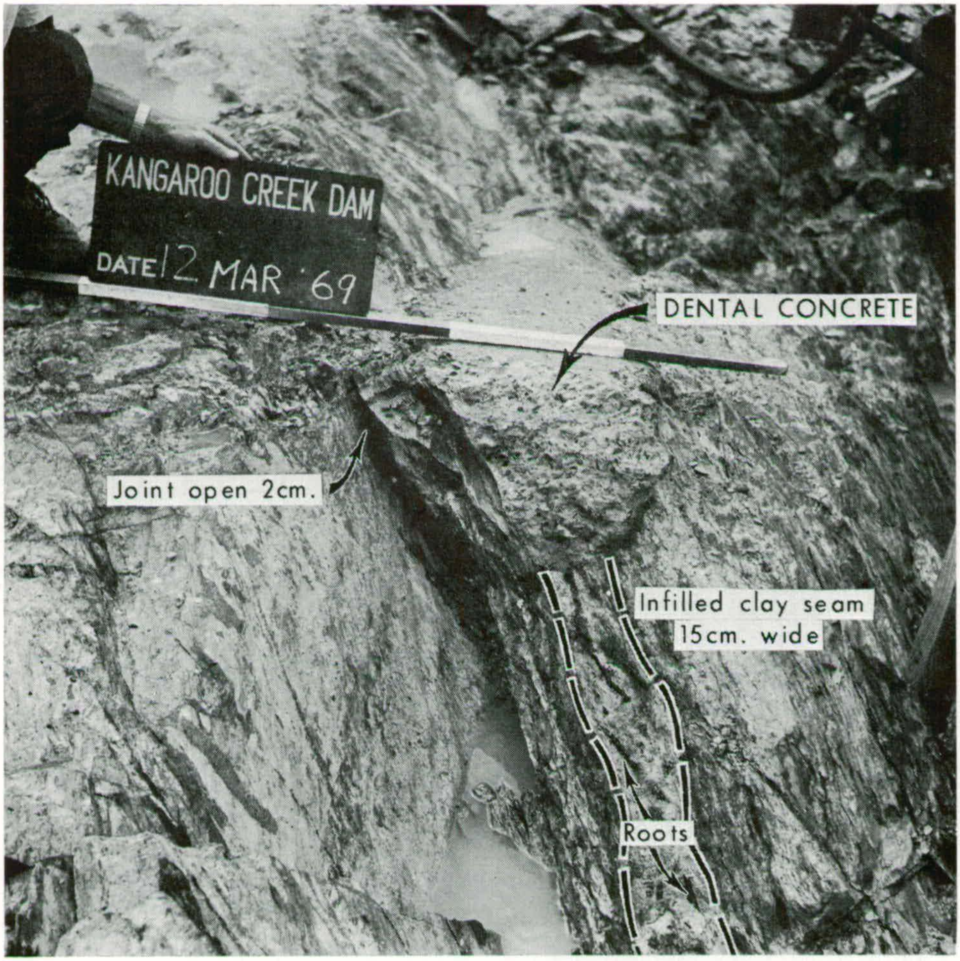
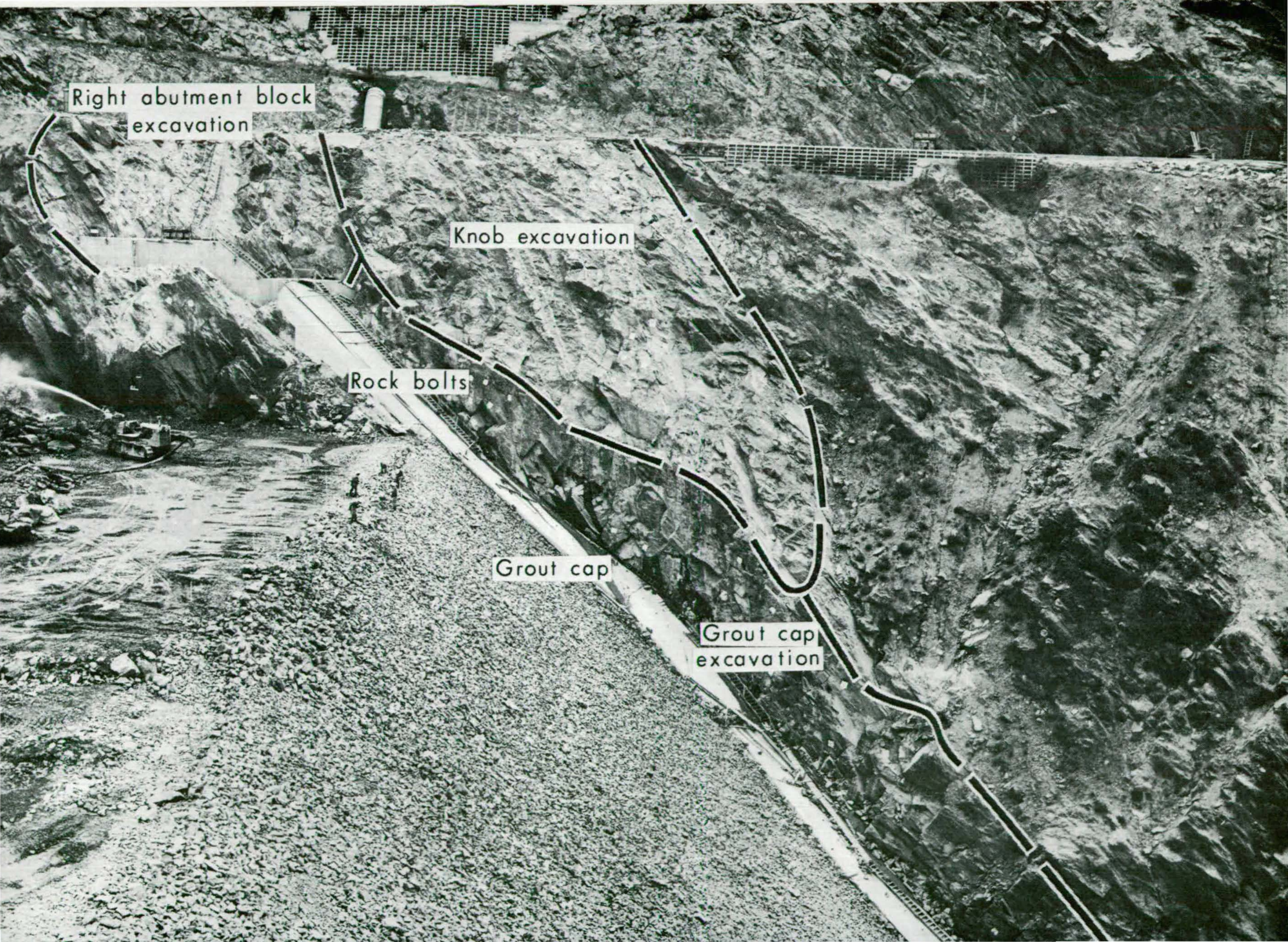


Fig. 34. Cut-away view of dental treatment of infilled clay seam in foundation for grout cap, block A.



Right abutment block excavation
Knob excavation
Rock bolts
Grout cap
Grout cap excavation

A different type of dental treatment was adopted on the right bank in areas of gaping joints. Joints open more than three centimetres were treated by pouring one to three cement/sand mortar into them until overflow occurred. A total of 10 cubic feet of mortar was used in the foundation for block K.

Anchor bars additional to the six feet pattern specified, were installed to reinforce parts of the foundation which were partly disturbed or semi-detached. In some cases the extra bars were installed to depths of 20 feet into the foundation. Approximately 120 linear feet of bars were installed in blocks A, B and C on the left bank, in addition to the specified pattern.

Stability of final cuttings

Right bank. The batter ranges in height from 10 feet to approximately 30 feet and slopes towards the embankment at 70 to 90 degrees (Figs. 35 and 30—cross-sections).

Joint faces forming the batter are rare and the traces of drill holes are visible in most places.

Above blocks K and J several clay-filled sheet joints occur, dipping towards the embankment at 20 to 40 degrees (sections KC, JA and JB, Fig. 30). The joint blocks overlying these joints were pinned by rock bolts to the underlying rock mass. Rock bolts were also installed in other parts of the batter to minimise loosening of jointed areas and to pin individual blocks.

A total of 28 expanding-shell rock bolts were installed, ranging in length from eight to 20 feet. They were all grouted to ensure permanent support of the batter.

Left bank. The batter ranges in height from zero feet to 32 feet, and slopes towards the embankment at 40 to 65 degrees. Set (2) joints are near-parallel to the batter direction and form the excavation surface above most of block C and parts of blocks A and B.

A wedge-shaped mass of rock which occurs above the upper part of block C, dips towards the embankment at about 30 degrees. The stability of this mass is discussed in "Stability of left abutment" (pages 141 to 147). No other large masses dipping towards the embankment were apparent. Small joint blocks up to two cubic yards in volume which occur in the upper part of the excavations, are likely to become unstable after filling of the reservoir.

Right abutment block

Design

The right abutment block located at the top of the right abument, is an irregular block of mass concrete, approximately triangular in both longitudinal and cross-section (Fig. 30).

Fig. 35. Right abutment upstream of embankment.

The dimensions are as follows:—

Base length	70 feet
Minimum (base) width	5 feet
Maximum (crest) width	22 feet
Maximum height	36 feet
Volume	800 cubic yards (approximately)

The block forms a continuation of the grout cap, to which it is connected by nine inch PVC waterstop.

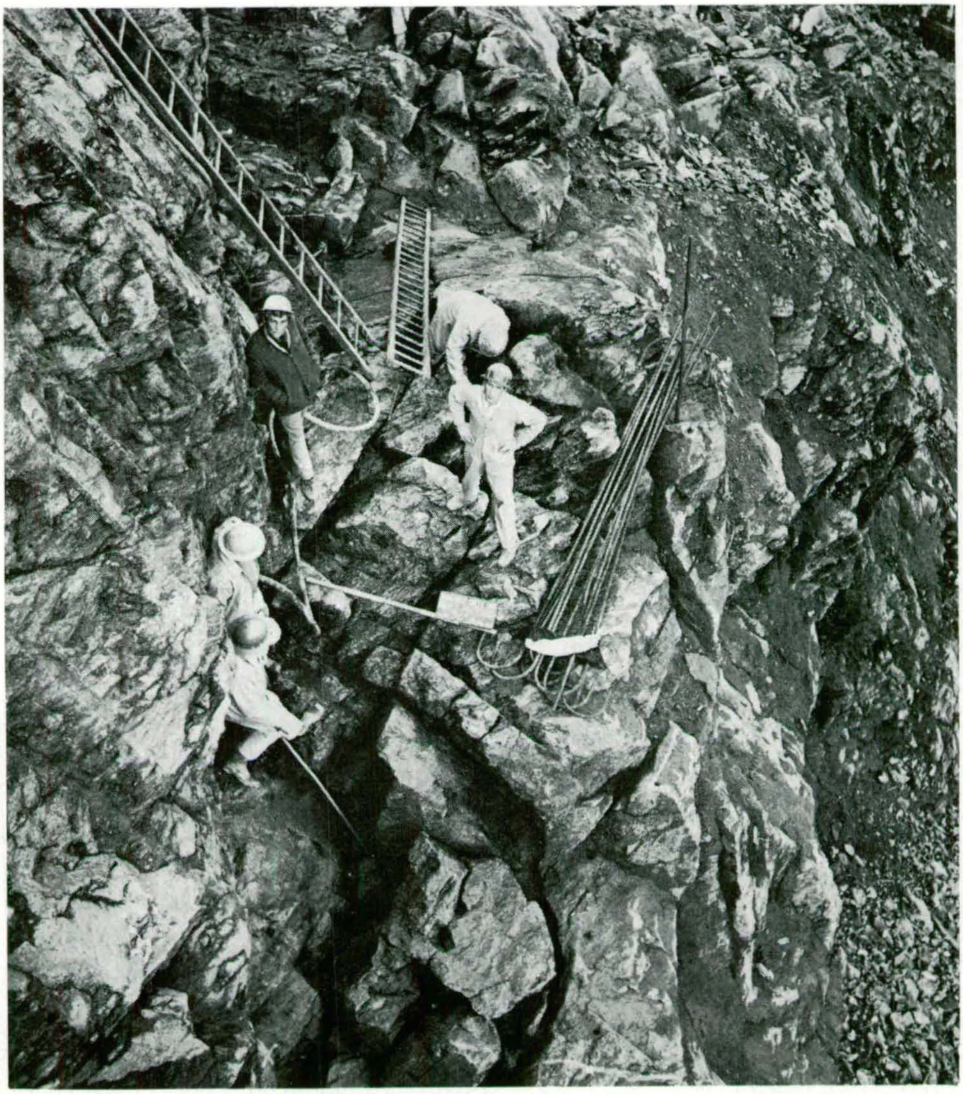


Fig. 36. Floor of right abutment excavation, with numerous gaping joints open up to 30 cm.

Method of excavation

The excavation was carried out in two horizontal stages by drilling and blasting from above, and removal of loose material by hand barring.

Foundation conditions

The excavation was mainly in fresh gneiss with some slightly weathered granitic gneiss. The rock mass in the north and east (upstream) batters and the upstream half of the floor has been only slightly affected by mechanical weathering, however the upstream part of the floor and part of the downstream wall are in the zone of severe mechanical weathering. Hand clean-up of the floor in this area revealed numerous near-vertical gaping joints (Figs. 30 and 36), open up to 15 centimetres and apparently extending to considerable depths.

Fig. 37. Dental treatment of gaping joints in floor of right abutment block excavation. After clean-up joints were filled with slurry of 1 to 3 sand/cement mortar.

It is considered that these joints, most of which were near-parallel to the steep valley wall, were opened up by mechanical weathering associated with stress relief. Detailed examination of the rock in the valley wall for 100 feet below the foundation failed to reveal any structural defect which could have formed the base for these movements, and it was concluded that the movements had been rotational with the tops of the blocks moving outward toward the valley, as shown in Fig. 38.

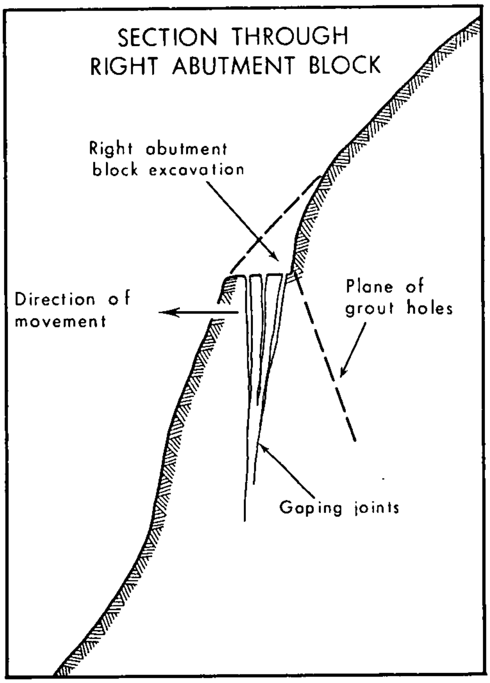


Fig. 38. Section through right abutment block showing direction of movement that caused opening up of joints.

Foundation treatments

It was apparent that the foundation rock as exposed, was not adequate to support the weight of the block without some treatment. It was also apparent that further movements of the disturbed area would be prevented by the presence of the embankment once it was constructed. However, it was considered that there was a possibility of movements occurring before the embankment reached the level of the block. To avoid such movements and to strengthen the foundations, the following measures were adopted:—

1. Only the lowermost five feet deep concrete lift was placed before the embankment advanced within 20 feet of the base of the block.
2. A single row of six by 20 feet long rock bolts was installed below the base of the block.
3. The gaping joints were treated with thick sand/cement (three to one) mortar slurry (Fig. 37), following the procedure outlined on page 81.

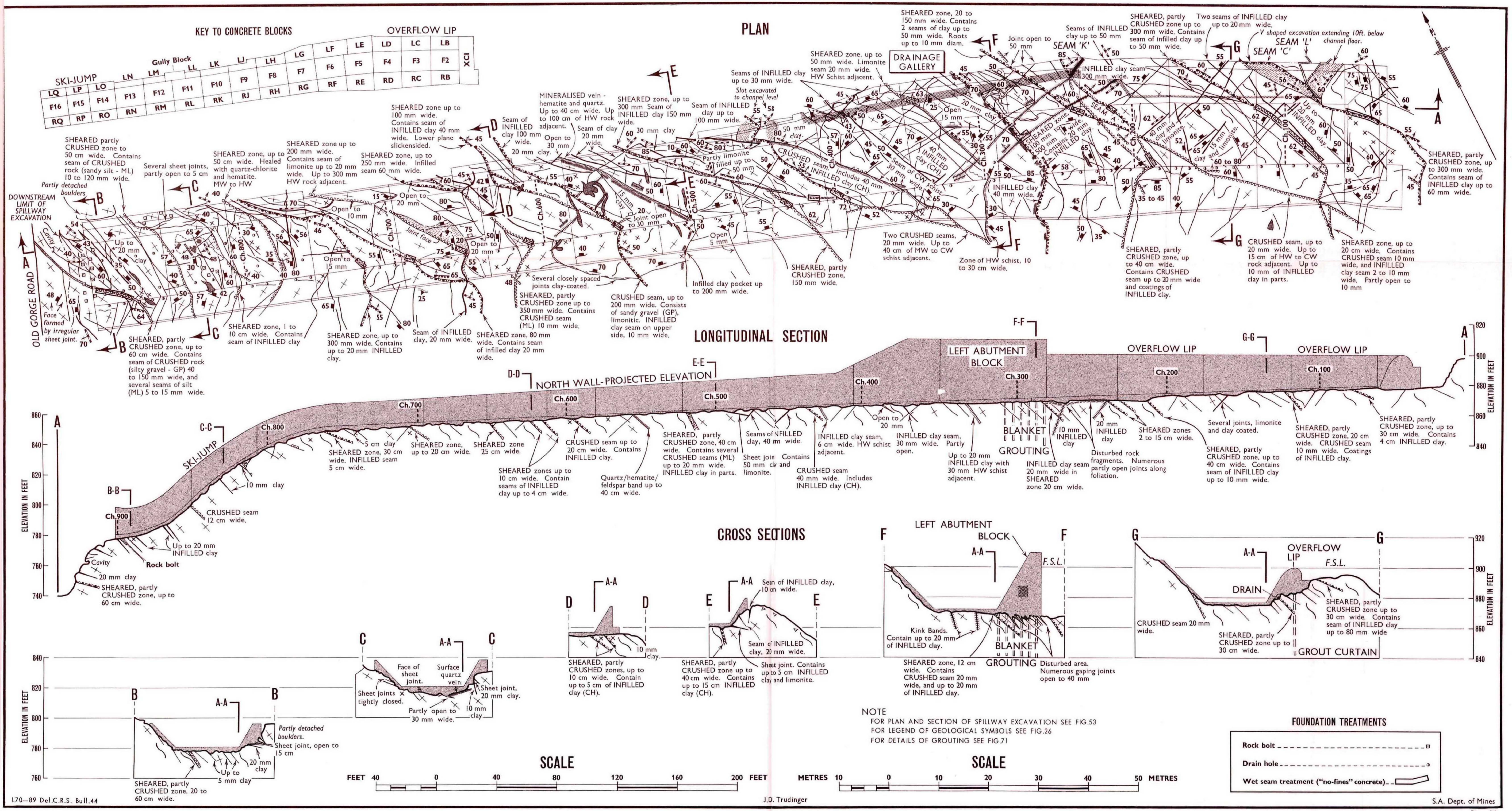


Fig. 39

Fig.39 Spillway channel,plan and sections.

4. Mortar pads were installed across the major joints to indicate any movements.
5. A total of 15 anchor bars were installed in the north wall of the foundations to help support the block.
6. The grout curtain which had been designed as a series of vertical holes in a plane at right angles to the dam axis, was angled into the abutment at 70 degrees to avoid the highly disturbed area.
7. Three rock bolts were also installed in the batters to stabilize individual semi-detached blocks.

Spillway channel

Design

The concrete-lined spillway channel occupies most of the lowermost bench of the spillway excavation (Fig. 53). The outlines of the main parts of the structure are shown in plan and section on Fig. 39.

The main dimensions are as follows:—

Overall length	880 feet
Overall width	70 to 110 feet
Width of invert	40 to 42 feet
Slope of invert (up to start of ski-jump)	1 on 24
Height of wall above invert	15 to 42 feet
Maximum slope of ski-jump invert	45 degrees
Slope of downslope (north) wall	60 degrees
Slope of upslope (south) wall	45 degrees
Minimum specified thickness of concrete slabs	1 foot
Number of concrete slabs	47
Length of overflow lip	310 feet

The reinforced concrete slabs or blocks are connected to each other by PVC waterstops and are anchored to the foundations by grouted anchor bars installed eight feet into rock, on a six by six feet pattern.

The north wall of the spillway adjacent to the embankment is formed ɔncrete gravity blocks of trapezoidal cross-section. In several other places, over-excavation precluded the use of thin slabs as designed, and necessitated the construction of similar concrete gravity blocks., *e.g.* block XCI, Gully block, block LQ.

Drainage of the walls and floor is provided by means of weep holes, and in the overflow lip foundations by a drain which flows through a gallery in the left abutment block (LE), and out onto the downstream face of the embankment.

Requirements of foundations

The criteria for acceptance of foundations for the channel were that they be:—

1. Mainly strong and durable; sufficiently strong for the anchor bars to resist uplift stresses up to 16 tons per bolt.
2. Mechanically intact; that is not detached from the rock mass or with a freedom to become detached.

In practice weak and very weak rock materials and mechanically suspect rock were accepted where they occurred in localized zones or in places where the operational stresses would be low, and the consequences of failure slight, *e.g.*, near the top of the channel wall, downstream of the embankment. In other places rock bolts were installed to strengthen the foundations.

Geological investigations

The following procedures were adopted to determine the suitability of each section of foundation:—

1. Immediately after preliminary clean-up the foundation was inspected by the resident geologist, a geological plan was prepared, and areas marked where special foundation treatment appeared necessary.
2. A joint inspection between the resident geologist and the feature engineer was carried out to specify the details of foundation treatment.
3. After final clean-up, immediately prior to concrete placement, the foundation was inspected again and extra details added to the geological plan.

Foundation conditions

The detailed geology of the spillway channel foundation is shown in plan and section on Fig. 39. The relative occurrence of the main rock types is shown in Table 12.

TABLE 12

OCCURRENCE OF MAIN ROCK TYPES, SPILLWAY CHANNEL

Rock type	Proportion of foundation	Predominant degree of weathering
Schist	50 per cent	Fresh to slight
Gneiss	40 per cent	Slight to fresh
Granitic gneiss	10 per cent	Slight to moderate

Towards the upstream end of the channel, upstream of Ch.250, the foundations consist mainly of schist; and in the ski-jump area downstream of Ch.750, mainly of gneiss. In the central part schist and gneiss occur in approximately equal proportions in alternating bands five to 50 feet apart.

Sheared zones four to 60 centimetres wide and crushed zones 0.1 to 15 centimetres wide occur in the foundations spaced mainly 20 to 100 feet apart and in places merging and intersecting. The rock for distances up to four feet adjacent to many of the sheared zones, is moderately to highly weathered and hence weak to very weak.

Fig. 40. Spillway channel, ski-jump, north wall, blocks LP and LO. Foundations after initial cleaning up showing numerous irregular sheet joints.

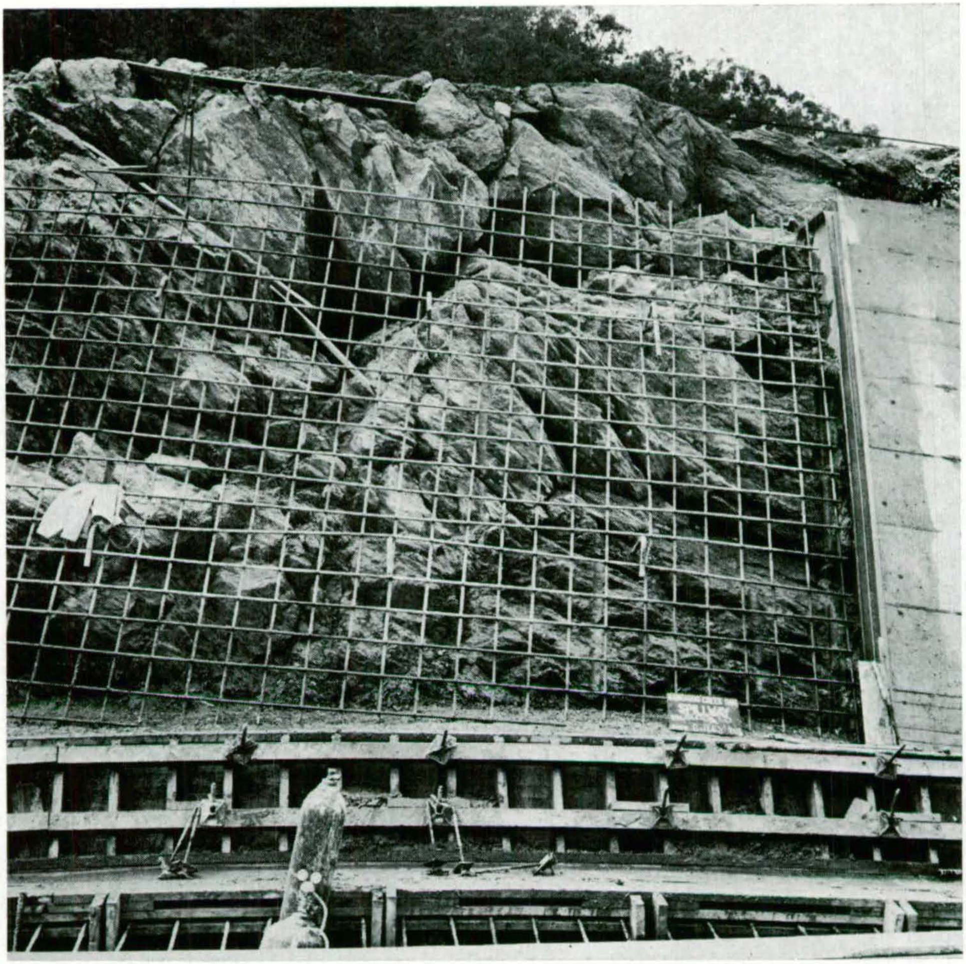


Fig. 41. Spillway channel, north wall, block LO, with disturbed wedge shaped block in upper part of foundation.

As shown on Fig. 53, most of the rock in the foundation has been only slightly disturbed by mechanical weathering and contains few infilled seams or gaping joints. However parts of the foundation for the north wall have been considerably disturbed and contain numerous infilled clay seams, some of which are up to 30 centimetres wide (Figs. 41 and 46). The initial excavations for the overflow lip, left abutment block (Fig. 49), gully block and ski-jump north wall (Fig. 40) exposed highly disturbed rock, and a considerable amount of extra excavation was necessary in each case to expose acceptable foundations.

Overbreak

Overbreak which occurred during excavation of the spillway channel, was due to three main causes:—

1. Inaccuracies and errors in drilling and blasting technique, such as overdrilling, deviation of holes, overblasting, surveying errors, etc.
2. Lack of planarity of pre-split surface due to concavities between pre-split holes.
3. Loosening along geological weaknesses resulting in back-break due to slip-out, or necessitating barring down of joint blocks.

In the central part of the channel floor (blocks F4, F5 and F6) the two four feet of overbreak which occurred was due mainly to over-drilling of burden holes. In other parts of the floor, the overbreak was localized and mainly associated with loosening along geological defects.

In the sloping batters for the north wall overbreak was slight except, in the ski-jump area where up to six feet of highly disturbed rock in the north wall (Fig. 40) was removed by barring.

In the south wall overbreak was up to three feet and was mainly due to inaccurate drilling.

Foundation treatments

Provision was made for drainage of the channel foundations to reduce uplift forces on the channel lining which would otherwise occur due to the presence of a positive head of water in the hillside and in the reservoir. The treatments which are shown in Fig. 39, consisted of the installation of drains, located so as to intersect geological zones along which water could travel. The probable effect of this treatment on the groundwater situation is shown in Fig. 21.

A single line of "weep holes" was drilled on a 10 feet spacing to penetrate through the concrete lining and three inches into rock, along the toe of the upslope (south) wall of the channel. In the channel floor "wet seam drains" were installed to relieve water pressure from all major seams. Upstream of the embankment the seams were drained on both sides of the channel floor. Downstream of the embankment the seam drainage was confined to a short section of three to 10 feet on the southern side.

The procedure adopted for wet seam treatment was as follows:—

1. The seam and adjacent rock were excavated using jack hammers to form a channel of U-shaped or V-shaped cross-section (Fig. 42).
2. The excavation was thoroughly cleaned by air/water jet.

3. The excavation was filled with "no-fines" porous concrete (Figs. 43 and 44) composed of 380 lb of cement per cubic yard of three-quarters of an inch aggregate, and covered with tar paper.
4. A drain pipe was installed from the porous concrete to protrude above the channel floor level.
5. After pouring of the lining the drain pipe was trimmed to the level of the channel floor.

Fig. 42. Narrow crushed seam exposed in the floor of the spillway channel excavation before wet seam treatment.

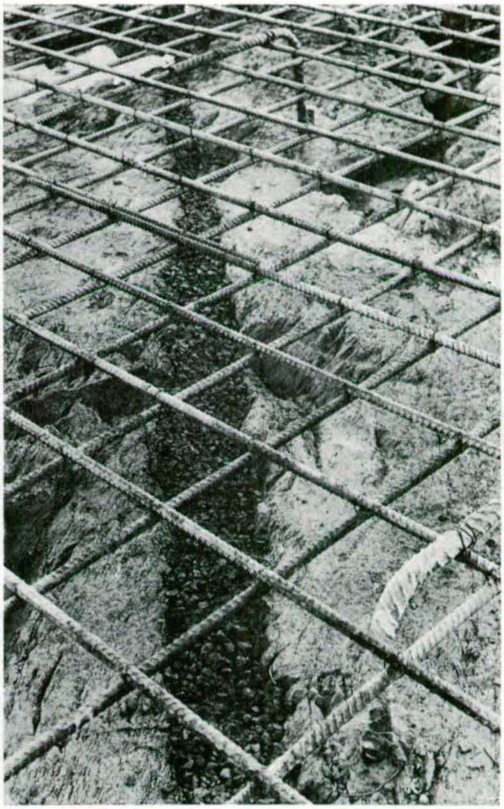


Fig. 43. Narrow crushed seam exposed in the floor of the spillway channel excavation after wet seam treatment.

In three separate parts of the ski-jump foundations the rock mass contained open or clay-filled sheet joints resulting from movement of joint blocks outward towards the valley. These areas were strengthened by the installation of rock bolts eight to 20 feet in length (Fig. 39) which were anchored to intact rock.

The rock mass between the end of the ski-jump and the Old Gorge Road was also strengthened by rock-bolting to pin large, disturbed joint blocks which could otherwise have become detached during periods of spillway discharge, resulting in undercutting of the spillway concrete. Several seams in this area were dentally treated to avoid erosion during periods of small discharge.

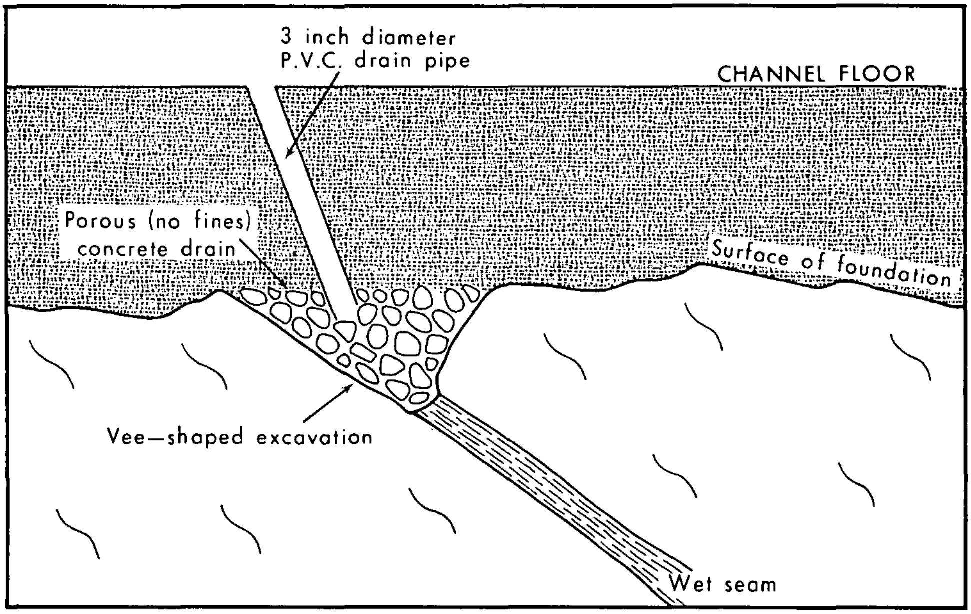


Fig. 44. Diagrammatic section through typical "wet seam drain" in floor of spillway channel.

Changes in design during construction

In several parts of the foundations for the downslope (north) wall of the spillway channel, the presence of geological weaknesses in the rock mass necessitated re-design of the concrete structures.

Spillway overflow lip

(Blocks XC1, LB, LC, LD and LE)

In the original specifications the overflow lip was designed as a series of concrete slabs topped by curved gravity crest blocks, and anchored to the foundation rock mass by means of a pattern of bars installed horizontally through the slabs at six feet centres (Fig. 45). During excavation it became evident that the rock mass in this area had been disturbed by small-scale slide movements (see pages 145 to 150). It was considered that this rock mass, particularly in the upper part of the ridge between the channel excavation and the valley slope, could not be relied on to provide the necessary resistance to the dynamic stresses which would occur during peak overflow. In particular it was considered that the spillway concrete should be separated from one particular wedge-shaped mass which was considered to be unstable (wedge mass No. 2).

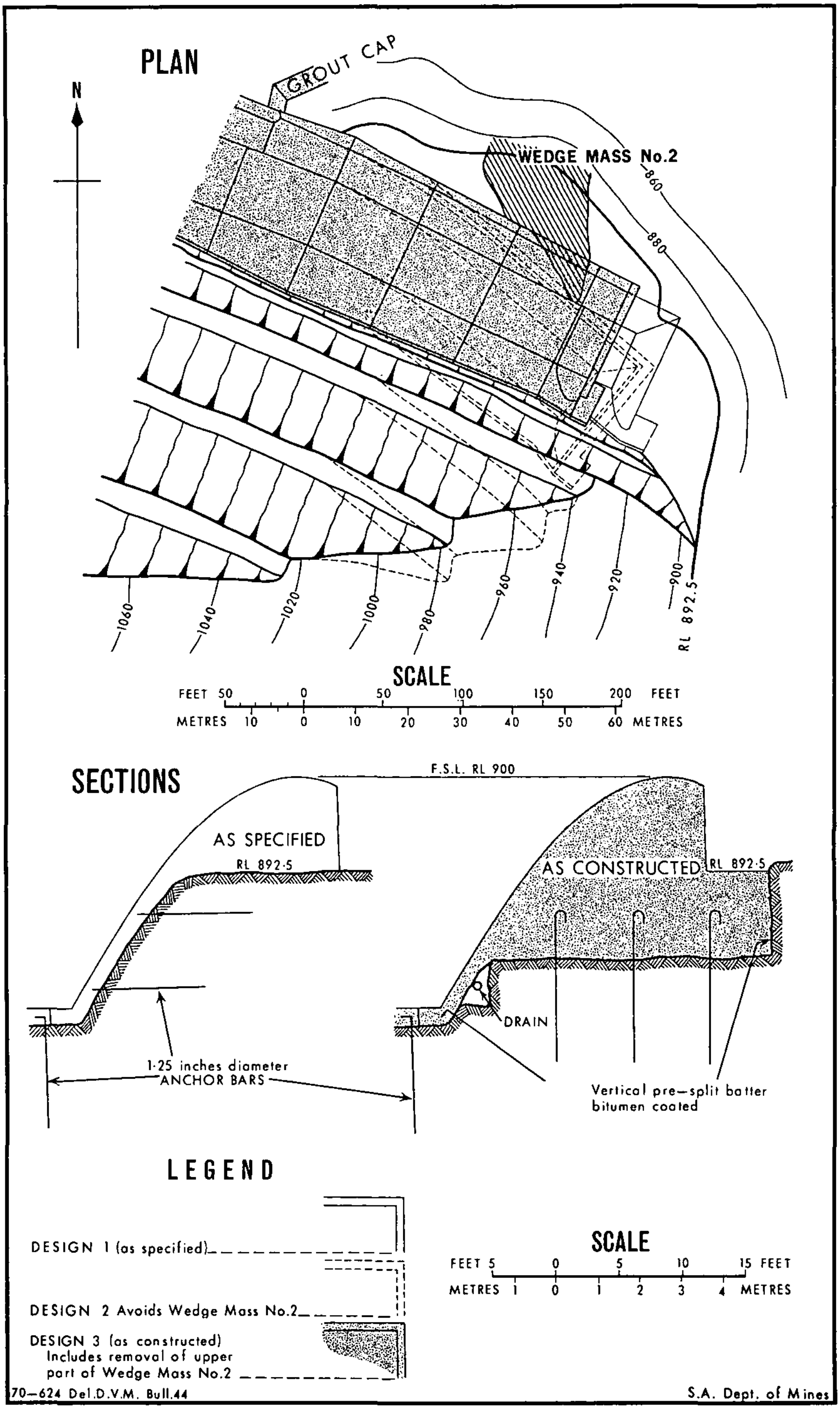


Fig. 45. Spillway overflow lip; design changes during construction.

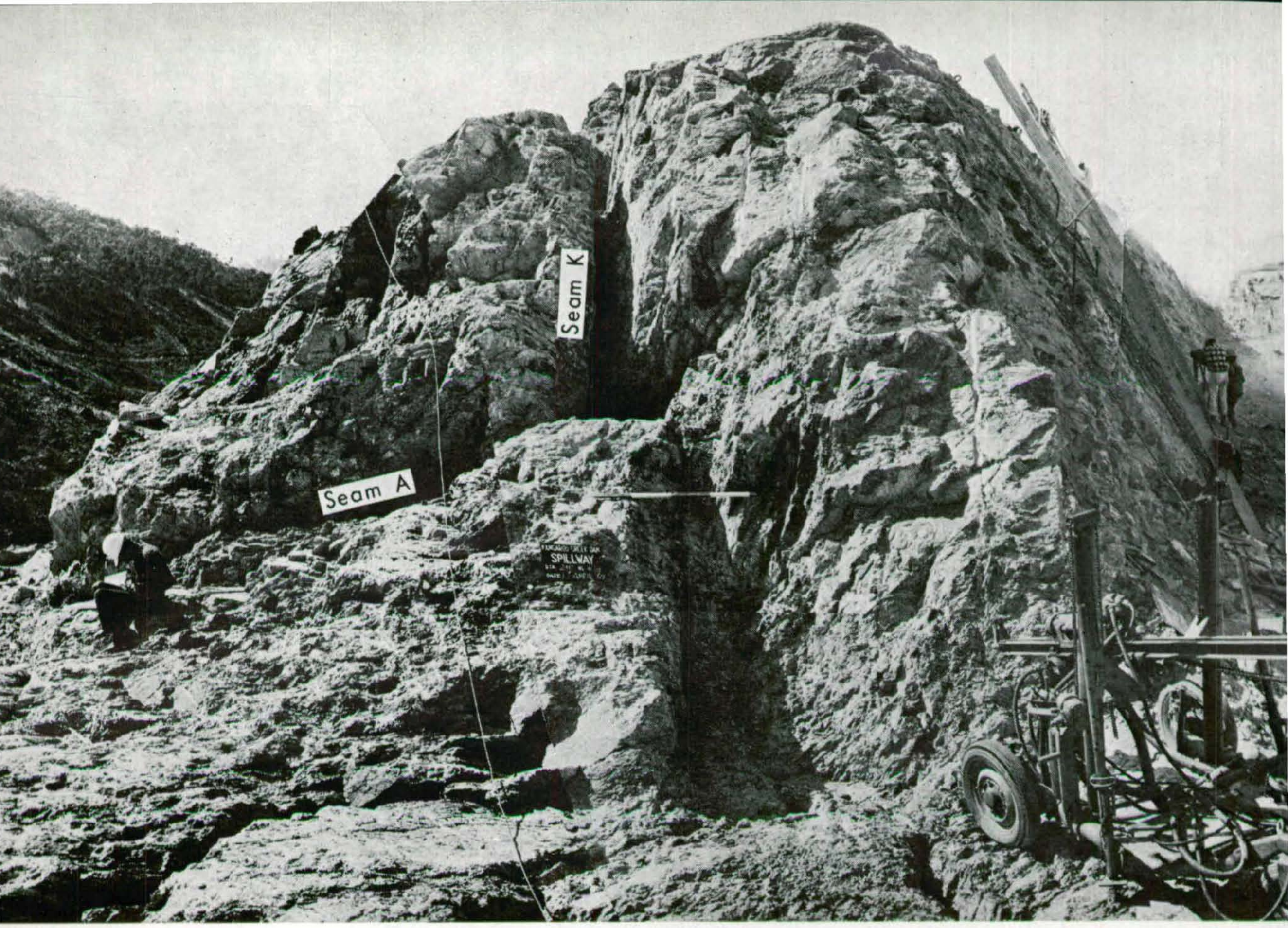


Fig. 46. Initial excavation for spillway block LE showing disturbed rock containing thick seams of infilled clay (seams K and A).

Excavation at the upstream end of the spillway channel proved to be deeper than anticipated due to the presence of a soil-filled gully, necessitating re-design of the lip in this area. Two possible design layouts (design 2 and design 3) were considered to avoid this area (Fig. 45).

1. Design 2 involved curving the channel into the hillside to avoid not only the gully in the corner, but also wedge mass No. 2.
2. Design 3 involved shortening the channel to avoid the gully, and excavation of part of wedge mass No. 2 so that the structure could be anchored to the relatively intact underlying rock (Fig. 76).

Design 3 was chosen for economic reasons. It involved considerably more excavation than the original design, with a corresponding increase in volume of concrete, as shown in Fig. 45. The aim of these modifications was to make the structure less dependent on the foundation rock for its support. To increase the resistance to uplift and to dynamic stresses during peak discharge, the re-design also incorporated a six by six feet pattern of vertical and sloping anchor bars and a drain of porous concrete on the channel side of the foundation.

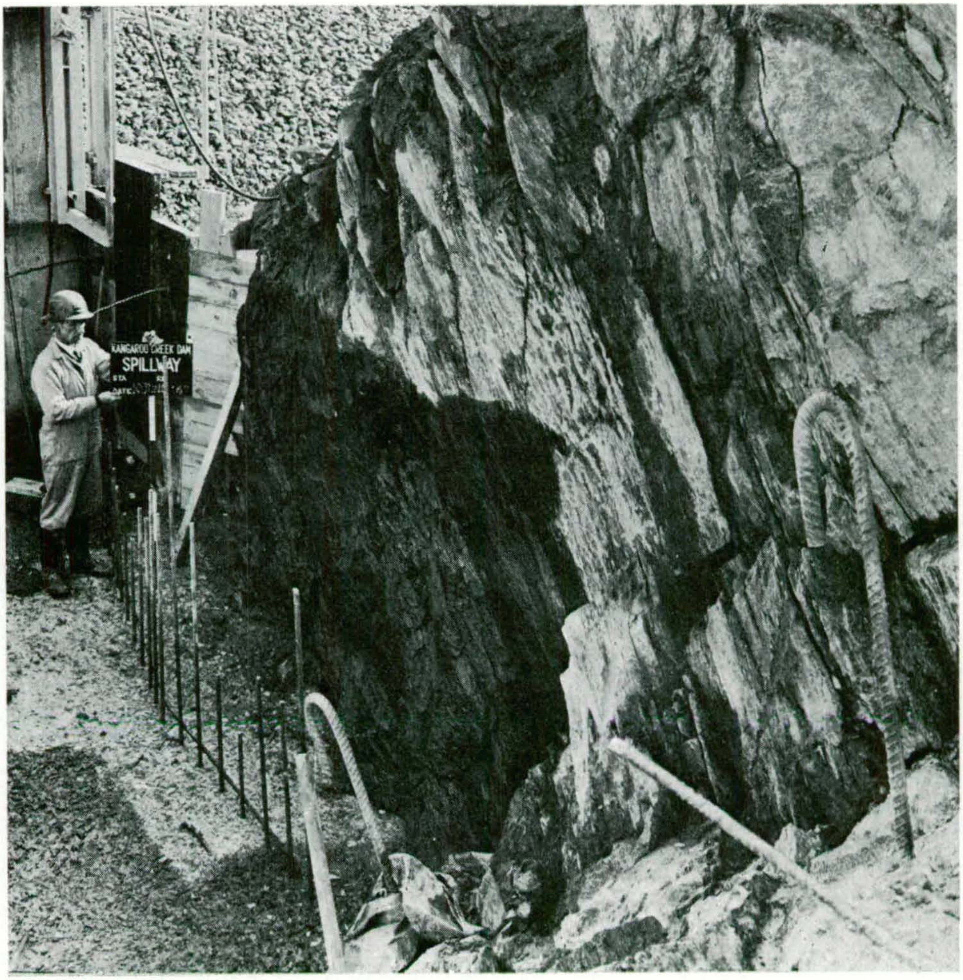


Fig. 47. Vertical batter of final excavation for spillway block E. The surface of the disturbed rock has been bitumen coated to eliminate bonding between rock and concrete.

As the rock in the northern wall of the excavation was of suspect stability, it was separated from the concrete by means of a non-adhesive bitumen coating. At the downstream (western) end of the overflow lip (block LE) the rock mass contained several thick infilled clay seams including seams A and K (Fig. 46) which contained up to 40 centimetres of highly plastic clay. Additional excavation amounting to 40 cubic yards was carried out to remove this disturbed rock down to RL870 (just below the level of the spillway channel floor) so that seams A and K were almost completely avoided (Fig. 39).

The foundation floor in block LD was sloped upwards at 45 degrees from RL870 to RL885. The vertical batter on the northern side of the excavation was coated with bitumen to render it non-adhesive (Fig. 47). In order to reduce uplift on the block from water passing through the grout curtain, a six by six feet pattern of anchor bars was installed and a row of drainage holes drilled from a gallery formed within the block.

Left abutment block

(Block LF)

The excavation was initially taken to the level of the adjacent spillway floor foundation, as specified. The rock mass at this level was highly disturbed, particularly towards the upstream end of the block where hand clean-up revealed numerous gaping joints, some open up to 50 millimetres (Figs. 48 and 49) and apparently extending to considerable depths. As this part of the spillway foundation provides a relatively short leakage path around the embankment, it was decided that for the foundations to be acceptable, they required the following treatments:—

1. Reinforcement of the rock mass.
2. Grouting, to reduce leakage past the embankment.
3. Drainage, to alleviate uplift forces which could be caused by water penetrating through the grout curtain.

To achieve these requirements the following procedure was adopted:—

1. Extensive hand barring and light blasting to remove the most severely disturbed and weathered rock.
2. In the upstream 20 feet all gaping joints were treated with a slurry of sand, cement and water.
3. The irregularities in the foundation were filled with concrete up to the general foundation level.
4. Anchor bars were installed in the upstream 30 feet of the foundation, on a six by six feet pattern, 15 feet into rock.
5. Blanket grouting of the upstream 30 feet of the foundation was carried out after the pouring of two six feet lifts of concrete. Holes were drilled to a depth of 30 feet and grouted in a single stage at 20 pounds per square inch. Primary holes were spaced in a 10 feet grid pattern, and secondary holes were also grouted in most places.
6. A drainage gallery was formed through the centre of the block to connect with the drain from the overflow lip.
7. In the upstream 35 feet of the foundation a single row of drainage holes was drilled from the drainage gallery, with holes six feet apart and extending 20 feet into the rock.

The locations of grout holes, anchor bars and drainage holes are shown on Fig. 48.

It was considered that the foundation for the downstream part of the block would be best left untreated, as the rock mass in this area was obviously free-draining due to the large number of gaping joints.

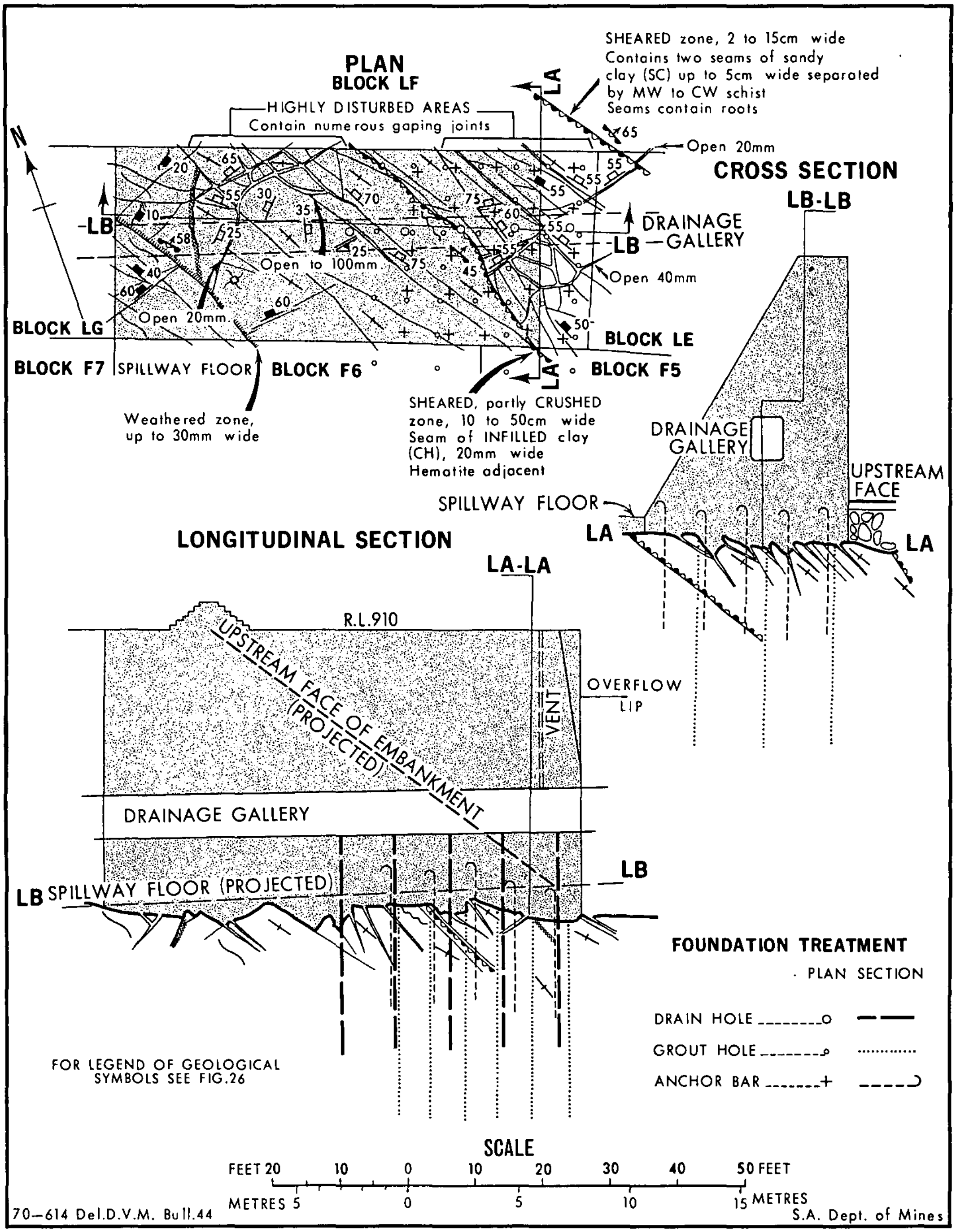


Fig. 48. Left abutment block; geological plan and sections.

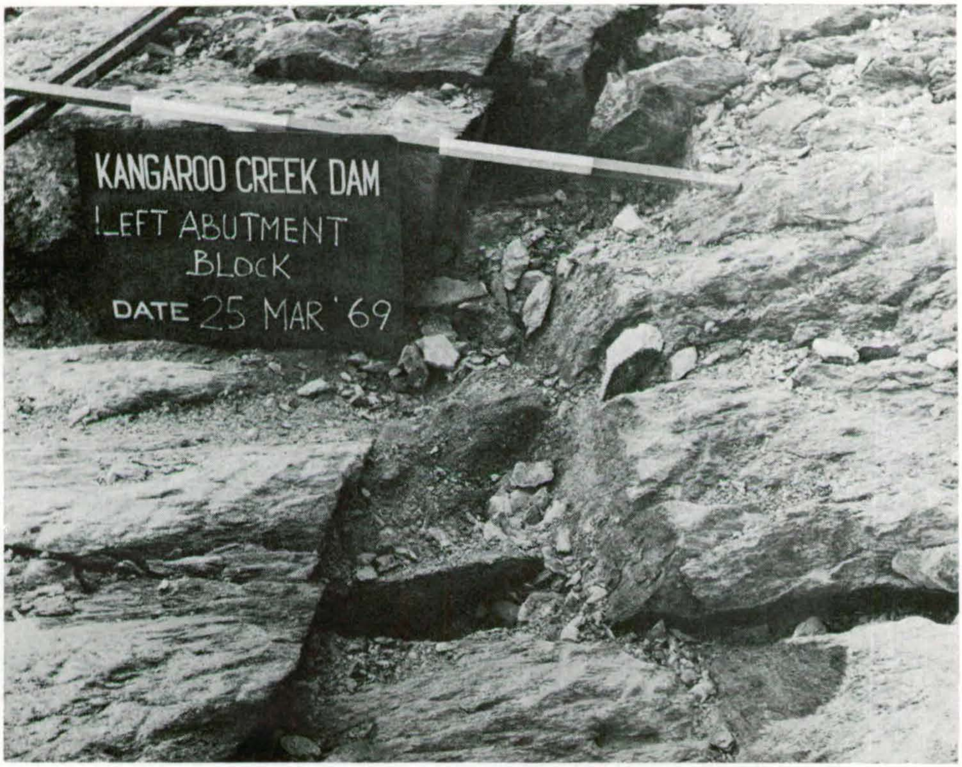


Fig. 49. Gaping joints exposed in foundation for left abutment block.

Block LH and Gully block

Excavations for these blocks included the removal of thicknesses up to 20 feet of soil materials which occurred in gullies formed by old wedge slides (Fig. 19). The extra excavation involved in removing this soil necessitated re-design of these blocks from thin concrete linings to self-supporting structures of gravity section (section D-D, Fig. 39). Some extra rock excavation was also necessary to provide suitably shaped bases to support the blocks.

Ski-jump

Considerable over-excavation occurred towards the top of the north wall due to the method of excavation. Further over-excavation occurred due to the removal by barring, of loose rock fragments which occurred towards the downstream end of this wall (Fig. 40). The resulting foundation was considered inadequate to support the lining as designed, and so the blocks in this area (LO, LP and LQ) were widened by five feet to make them more self-supporting (section B-B, Fig. 39).

A concrete key near the downstream edge of the ski-jump which was specified in the initial design, was not constructed as it was considered unwise to cause further disturbance to the rock mass in this area.

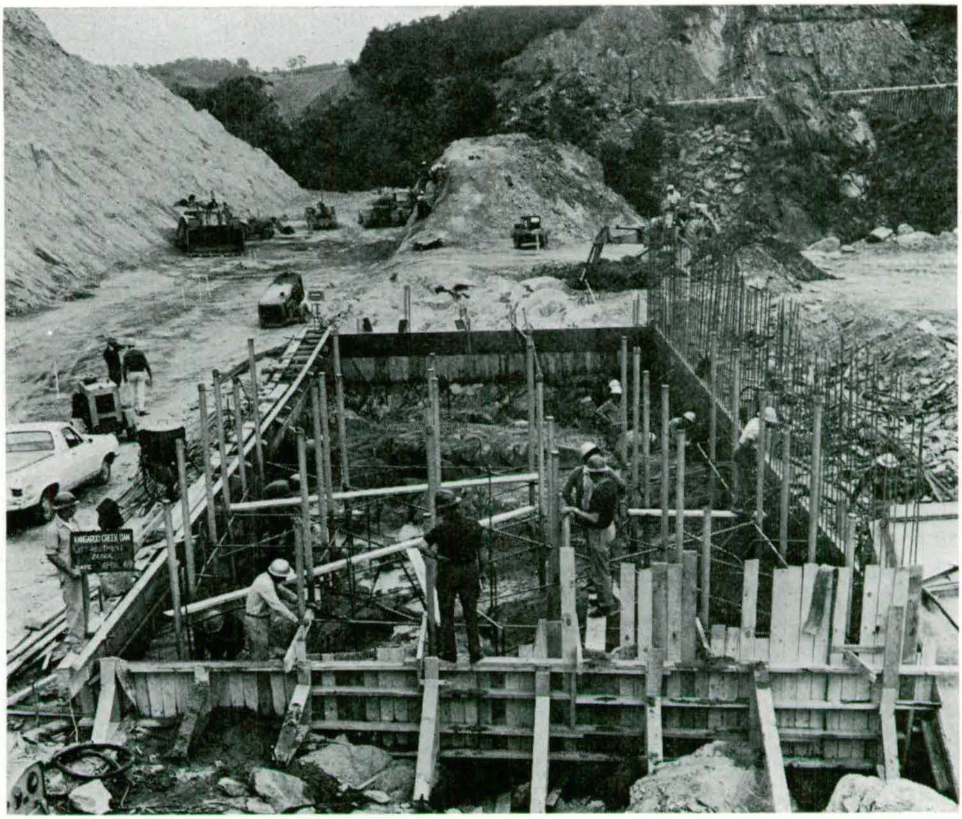


Fig. 50. Erection of form work, reinforcement and grout pipes for left abutment block, after treatment of open-jointed area at upstream end.

Intake structure

Design

The intake structure is a concrete tower inclined against the right bank, immediately upstream of the diversion tunnel (Fig. 4). The main advantage of the inclined tower design over the conventional vertical tower, is its resistance to damage by earthquakes. The dimensions are as follows:—

	Feet
Length (measured along slope)	270
Vertical height	210
Height of walls (measured normal to the floor)	17
Width of floor	16.5 to 17.8
Thickness of walls	2.5 and 3

	Degrees
Angle of inclination (floor)	46
Angle of inclination (walls)	90 (vertical)

Water from the reservoir may be selected at different levels by a series of six bellmouth inlets located in the roof of the structure and controlled by butterfly valves. The water passes from the inlets through a pipe (48 inches in diameter) within the structure, into the diversion tunnel.

The control room at the top of the structure is constructed on a horizontal foundation which originally formed part of the access road excavation. The inclined section of the structure is founded within a vertical-sided excavation up to 19 feet deep, with the floor sloping at 46 degrees. The lower part of the structure (base) is approximately cubic in shape with vertical walls, and is connected by PVC waterstop to the upstream portal of the diversion tunnel. It is situated on a horizontal foundation and rests against a vertical wall, 20 feet in height, on the north side.

Requirements of foundations

The intake structure is largely self-supporting so that the loads imposed on the foundations are quite small. The main requirement of the foundations is that they be stable under the range of conditions which might occur during the life of the structure.

In order to achieve foundations of suitable strength and stability to support the structure, it was necessary to remove:—

1. Spoil, topsoil, tree stumps, etc.
2. Completely and highly weathered rock except where it occurred in localized zones, or in the upper parts of the walls of the excavation.
3. Parts of the rock mass which had been severely disturbed by mechanical weathering processes and contained a significant proportion of gaping or soil-filled joints.

Method of excavation

The following excavation procedure was adopted:—

1. Soil and loose boulders were removed from the surface by dragline.
2. Vertical pre-split holes two feet apart were drilled for each wall, to within six inches of the floor level, and blasted.
3. For each 20 feet stage commencing at the top, pre-split holes one and a half or two feet apart, were drilled along the line of the proposed floor, and fired. The burden was fragmented with vertical holes or with a combination of "lifter" holes angled at 46 degrees and vertical holes.
4. The fragmented rock was removed by scraping with a dragline.
5. Loose material in the walls and floor was removed by hand barring.
6. Corners and high spots were trimmed with jack hammers.

Geological investigations

Regular inspections of the excavation by the resident geologist and feature engineer were conducted throughout the excavation period, to determine the necessity for extra excavation and the installation of rock bolts.

The excavation was logged in plan and section on a scale of one inch to 10 feet.

Foundation conditions

Most of the rock in the walls and floor of the excavation consists of gneiss, slightly weathered in the walls and fresh to slightly weathered in the floor. Several bands two to 12 feet wide, of slightly to moderately weathered granitic gneiss, trend across the excavation.

The rock mass adjacent to the excavation has been severely affected by mechanical weathering to depths up to 15 feet normal to the natural ground surface. Irregular sheet joints near-parallel to the ground surface, occur

Fig. 51. Sheet joints gaping open to 20 cm in upper part of intake structure excavation, downstream wall.

along most of the length of both walls (Fig. 51) and form the floor in many places where overbreak has occurred. These joints extend up to 40 feet, and are commonly partly open or filled with infilled soil materials for thicknesses of up to 15 centimetres. Many tectonic joints and several sheared zones also contain infilled clay, due to past downslope movement.

The rock mass beneath the floor of the structure is only slightly affected by mechanical weathering and the joints are either tightly closed, or contain less than 15 millimetres of infilled clay. In the excavations for the intake structure base and the control room, the joints are mainly tightly closed except for a few which are clay-coated.

Several seepages from slightly open seams and joints in the floor of the excavation occurred during excavation and gradually diminished in the following weeks. Further seepages and small flows occurred following periods of heavy rain.

Foundation treatments

Sheet joints in the walls and floor of the excavation dipping downslope, caused overbreak in many places. In order to restrict the overbreak and stabilize the rock adjacent to the excavation, a total of 18 rock bolts was installed.

In the lower part of the excavation, a sheet joint containing up to 10 centimetres of clay was exposed in a small area of overbreak three feet below the design floor level (Fig. 52) and apparently extending beneath the foundation on the upslope side. Jack hammer holes drilled in this area confirmed the extension of the seam for a distance of 25 feet upslope. The rock above the seam amounting to 50 cubic yards, was then removed.

Stability of foundations

It is considered that the floor of the excavation is below the zone of severe mechanical weathering and that sheet joints beneath the floor are mainly tightly closed and relatively restricted in extent, and therefore not likely to cause instability. Several sheet joints which occur in the vertical wall of the excavation for the base of the structure, extend less than 20 feet along the batter parallel to the river direction, and contain less than 10 millimetres of infilled soil. This wall has been reinforced by the installation of 22 rock bolts.

Sheet joints in the walls of the excavation caused small stability problems during excavation. The walls were reinforced by the installation of 18 rock bolts.

The intake structure itself provides additional support for loose blocks in the walls.

A sheared partly crushed zone which outcrops downstream of, and dips beneath the intake structure, was considered as a possible source of major instability in combination with Set (4) joints. However the Set (4) joints exposed in the excavation extend less than 10 feet and hence do not constitute a danger.

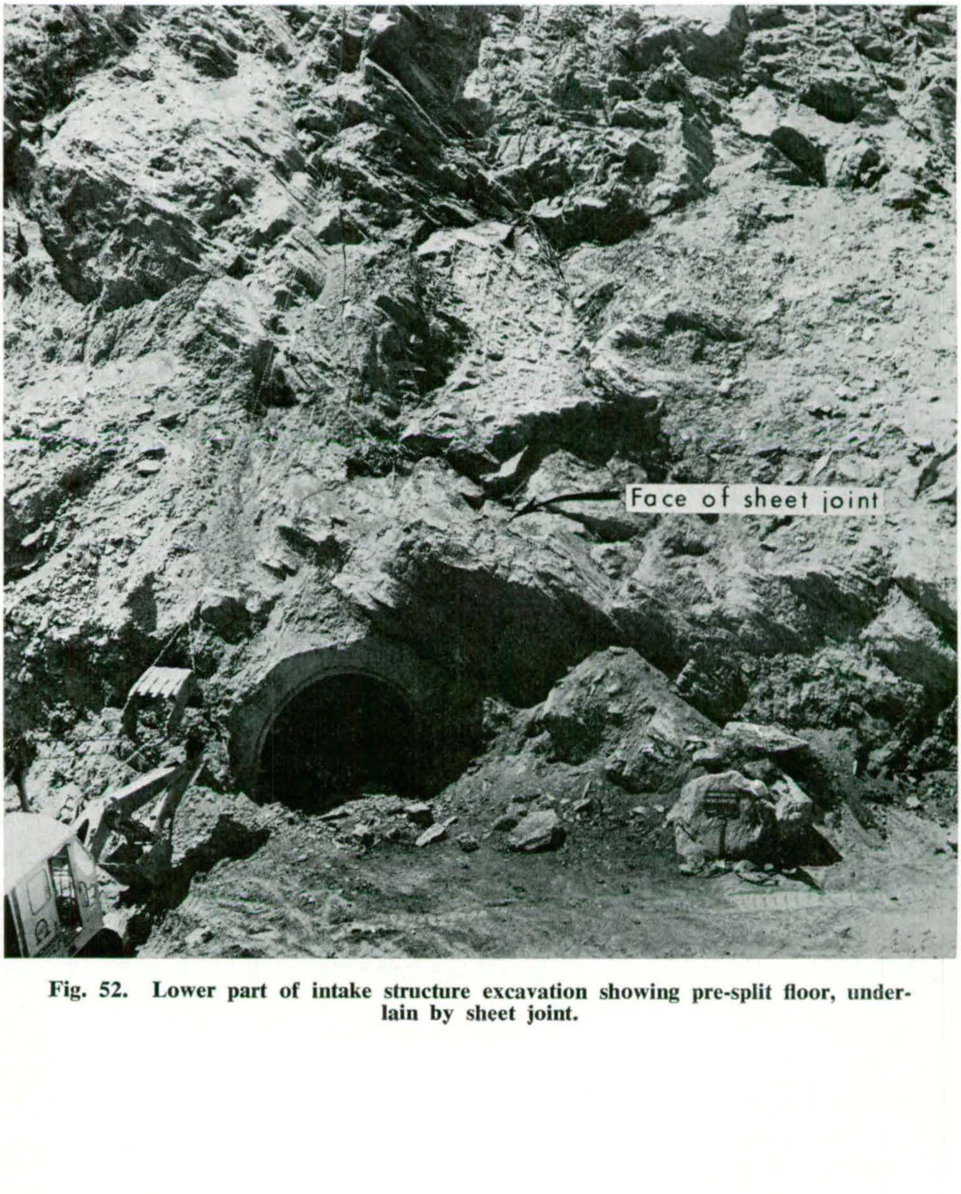


Fig. 52. Lower part of intake structure excavation showing pre-split floor, underlain by sheet joint.

QUARRIES

Spillway excavation

Design

The spillway excavation consists of a series of five benches and a channel which terminates in a ski-jump at the downstream end. The excavation as constructed is shown in plan and section on Fig. 53. The dimensions are as follows:—

	Feet
Length (horizontal distance)	900
Maximum height of upslope face in elevation	210
Maximum plan width of excavation	360
Width of channel floor	42 to 44
Vertical distance between berms	50
Width of berms	45
Maximum depth on channel centre line	80
	Degrees
Cutting angle, upslope (south) batters (between berms)	45
Overall angle top to bottom (including berms)	40
Cutting angle, downslope (north) batter	60
Slope of channel floor and berms	2.3 (1 on 24)
Total volume of excavation	360,000 cu yds

Method of excavation

The first excavations in the spillway were haul roads cut at three different levels to provide access to quarry No. 1. The uppermost road (RL1020 to RL1000) was mainly excavated by ripping and scraping using Caterpillar D8 bulldozers, with blasting in some sections. The lower haul roads (RL970 to 950 and RL910 to 890) were excavated mainly by drilling and blasting from a face advancing from upstream to downstream.

The excavation of haul roads was followed by stripping of surface soil and loose rock fragments by scraping with Caterpillar D8 bulldozers. The remainder of the spillway was then excavated berm by berm by drilling and blasting, commencing from the top (berm 1). Each berm was excavated progressively in stages from upstream to downstream.

Drilling patterns. The drill holes in the pre-split batters were nominally three feet apart for batters 1 to 5 and two feet apart for the channel walls.

The pattern of burden holes ranged from eight by six feet to 10 by 10 feet, the most common spacing used being nine by seven feet. Burden holes were all vertical except in the channel where a V-shaped pattern was adopted. All holes were three inches in diameter and were drilled using Air-trac percussion rigs.

Explosives. Powder factors ranged from 0.45 pound per cubic yard to 0.8 pound per cubic yard, with 0.65 pound per cubic yard the most commonly used charge. The main explosive used was ANFO (Ammonium nitrate/fuel oil mixture) in conjunction with cordtex relays and electric detonators. Gelignite (Anzite yellow) was also commonly used.

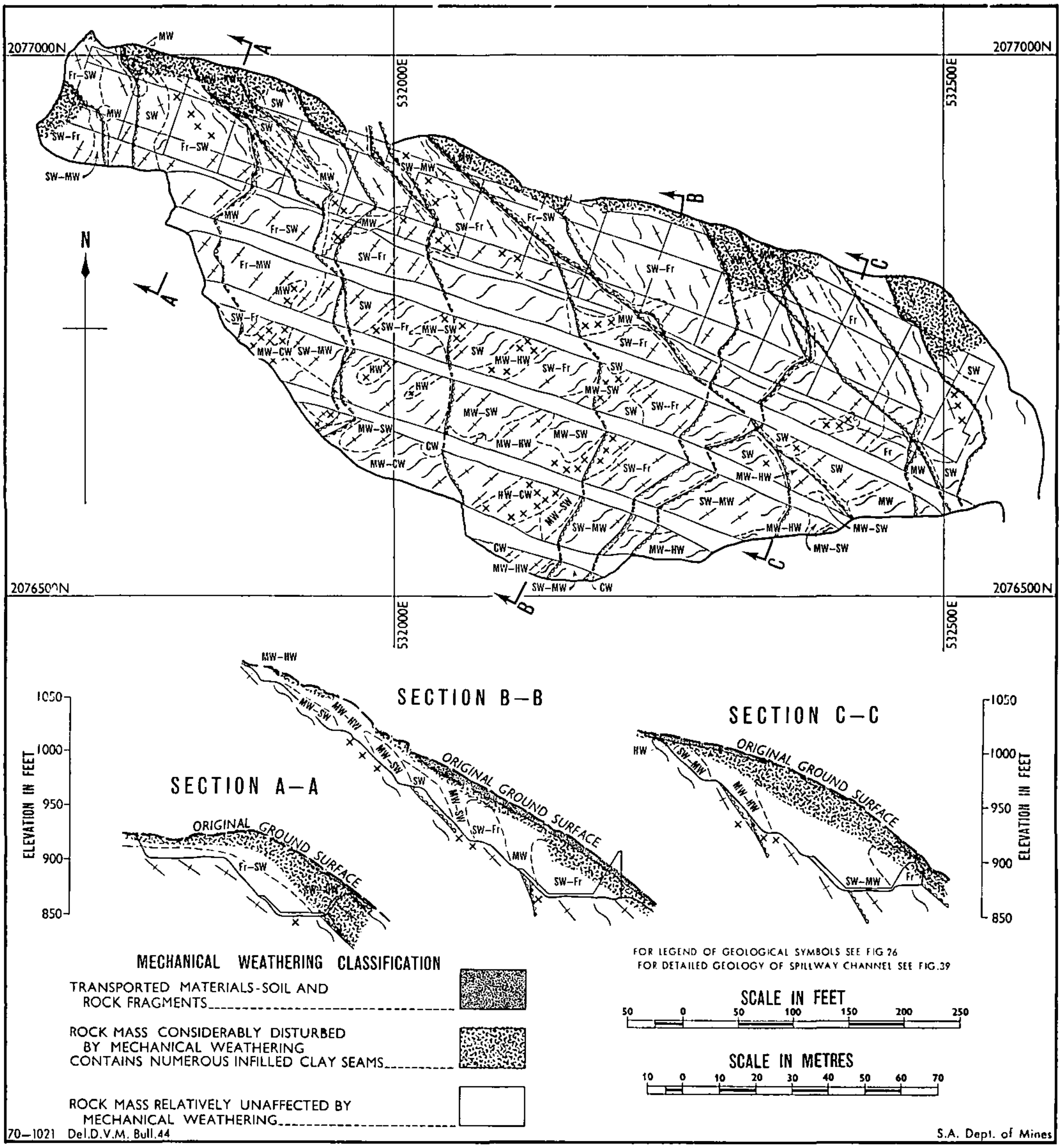


Fig. 53. Spillway excavation, rock types and weathering; plan and sections.

The pre-split was achieved by the attachment of half or quarter plugs of gelignite to strings of cordtex at spacings of one to two feet, with a powder factor of 0.8 ounce per square foot.

Secondary blasting. Most blasts produced considerable amounts of materials too large to be handled efficiently by the excavating machinery. This oversize material usually came from near the free face of the bench being excavated, and was usually located towards the outside edge of the pile of blasted rock. Where oversize rock formed a small proportion of the volume (less than 10 per cent) it was usually side-cast, and secondary blasting was carried out after completion of loading of the rest of the pile. Where it formed more than 10 per cent of the volume, the oversize rock was re-drilled and blasted before loading of the pile commenced.

Loading. The blasted material was loaded by means of a four and a half cubic yards capacity shovel or front end loader of five and a half cubic yards capacity, into rear dump trucks with capacities of 20 or 27 cubic yards.

Trimming of batters. The designed batter angle of 45 degrees proved too steep to allow removal of loose material by bulldozers pushing directly downslope or by loading equipment working upslope. However it was not sufficiently steep for the material to roll down due to gravity. The procedure adopted to form the batters consisted of trimming with the side of a bulldozer blade. For this purpose it was necessary for the bulldozer to work back and forwards along a receding ramp located against the batter. This procedure resulted in considerable breakdown of the rock as the ramp receded.

Geology of excavation

The original valley surface in the spillway area consisted of three steep rocky ridges up to 100 feet wide separated by soil-filled gullies up to 200 feet wide. Excavation showed that the soil-filled gullies contained up to 20 feet of transported material underlain by mainly intact schist and gneiss, and that the rocky ridges consisted mainly of disturbed schistose rock materials. In the excavation as a whole (Fig. 53), schist, gneiss and granitic gneiss occurred in the proportions of 50, 35 and 15 per cent respectively. Within each particular rock type the severity of weathering effects increased upwards, towards the crest of the hill and outwards, towards the valley (Fig. 53). The rock in benches 5 and 6 was predominantly fresh.

The zone in which there has been considerable disturbance by mechanical weathering (Fig. 53) ranged up to 50 feet in thickness and comprised almost 40 per cent of the volume of the spillway. This zone contained numerous infilled clay seams up to 40 centimetres wide, along joints and sheared zones. In the remainder of the excavation including the upslope batters and the channel floor, the rock mass had been relatively unaffected by mechanical weathering. Infilled seams in this zone were rare and mainly less than two centimetres in width.

Fragmentation

The grading of the rock produced by blasting varied considerably due to wide differences in geological conditions and blasting technique between one part of the excavation and another. Except for five to 20 per cent of material which was too large to be handled by the excavation machinery, and which was reduced in size by secondary blasting, the gradings achieved by the primary blasting were generally within the limits required for the rockfill (Fig. 57). The percentage of minus one inch material at this stage in the excavation/placement cycle ranged from an estimated 25 per cent in areas where the surface soil had been incompletely removed, to 10 per cent in the areas of comparatively unweathered rock. The predominant fragment size ranged from 60 by 30 by 10 centimetres, up to 120 by 80 by 30 centimetres.

During handling, the grading of the rock changed considerably due to breakdown, particularly where the rock substance was moderately or highly weathered. In many places this breakdown was sufficient to cause rejection of the rock.

The shape of the rock fragments was related to the rock type and in particular to the degree of anisotropy. Blasting of the schistose rocks produced flaky fragments (Fig. 54) with the maximum dimension commonly three to six times that of the smallest. The fragments of gneiss were more bulky (Fig. 55) with the ratio of largest to smallest dimensions usually less than three.

Selection of embankment materials

The main criteria used to select rockfill for zones 2, 3 and 4 of the embankment are described on pages 119 to 125. Material unsuitable for the embankment was hauled to the disposal area. As far as possible once construction of the embankment had commenced, material for the embankment was produced from a single face as desired. However, owing to the difficulty of producing material suitable for zone 2, a particular area of the spillway excavation away from the main face was set aside for this purpose.

Fig. 54. Face of spillway excavation, bench 4. Primary blasting and some secondary blasting (foreground) have produced highly flaky fragments of the weak schist.

Fig. 55. Shovel loading bulky gneiss from quarry No. 1, bench 4. Fines occur mainly in material pushed down by D8 bulldozer from upper parts of the bench.

Large quantities of sound rock were sent to disposal due to contamination with fines derived from surface soil and highly to completely weathered zones. Control of this wastage of rock proved to be the most difficult aspect of the excavation.

In some places where the material was exceptionally sound it was mixed at the embankment with contaminated material that would otherwise have been directed to disposal.

Stability of batters

The overall angle of the upslope wall of the spillway cutting was designed to minimize the failure of the wedges of rock bounded by Set (1) and Set (2) weaknesses. Fig. 56 shows that although some wedges are free to slide with slopes of 45 degrees, the size of the blocks which could fail is limited by the presence of berms. Two wedges of rock, each less than two cubic yards in volume, failed in this way but the loose material was retained on the berm below.

No other wedges or blocks with freedom to slide into the excavation were evident from detailed examination of the batters, and no special treatments were considered necessary to improve the stability.

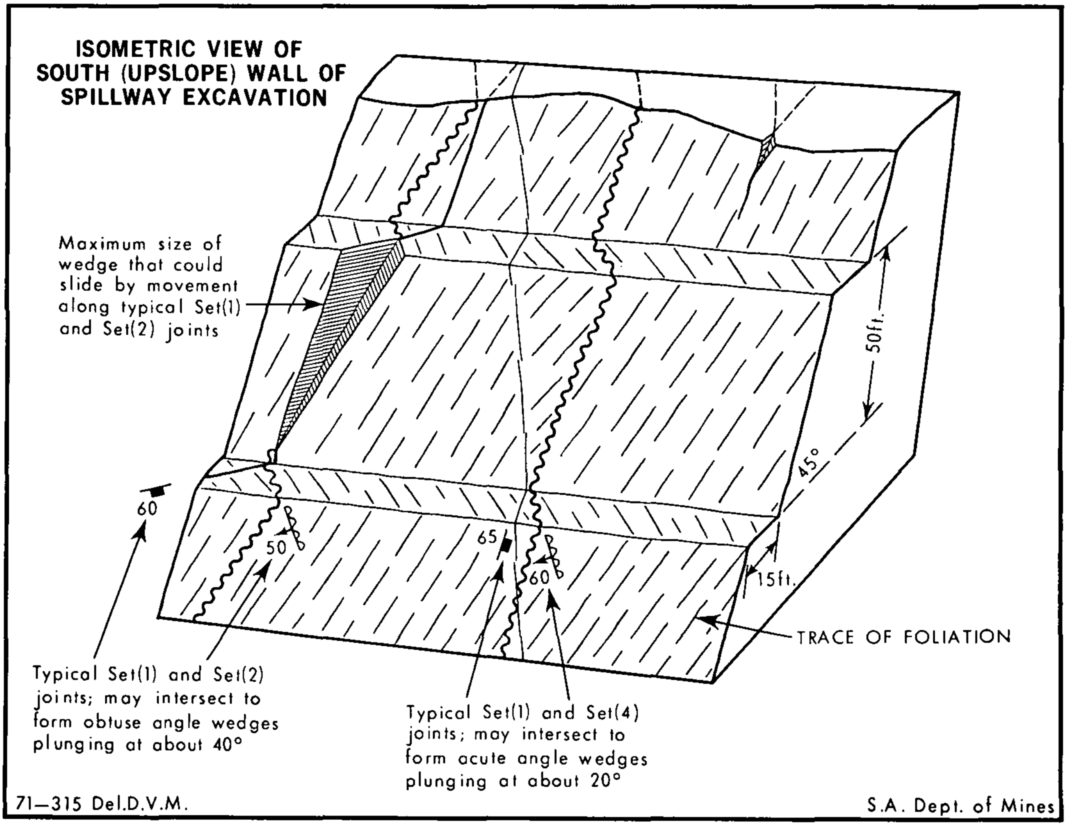


Fig. 56. Diagram showing stability of spillway cut.

Design changes during construction

Although the pre-split technique resulted in satisfactory batters from the viewpoints of stability and appearance, the batters tended to deviate from the specified lines. Most of the irregularity was due to the wandering of the 70 feet long drill holes. In order to achieve maximum accuracy of the channel wall foundations it was decided to shorten the lengths of pre-split holes by incorporating an extra berm (berm 5) five feet wide, located two feet above the level of the channel concrete. To accommodate the extra berm the angle of batter 4 was steepened to 50 degrees.

Quarry No. 1

Design

The quarry situated immediately downstream of the spillway consists of a series of six, three-sided benches (Fig. 4), extending to the level of the Old Gorge Road. The dimensions are as follows:—

	Feet
Vertical height of benches	50 (benches 1 to 4)
	70 (benches 5 and 6)
Width of berms	20 (berms 1 to 3)
	40 (berms 4 and 5)
	Degrees
Cutting angle (between berms) .	76 (1 on 4) for hillside wall
	63 (1 on 2) for downstream wall
Overall cutting angle	63 (1 on 2)
Total volume of excavation	115,000 cu yds

Method of excavation

Excavation commenced with stripping by Caterpillar D8 bulldozers of loose soil materials in the upper part of the quarry area (benches 1 and 2). Access to this area was through the spillway cut. Stripping of surface soil below the level of berm 2 was carried out from above using a dragline. Benches 1, 2, 3 and 4 were then excavated progressively in stages from upstream to downstream, and the material was hauled *via* the three haul roads constructed as the first stage of excavation for the spillway.

The material from benches 5 and 6 was loaded from the Old Gorge Road and hauled to the embankment *via* the downstream ramp. Caterpillar D8 bulldozers were used to push the blasted material from bench 5 onto the road below. The steep batters were "self mucking" and did not require trimming by bulldozers.

Drilling. The drill holes in the pre-split line were three inches in diameter and spaced four feet apart. The pattern of burden holes was mainly 9 by 7 feet, and holes were sloped parallel to the pre-split holes at 76 degrees and drilled to the level of the berm.

Explosives. In the pre-split holes single or half plugs of gelignite were attached to strings of cordtex, with a powder factor of approximately one ounce per square foot.

In the burden holes ANFO was used and the powder factor ranged from 0.6 pound per cubic yard in the upper benches to 1.2 pounds per cubic yard in the lower benches.

Secondary blasting and loading. As for the spillway excavation (see pages 104-105).

Geology of excavation

The original ground surface in the quarry area was formed by a prominent steep ridge of jagged rock ranging in width from 30 feet at the top to 80 feet at the bottom. The ridge was flanked on the upstream side by a shallow rocky gully containing a small creek and on the downstream side by a prominent soil-filled depression. The quarry was located to include as much as possible of the ridge, and a minimum of the soil-filled depression which drilling had shown to be underlain by weathered, feldspathic gneiss.

The rock from the excavation consisted of gneiss and schist in the proportions 60 and 40 per cent respectively. The degree of weathering increased with height above the valley floor, ranging from moderately and highly weathered in bench 1 to fresh and slightly weathered in benches 5 and 6. It also appeared to be related to the feldspar content of the rock which showed vertical as well as lateral variation, due to a complex pattern of metamorphic differentiation along both the S2 and S1 foliation directions.

Soil was mainly less than five feet thick, and unlike the rock mass in the spillway area, mechanical weathering effects in the quarry area were insignificant at depths of more than 15 feet below natural surface.

Fragmentation

The grading of the blasted rock ranged within the same limits as the rock from the spillway excavation. However the quarry rock required much less handling, due to the much steeper batters, and in particular there was little need for bulldozing, the process responsible for the most severe breakdown. For this reason and also because of the predominantly higher strength and durability of the quarry rock, the rockfill delivered to the embankment contained considerably less fine material than the rock from the spillway.

Stability of batters

In designing the quarry it was not feasible to adopt batter angles which would completely exclude the possibility of wedge or block slides. However the exploration had not revealed any weaknesses likely to contribute to major instability and it was considered that minor instability could be handled during construction.

The quarry batters contain relatively few geological defects and most of the prominent ones dip into the slope. Set (2) and (4) joints which could combine with Set (1) seams and joints to give wedges with freedom to slide, mainly extend less than 10 feet.

The only slides to occur were of wedge blocks up to five cubic yards at the tops of the batters. Several of these blocks were removed by hand barring after removal of materials from each bench.

Quarry No. 2

During construction it became apparent that the spillway and quarry No. 1 excavations as designed, contained insufficient material to complete the embankment, and that up to 60,000 cubic yards of additional rock would be required. A rapid exploration programme consisting of surface mapping, four bulldozer trenches and eight Air-trac (percussion) drill holes, was carried out at a site located 600 feet northeast of the disposal area (Fig. 5). The exploration revealed a sequence of dolomites with alternating strong and weak bands up to 20 feet thick, dipping east into the hillside at 20 to 40 degrees. The weak bands although similar in composition to the strong bands appeared to have been far more severely affected by chemical weathering. In many places a "skin" up to four feet thick of strong case-hardened dolomite obscured the nature of the underlying material. Results of the exploration suggested that with considerable selection and rejection, the required quantities of rock would be available.

Petrographic examination of the dolomite revealed that it consisted mainly of microcrystalline dolomite carbonate with 15 to 30 per cent of sand-sized quartz and some feldspar (see Appendix 2). Weathering has resulted in the breakdown and leaching of the carbonate minerals.

Sulphate soundness tests were carried out to assess the durability of the dolomite materials. The results indicated that the rock, even in a highly weathered state, is resistant to sulphate attack (see Table 8).

Geology of excavation

Stripping of the area resulted in the removal of an average of six feet of soil materials, with thicknesses up to 25 feet in places. The quarry was excavated in a single bench up to a height of 65 feet and with batters sloping at approximately 76 degrees or 1 on 0.25.

Excavation showed that in addition to the weak bands orientated along bedding, there were numerous patches of soft weathered materials, apparently unrelated to geological structure. The places where these patches intersected the surface corresponded to the deep pockets of materials removed during stripping. The degree of weathering showed only slight decrease with depth below the ground surface.

The dolomite material was closely jointed, with unit blocks mainly bulky in shape and ranging from five by five to 50 by 50 centimetres in size. The case-hardened surface was relatively free from joints.

Fragmentation

The stronger bands of rock were reduced by blasting to unit block sizes and retained these sizes during handling. Blasting and handling of the weak rock caused considerable breakdown, reducing much of the rock to sand and gravel-sized fragments. Excavation of the case-hardened rock resulted in blocks up to 200 by 100 by 100 centimetres in size.

Little attempt was made during loading to separate the finer, weak dolomite from the strong rock, as the materials were intimately mixed together. The alternative approach was to excavate the areas containing strong rock in preference to the weak areas. This also proved difficult due to the complex weathering pattern.

Zone 1 Quarry

The original specifications stipulated that rock for the embankment should be obtained from the spillway and quarry No. 1 excavations. However the obvious difficulties involved in producing rock of zone 1 grading (see Appendix 6) from these areas influenced the contractor to search for an alternative, more practical source.

A reconnaissance geological survey of rock within two miles of the site carried out on behalf of the contractor, indicated a promising occurrence of quartz gneiss 1.2 miles east of the dam site (Fig. 5).

A programme of trenching and drilling was suggested to prove the suitability and quantities of materials, however the contractor decided to explore the proposed quarry area by stripping of surface soil and excavation of a trial quarry. The trial quarry produced closely jointed, weathered quartz gneiss which after screening on a two inch screen, fitted the required grading envelope for zone 1. Although weathered, this rock was strong to very strong due to the interlocking of granular quartz and feldspar (Appendix 2), and breakdown during handling proved to be insignificant.

Geology of excavation

The overburden of surface soil averaged four feet but was up to six feet thick, and consisted mainly of brown silty clay. Excavation was carried out by extending the trial quarry southwest into the hill. The rock proved to be progressively less weathered away from the surface.

The rock mass is closely jointed with at least four joint sets forming unit blocks ranging from three by five by five centimetres to 50 by 30 by 30 centimetres in size, and mainly bulky in shape. In the mechanically weathered zone within 25 feet of the surface, most of these joints were cohesionless, however at depths greater than 30 feet many of the joints were strongly cemented.

Fragmentation

In the outer zone of mechanically weathered rock most of the unit blocks were separated during blasting and handling so that the material produced consisted mainly of joint blocks five by five by three centimetres to 50 by 30 by 30 centimetres in size.

In the zone of rock relatively unaffected by mechanical weathering, the blasting and handling processes were insufficiently severe to rupture the cementing bond between many joint blocks, and blocks up to two by one by one metres were produced. Attempts were made to overcome this problem by using higher powder factors, however much of the blasted material exceeded the specified grading. The oversized material was further broken down at the quarry site by the impact of a crane-mounted ball weight, and on the embankment face by sledge hammers.

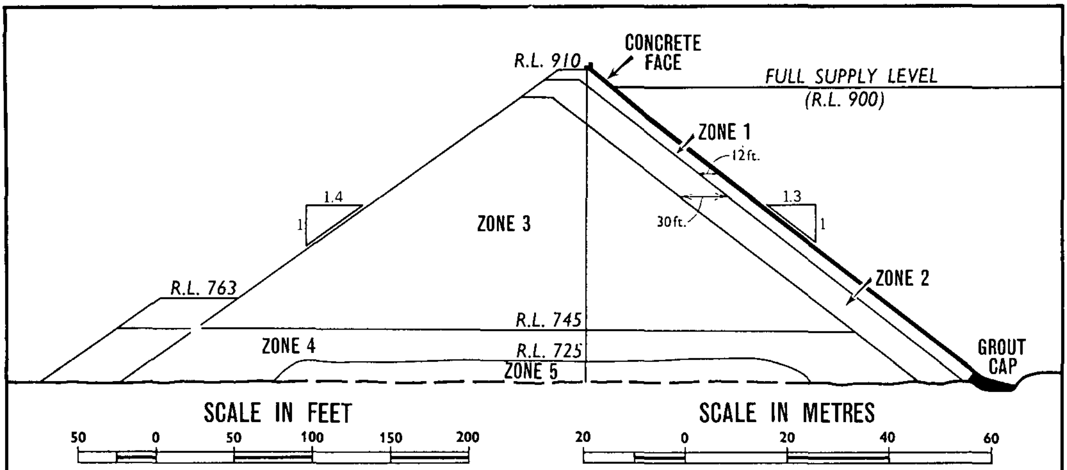
EMBANKMENT

Design

The embankment is a decked rockfill structure of trapezoidal cross-section (Fig. 57), 200 feet high and with a crest length of 400 feet. The crest width is 25 feet and the width at the base is 600 feet. A 50 feet wide berm on the downstream side accommodates the screening chamber. The upstream face slopes at 1.3 to 1 and the downstream face at 1.4 to 1. A concrete membrane consisting of 40 feet wide slabs ranging in thickness from one foot at the crest to three feet at the toe, rests on the upstream face. The total volume of material in the embankment is 435,000 cubic yards.

Internally the embankment consists of five zones as shown in cross-section on Fig. 57. The zones consist of rockfill placed in layers, sluiced and compacted. The main statistics and requirements of each layer are tabulated in Fig. 57.

Zones 1, 2 and 4 are filter zones, designed to drain any leakage which occurs through the face and abutments. Zone 1 was manufactured to a specified grading envelope (see Appendix 6) to ensure adequate permeability and also to ensure that the face when rolled, provides a suitable foundation for the placement of the upstream face. Zone 2 also acts as a "buffer zone" to eliminate contamination of the clean zone 1 material by fines from zone 3.



ZONE	LAYER THICKNESS (ft.)	MAXIMUM FRAGMENT SIZE (ft.)[1]	AMOUNT OF FINES[2]		COMPACTED DENSITY—[3] Wet (lbs./cub. ft.)	VOLUME IN DAM (cub. yds.)	REQUIREMENTS
			Specified	Actual			
1	2	1	0	12%–20%	116–124	24,800	– High permeability in vertical and horizontal directions so that all water leaking through the face and abutments is readily drained – Voids small to minimise loss of concrete
2	3	3	20% MAXIMUM	20%–50%	140–160	54,100	– Moderate permeability so that Leakage water drains readily from Zone 1 to Zone 4
3	3	3	30% MAXIMUM	20%–50%	143–169	311,300	– Resistance to settlement – Some horizontal permeability to drain water from abutments and excess from Zone 2
4	4	4	20% MAXIMUM	10%–30%	130–160	52,900	– Resistance to settlement. – High permeability in all directions to ensure unimpeded drainage of water out of the embankment.
5		NO LIMIT					– Freedom from all large areas of organic matter. – Moderate permeability.

1 — Size limit applies only to smallest dimension

2 — The term "fines" refers to the —1 inch fraction

3 — For Zones 1 and 3 the figures are based mainly on Laboratory analysis of samples obtained in ring density tests. For Zones 2 and 4 the figures are based on visual estimation.

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Fig. 57. Embankment zones; main statistics and requirements for each layer are tabulated.

Method of construction

Wherever possible, embankment layers were placed systematically from downstream to upstream. The hauling trucks dumped the fill material in heaps at the edge of the layer and the material was then spread by several passes of a Caterpillar D8 bulldozer.

During dumping and prior to spreading, the material was thoroughly sluiced by a monitor mounted on a Caterpillar D7 bulldozer. Sluicing water was applied to each load for a minimum of three minutes to ensure that each cubic yard of rock received a total of 180 gallons of water. Some of the sluicing water flowed across the surface and down the embankment face, and other water percolated through the fill. The water was retained by the downstream measuring weir and continually re-circulated.

Each layer was compacted by four passes of a 10 ton vibrating roller towed behind a Caterpillar D7 or D8 bulldozer. Access to the lower 100 feet of the embankment was by means of external ramps, the downstream ramp to RL768 and the upstream ramp to RL800. Above RL800 an internal ramp was placed extending into the spillway channel at RL860, downstream of the left abutment block. Fill material brought in *via* the ramp was used to complete the embankment to this level. Above RL860 another internal ramp was constructed rising to crest level against the right abutment block, and the embankment was finally completed with rock transported *via* the right abutment access road.

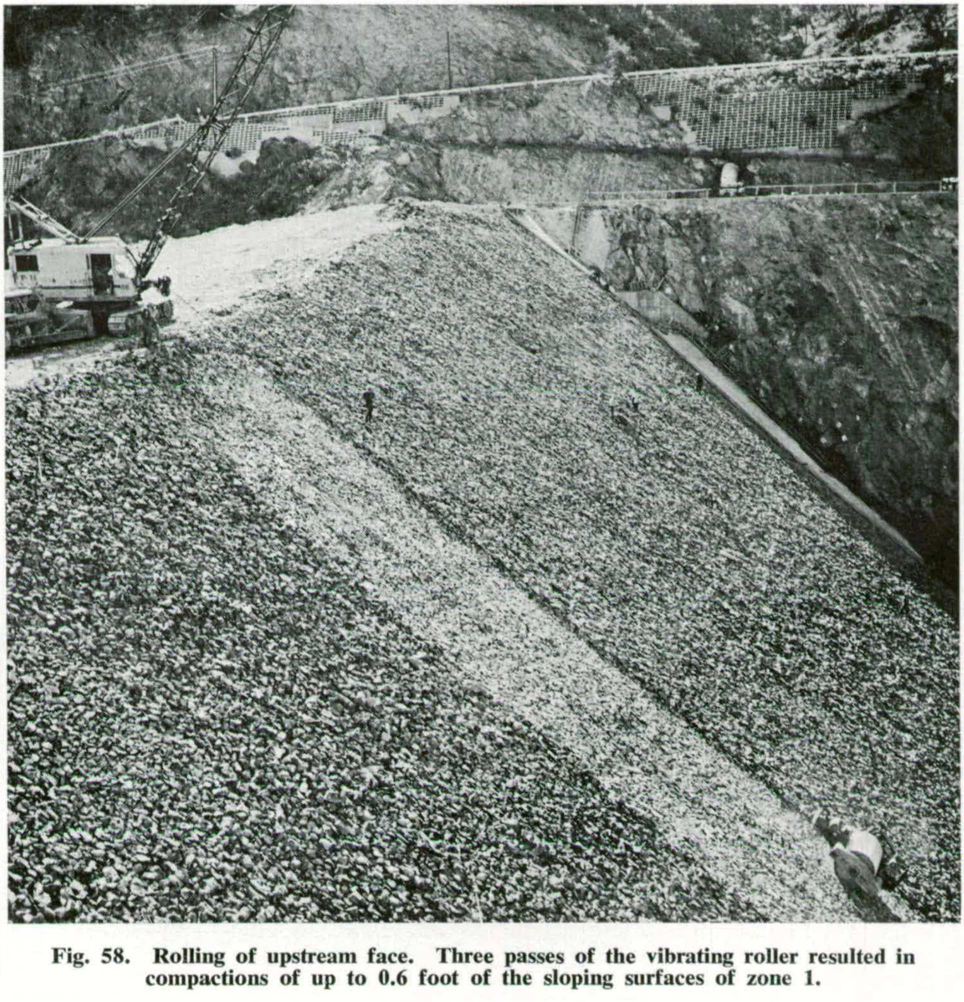


Fig. 58. Rolling of upstream face. Three passes of the vibrating roller resulted in compactions of up to 0.6 foot of the sloping surfaces of zone 1.

After completion of the embankment, the face of zone 1 was compacted by three passes of the vibrating roller (Fig. 58). The upstream face was then placed on this surface in 40 feet wide slabs, connected by PVC waterstop. The lowermost part of each slab was placed and surfaced by hand. The rest of the slab was top-formed by a moving screed (Fig. 59).

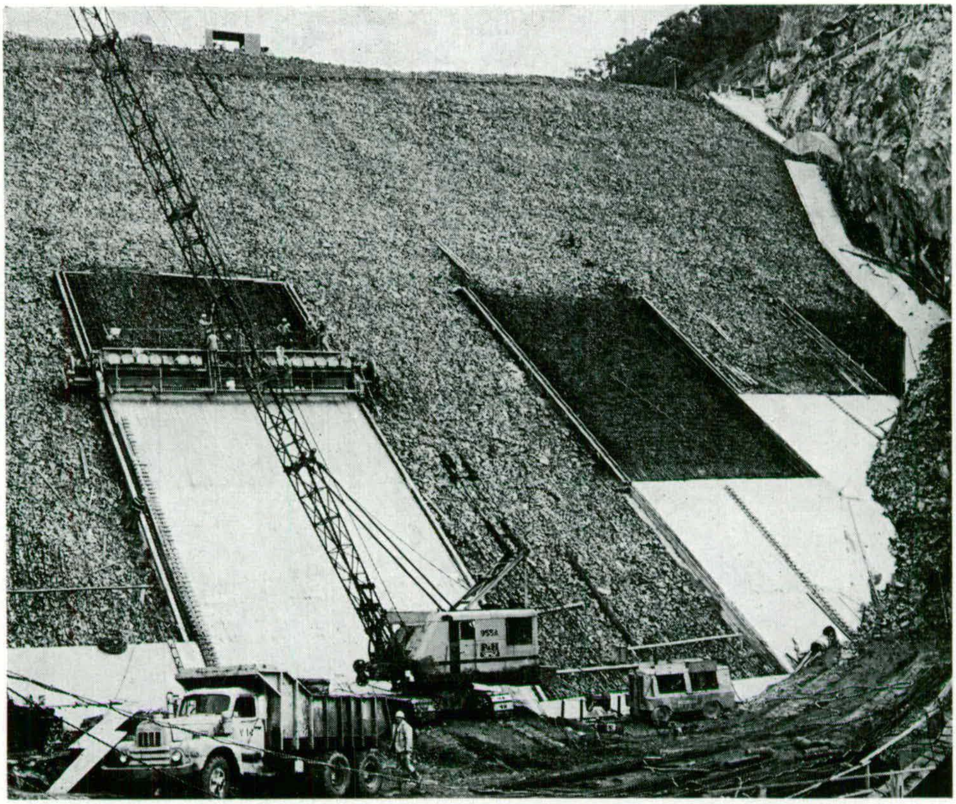


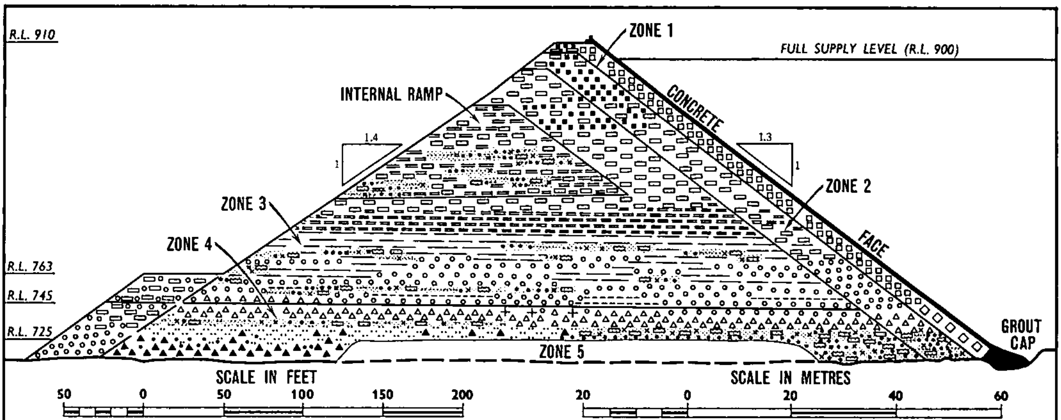
Fig. 59. Placing of concrete membrane on the upstream face of the embankment. The triangular corner piece of each slab was surfaced by hand and the remainder top-formed by a moving screed.

Sources of rockfill

The main rock materials and their distribution within the embankment are shown on Fig. 60.

Excavation of access roads through the spillway and excavation of quarry No. 1, benches 1 and 2, and part of bench 3 were carried out prior to commencement of the embankment. Usable materials from these excavations were classified at the face as either suitable for zone 3 or suitable for zone 4, and accordingly removed to zone 3 stockpile or zone 4 stockpile. A stockpile was also made of material from the downstream access road. Once placing of rockfill had commenced, all material classified as suitable was transported directly to the embankment. Placing of the stockpiled material occurred mainly during delays in production of quarry and spillway material.

Zone 4 was constructed mainly of rock from benches 3 and 4 of quarry No. 1 and from road spoil (contract variation No. 7). Production of rockfill materials on a large scale from the spillway did not occur until bench 3 was reached at which stage the embankment had reached RL772, and quarry No. 1 which was 90 per cent complete, was temporarily abandoned.



KEY TO ROCK SOURCES

SYMBOL	SOURCE	TYPE OF MATERIAL
	QUARRY Bench 2	**Gneiss and Schist**, *moderately to slightly weathered*
	QUARRY Bench 3	**Gneiss** *and Schist, slightly to moderately weathered*
	QUARRY Bench 4	**Gneiss** *and Schist, mainly slightly weathered*
	QUARRY Benches 5 & 6	**Gneiss**, *mainly fresh*
× × × × ×	SPILLWAY-QUARRY ACCESS ROADS	**Gneiss**, *slightly to moderately weathered*
+ + + +	DOWNSTREAM ACCESS ROAD	**Gneiss and Schist**, *mainly fresh*
	SPILLWAY Bench 2	**Schist**, *moderately to slightly weathered*
	SPILLWAY Bench 3	**Schist** *and Gneiss, slightly to moderately weathered*
	SPILLWAY Bench 4	**Schist** *and Gneiss, mainly slightly weathered*
	SPILLWAY Bench 5 and Channel	**Schist** *and Gneiss, slightly weathered to fresh*
	KNOB EXCAVATION	**Gneiss**, *mainly fresh*
	ROAD SPOIL (Variation No. 7)	**Gneiss**, *fresh*
	QUARRY No. 2	**Dolomite**, *fresh and weathered*
◇◇◇◇◇	RIVERVIEW QUARRIES	**Quartzite**, *weathered*
	ZONE 1 QUARRY	**Gneiss**, *mainly slightly weathered*

STOCKPILED MATERIAL

NOTE: FOR CHARACTERISTICS OF EMBANKMENT ZONES SEE FIG.57

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Fig. 60. Distribution of the main rock materials in the embankment.

Where possible, production of zone 2 material was kept separate from the other rockfill production. An area of strong gneiss in the upper part of the spillway ski-jump excavation was reserved for this purpose. In some places the rill of a pile of blasted rock was used for zone 2 and the inner part for zone 3. This method was used to produce zone 2 material from zone 3 stockpile.

The bulk of zone 3 was constructed of material from the spillway excavation. However as construction proceeded, it became evident that insufficient material remained in the spillway to complete the zone. Material from a supplementary quarry (quarry No. 2) formed a major part of zones 2 and 3 above RL854.

Zone 1 was constructed entirely of materials quarried away from the site area, initially from Riverview Quarry, and subsequently from zone 1 quarry.

The downstream berm was constructed of rock from quarry No. 1 benches 5 and 6, and from the spillway ski-jump excavation.

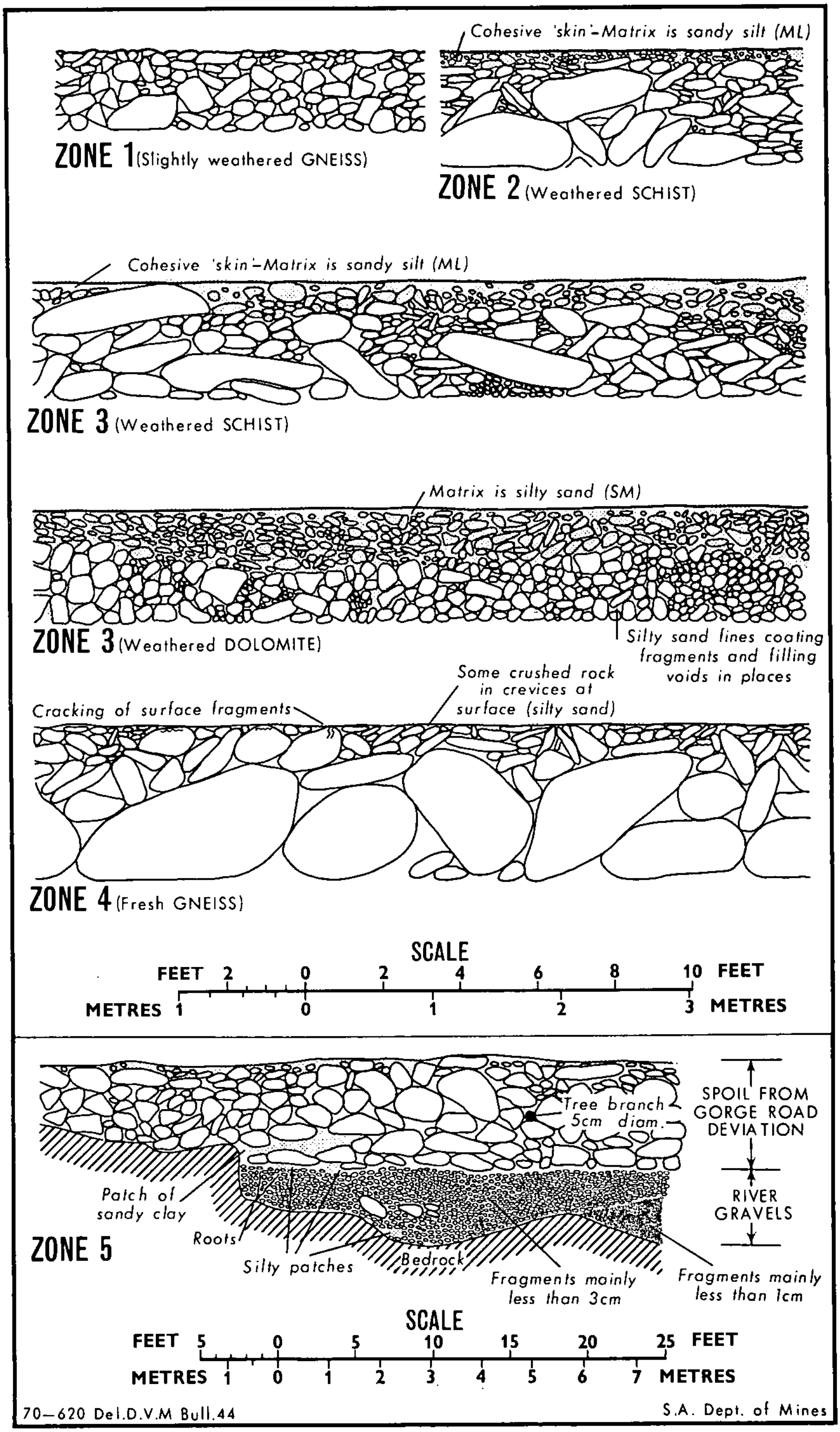


Fig. 61. Embankment zones; diagrammatic sections through typical layers.

Nature and behaviour of rockfill

Zone 1. The main criteria used to determine the acceptability of rock for zone 1 were as follows:—

1. A particle size distribution which occurred mainly within the specified grading envelope (Appendix 6).
2. Sufficient durability to withstand significant crushing during handling and rolling *i.e.* mainly strong to very strong materials.

The first 12 layers or 24 feet of zone 1 were constructed of strong quartzite material which although otherwise satisfactory, contained approximately 5 per cent of highly plastic clay fines which adhered to the fragments, and substantially lowered the permeability of the rolled rockfill. A typical grading is shown in Appendix 6.

Above RL725, zone 1 was constructed of quartz gneiss. The rock ranged from strong, moderately weathered material in the lower part of the zone, upwards into very strong slightly weathered to fresh material. The increase in strength was accompanied by a coarser grading.

A diagrammatic section through a typical layer of zone 1 is shown on Fig. 61. The fragments are mainly bulky and subangular with less than 10 per cent of flakey, more schistose material. Only very slight crushing of fragments occurred at the surface of the layers and this was caused mainly by the impact of the cleats of the bulldozer caterpillar treads on rock fragments at the surface. Each layer rolled to an even pavement surface in which the fragments and voids could be easily seen (Fig. 62). In localized parts of some layers about RL870, contamination by fines from the adjacent zone 2 material caused some filling of voids and the development of a thin permeability barrier over part of the surface.

Ring density tests through typical layers of zone 1 (Table 13 and Figs. 62 and 63) indicated dry densities averaging 120 pounds per cubic foot which corresponded to porosities of approximately 25 per cent. A test in a contaminated area indicated a density of 141 pounds per cubic foot and a porosity of 16.4 per cent.

Rolling of the face with three passes produced an even surface similar to the surfaces of individual layers (Fig. 58).

The measured vertical compaction due to rolling ranged up to 18 centimetres.

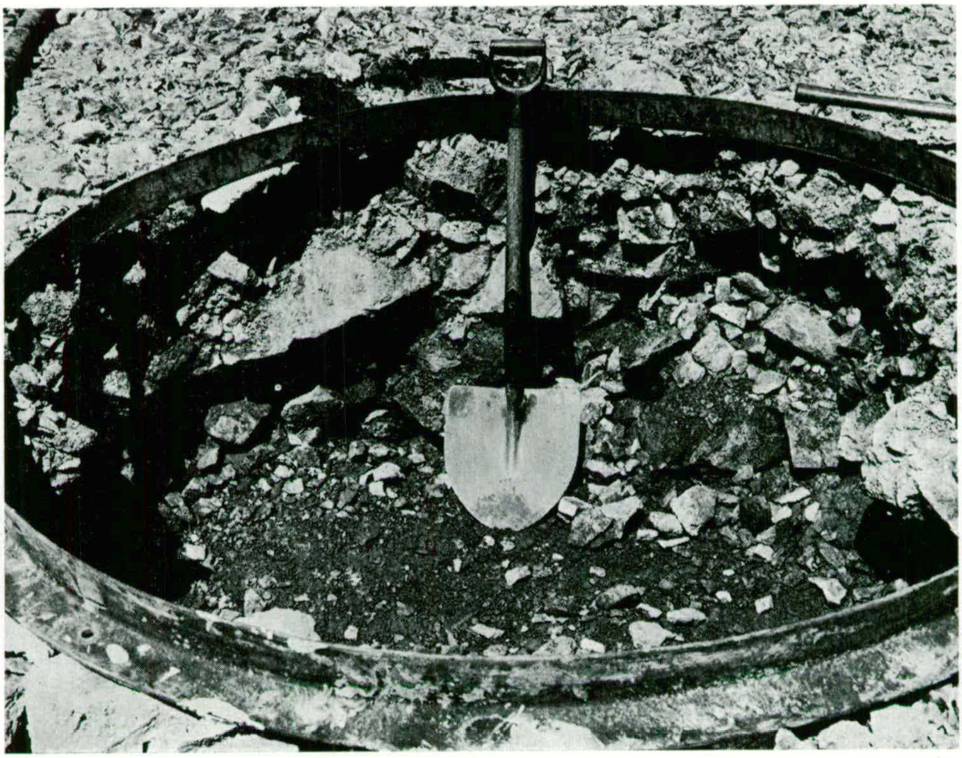


Fig. 62. Ring density test hole No. 5 excavated in zone 1 at RL792, through gneiss from zone 1 quarry.

Zone 2. The main criteria used in the selection of materials for zone 2 were:—

1. Low fines content—preferably less than 10 per cent of minus one inch material (visual estimation).
2. Low content of highly to completely weathered (weak to very weak) material.
3. Absence of fragments exceeding three feet in largest dimension.

Below RL770 the material was mainly strong, fresh gneiss, and individual rock fragments remained clearly visible on the surface after rolling, ensuring continuous rock to rock contact.

A tendency to form silty "skins" which occurred on the surface of layers between RL770 and RL860, was partly due to breakdown of weak material in the upper foot of each layer (Fig. 61), and partly to contamination by fines from adjacent areas of zone 3.

Above RL860 the criteria for selection of material were relaxed to accept dolomite material containing up to 50 per cent of minus one inch material. The only ring density test carried out in zone 2 was located in an area of dolomite (Table 13).

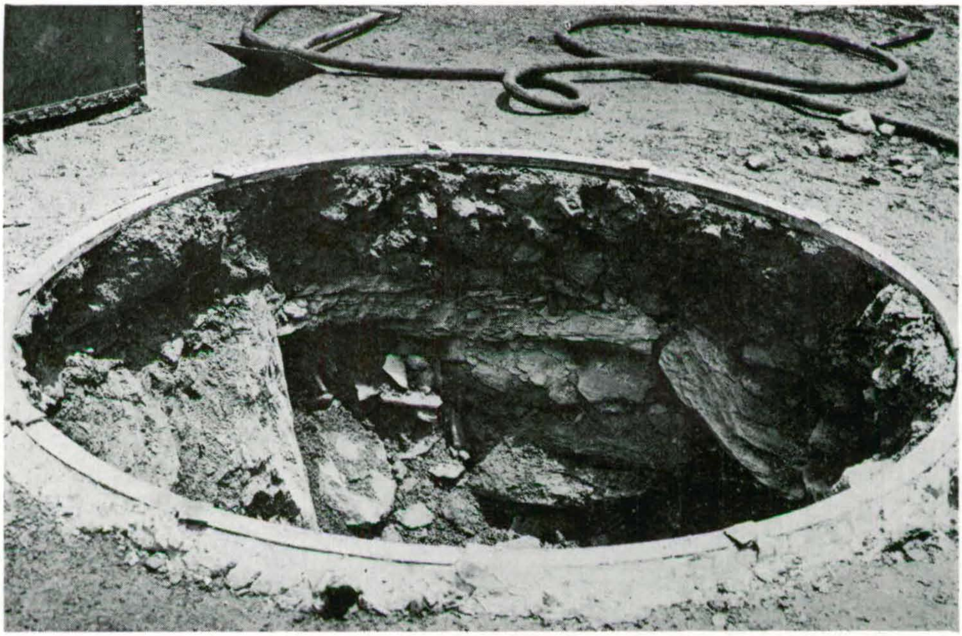


Fig. 63. Ring density test hole No. 1 excavated in zone 3 at RL751, through schist from quarry No. 1.

Zone 3. Initially the criteria for selection at the quarry face, of material for zone 3 were:—

1. A proportion of minus one inch fraction less than 25 per cent (determined by visual estimation).
2. Less than 20 per cent moderately to completely weathered (weak and very weak) material.
3. Absence of highly plastic clayey fines.

These criteria proved unsatisfactory due mainly to the difficulties involved in estimation of the proportion of fines. On the advice of the consultant J. Barry Cooke, a new criterion was adopted, namely that material be rejected if, after spreading on the embankment, the layers failed to support the weight of the construction equipment.

TABLE 13

Ring density tests

Test No.	Zone	R. L. From	R. L. To	Layers penetrated	Type of material	Source of material	Dry density	Calculated porosity	Notes
							lbs/cu. ft.	per cent	
1	3	751	737	1.3	Gneiss and schist slightly weathered to fresh	Quarry No. 1..........	150.4	12.6	6ft. ring used
2	3	758	754.5	1	Schist slightly to fresh	Quarry No. 1, bench 5..	165.0	4.4	6ft. ring used
3	3	766	762	1	Gneiss, mainly fresh	Quarry No. 1, bench 6..	142.1	17.86	8ft. ring used
4	1	778	774	2	Quartz gneiss, fresh to slightly weathered	Zone 1 quarry	123.6	24.5	5.5ft. diameter ring used. 15 per cent passing 1in. (1.6 per cent—100 sieve)
5	1	792	788.5	1.75	Quartz gneiss, fresh to slightly weathered	Zone 1 quarry	116.0	27.5	5.5ft. diameter ring used
6	3	809	804.5	1.5	Schist, mainly slightly weathered	Spillway, bench 4	150.6	12.90	8ft. diameter ring used
7	3	823	818.5	1.5	Schist and gneiss, slightly weathered to fresh	Spillway, benches 4 and 5	148.1	14.1	8ft. diameter ring used
8	3	857	852	1.8	Schist and gneiss, mainly slightly weathered	Spillway, benches 4 and 5	155.4	9.5	7ft. diameter ring
9	3	854	849	1.8	Schist and gneiss fresh to slightly weathered	Spillway channel	160.6	6.5	7ft. diameter ring
10	2	875	872	1	Dolomite, weathered	Quarry No. 2..........	151.4	12.2	8ft. diameter ring 50 per cent passing 1in.
11	1	886	882.5	1.75	Quartz gneiss fresh (contaminated with fines from zone 2 dolomite)	Zone 1...............	141.0	16.4	6ft. diameter ring

Failure of this test would indicate that rock to rock contact had not been achieved and that the material was behaving as a cohesive soil rather than a rockfill. In practice, rockfill containing up to 50 per cent of minus one inch fraction was accepted using this criterion.

A variety of materials were used in the construction of zone 3 with the result that the characteristics differ markedly from place to place. The quality of much of the material used, particularly between RL745 and RL770, was far higher than the minimum standard required, and would have been suitable for placement in zone 4. The bulk of the material above RL770 consisted of slightly weathered schist with some moderately weathered schist. Breakdown of the weaker rock which occurred during placing and compaction, added to the existing fines to form a total of 25 to 40 per cent of minus one inch material, most of which was concentrated in the upper foot of the layer (see section in Figs. 61 and 63).

In the upper part of the zone, dolomite material was used (Fig. 61). A comparison of the properties of dolomite and schist rockfill is shown in Table 14.

TABLE 14

COMPARISON OF PROPERTIES OF DOLOMITE AND SCHIST ROCKFILL

	Strength of rock substance	Grading	Particle shape	Amount of fines	Nature of fines	Density	Porosity
Schist	Medium strong to weak	Up to 2 metres in largest dimension Mainly from 10 cm to 100 cm in largest dimension	Flakey. Angular to subangular	Commonly 30 per cent—1 in.	Silt soil—(ML) sandy, clayey	150 lb/cu. ft.	10 per cent
Dolomite	Mainly strong Some medium strong and some weak material	Up to 50 cm Mainly from 5 cm to 20 cm	Bulky, subangular	Commonly 50 per cent—1 in.	Sand, excess clayey fines (SC)	150 lb/cu. ft.	12 per cent

Above RL860, most of the rockfill was placed without sluicing, as it was considered that any settlements above this level would be insignificant. Rockfill placed in this way showed a more uniform mixing of fines within the layers.

Zone 4. The criteria for selection of zone 4 at the quarry face were:—

1. A proportion of fines estimated at less than 10 per cent.
2. A low proportion of minus six inches material (less than 20 per cent).
3. A low proportion of soil and very weak rock material.

Weaker rock was accepted in large fragments provided that it remained intact during loading at the face.

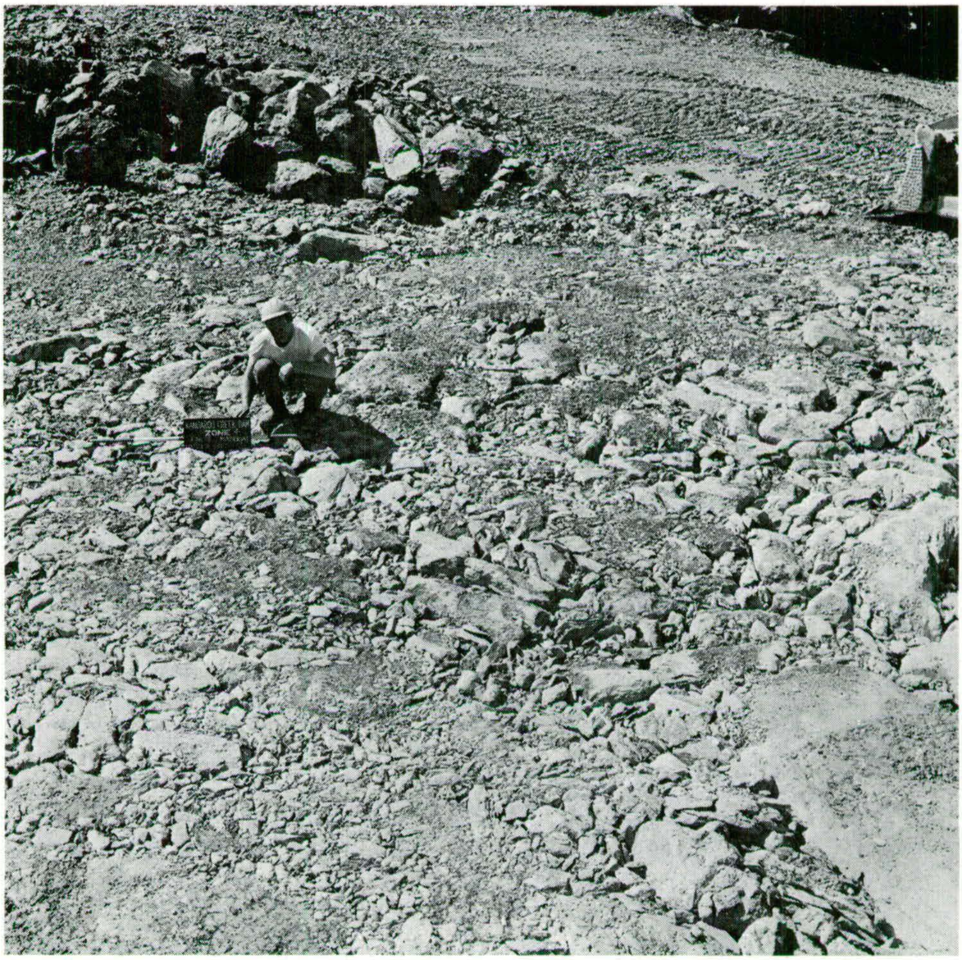


Fig. 64. Surface of typical zone 4 layer; material is gneiss from knob excavation.

On the embankment, the main criterion for acceptance of the material was that sluicing water did not form large ponds on the surface. In places where extensive ponding occurred, the full force of the sluicing jet was concentrated on the surface, washing the finer surface material into the large voids in the lower part of the layer so that the surface water could percolate through the fill.

Below RL737, the embankment contained a considerable proportion of slightly to moderately weathered rock, which underwent considerable crushing at the surface. However, the fragment size of the dumped material was mainly more than two feet, resulting in numerous large voids ensuring adequate permeability in all directions.

Above RL737, zone 4 consisted mainly of very strong to strong gneiss with a low proportion of fines (Figs. 61 and 64). This material withstood compaction with very little crushing and remained free-draining in all directions.

Zone 5. This zone consisted of existing road spoil and river gravel materials, compacted in place. After compaction, several trenches were excavated through the zone to check for any potentially compressible materials.

The trenches revealed that rock spoil was underlain by clean waterworn river gravels (Fig. 60). Patches of silty material occurred in the river gravels and in the spoil, however these were of small extent. No concentrations of organic material were revealed.

The rock material in the spoil and river gravels consisted mainly of strong to very strong gneiss with mainly less than 10 per cent clay and silt sizes.

Permeability barriers

Due to the method of placing of zone 4, sloping barriers of relatively impermeable material were formed across the embankment in several places.

The construction of the steel mesh protection at the downstream end of each layer necessitated a large amount of traffic travelling across the sloping face of each layer. The resulting breakdown and contamination caused the formation of silty barriers or "skins" sloping upstream as shown in Fig. 65.

In a similar way, delays in placing of rockfill at the upstream end of the dam resulted in the formation of a sloping barrier extending from the upstream toe of zone 5 to RL737 (see Fig. 65).

Prolonged sluicing was carried out in several areas to increase the permeability through these barriers.

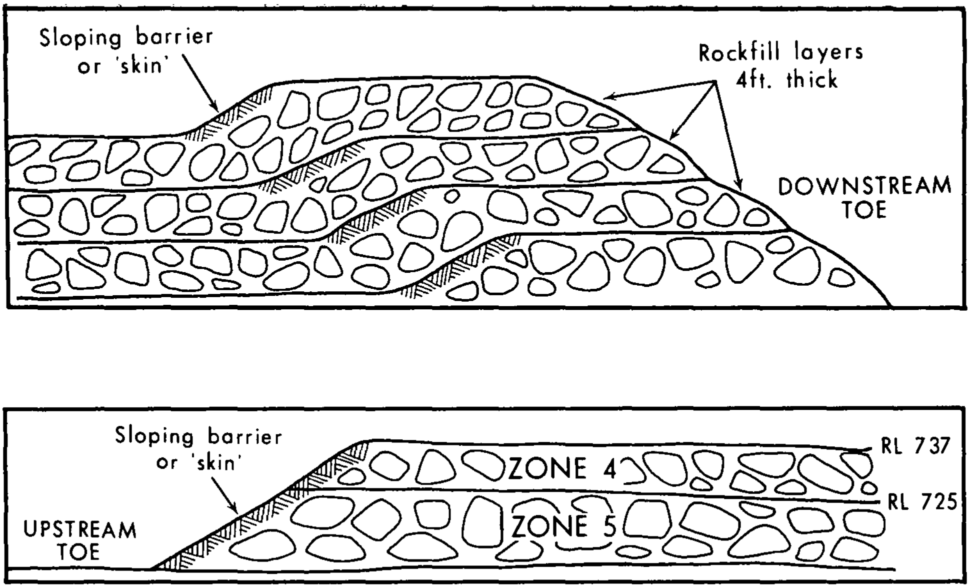


Fig. 65. Sloping barriers or "skins" developed on zone 4 and zone 5.

The surfaces of the internal ramps in zone 3 were also contaminated due to excessive traffic, however the direction of these zones is such that they do not constitute barriers to drainage through the embankment.

In most of zone 3 above RL770 and in the upper parts of zone 2, spreading and compacting of rockfill material caused considerable crushing of rock, resulting in the formation of bands of silty material in the upper parts of the layers (Figs. 61 and 66). The development of these surface skins is in direct proportion to:—

1. The amount of fines in the material transported to the embankment.
2. The amount of weak and very weak rock materials in the rockfill.
3. The amount and type of traffic passing over the surface.

Skins in the central part of zone 3 were commonly five to 15 centimetres thick and in some places attained a thickness of 50 centimetres. In some layers a continuous skin developed across the zone, but in most cases were interrupted by areas where rock fragments protruded through the skin. Sluicing water tended to form ponds on the surfaces, and most of this water eventually flowed across the surface and down the upstream or downstream face of the embankment.

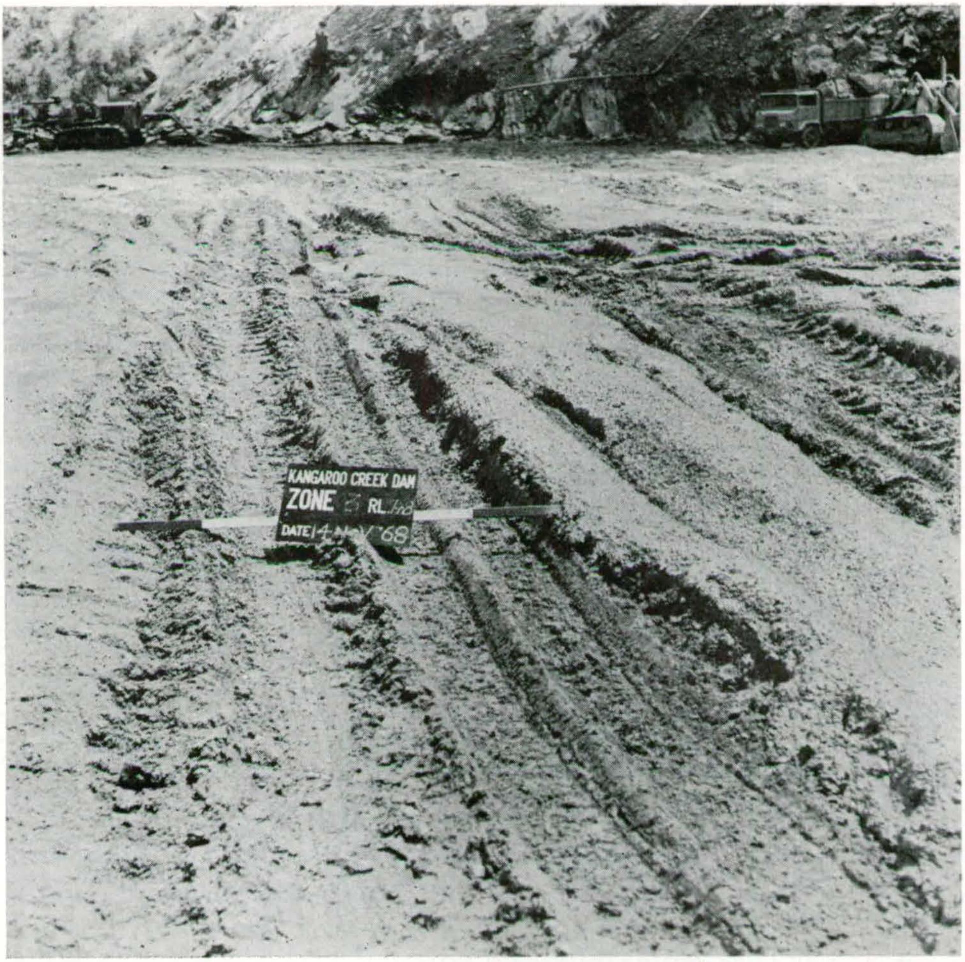


Fig. 66. Surface of typical zone 3 layer; weak schist has broken down to form cohesive skin.

The skin material consisted of rock fragments separated by a matrix of sandy silt (ML). The grading of the matrix indicates that considerable crushing had occurred of the weathered schist to its constituent minerals—mainly silt-sized mica and quartz. Detailed examination revealed rock to rock contact throughout the skins.

The results of laboratory tests carried out on relatively undisturbed samples of the skins are:—

Shear strength parameters	ϕ' 39.5° c′ 12 lb/sq. in.
Compressibility (under K_0 conditions *i.e.* no lateral strain)	1.3% under compression equivalent to 120 feet of rockfill.
Coefficient of permeability	3×10^{-5} cm per sec. *i.e.* the permeability of a silt.

These tests indicate that from the point of view of shear strength and resistance to settlement, the skins do not constitute weaknesses in the embankment. However the presence of the skins markedly decreases the vertical permeability of the zone.

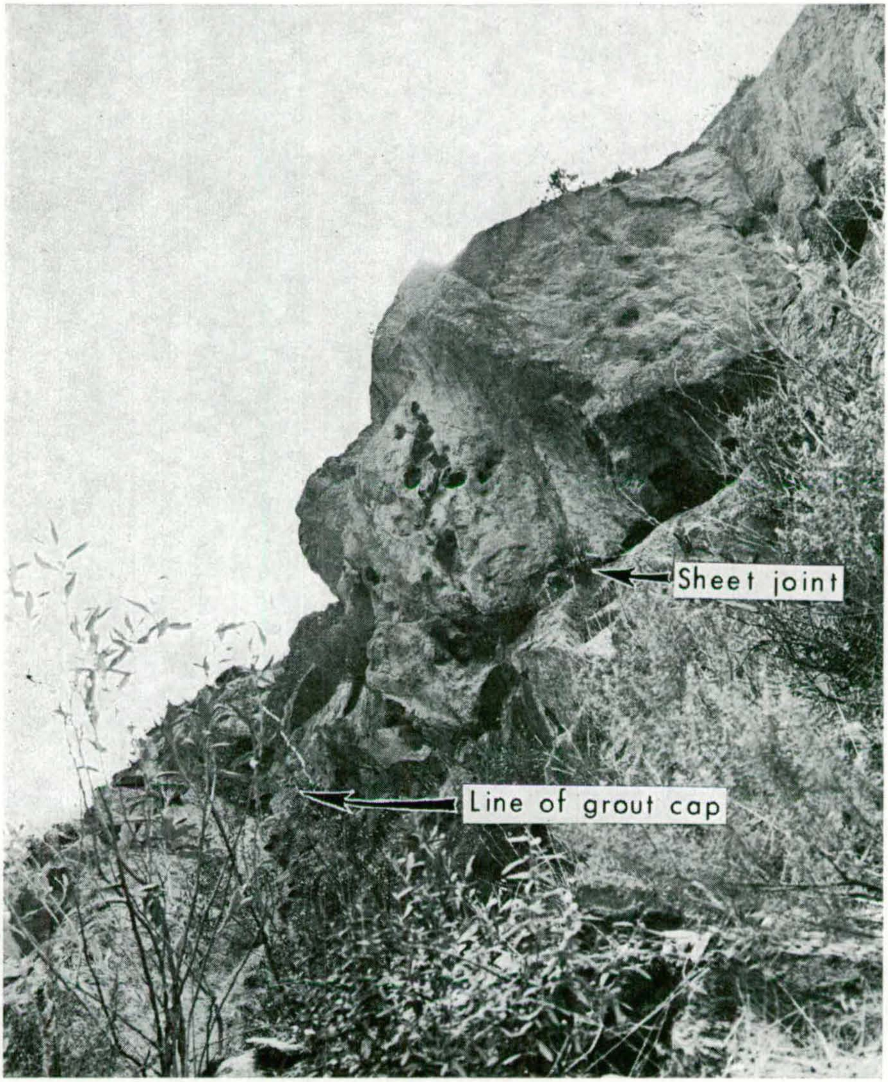


Fig. 67. Steep knob of rock in upper part of right abutment.

EXCAVATIONS TO STABILISE NATURAL SLOPES

The construction programme included the removal of two overhanging knobs of rock on the right bank (Fig. 4) which had been shown by design stage investigations to be potentially hazardous to construction activities.

Intake structure knob

The knob which occurred between the intake structure site and Batchelor's Bridge, was removed as part of the preliminary contract for the intake structure excavations. The knob was partly underlain by a prominent sheet joint which was considered to be a potential plane of sliding during excavation and construction of the nearby intake structure. A total of approximately 800 cubic yards of rock were removed and the resulting batter which slopes at approximately 60 degrees contains no defects which could cause instability.

Right abutment knob

This knob was located immediately above the proposed grout cap excavation. Several gaping sheet joints and numerous small cavities occurred in the lower part of the knob (Fig. 67). Water-flows from these joints indicated that they were connected with open joints near the level of the existing access road. It was considered that the knob if allowed to remain, would constitute a major hazard during and after excavation particularly as it would be undercut in part by the grout cap excavation. Approximately 4,300 cubic yards of rock were excavated to a pre-split batter dipping at 50 degrees towards the river. The slope was chosen to remove as much as possible of the mechanically disturbed rock and at the same time preserve a minimum width of 30 feet of access road.

Method of excavation

The contractor elected to excavate the entire knob with a single blast. The pre-split holes were drilled from the access road at intervals of three feet. In an attempt to penetrate the lowermost corner of the knob, holes were drilled towards the upstream at an angle oblique to the direction of dip of the proposed plane. Consequently these holes were up to 100 feet long compared to a maximum length of 70 feet if they had been drilled down-dip.

The burden holes were drilled from the access road on an eight by eight feet pattern in three rows. The outer rows were at a shallower angle so as to fan out into the knob.

The blast resulted in less than half of the material being removed from the knob. The remainder of the rock, although considerably disturbed by the blast (Fig. 68) proved extremely difficult to remove. When it was eventually removed by a variety of means including jetting, barring and a considerable amount of further blasting it became evident that the failure of the blast was attributable to two main causes.

1. Excessive wandering of pre-split and burden holes, which resulted in distances of more than 20 feet between the lower parts of some holes.
2. Misfires in several pre-split and burden holes.

The lowermost corner of the knob remained intact after the blast, and apparently had not been penetrated by any of the drill holes.

Stability of the knob excavation

The batter produced by the excavation (Fig. 35) contained numerous sheet joint faces, however none of these joints appear to penetrate into the rock mass.

The excavation for the grout cap exposed several prominent sheet joints underlying and approximately parallel to the knob excavation surface. The rock overlying these sheet joints was stabilized by the installation of grouted rock bolts, some of which were installed in the face of the pre-split batter (Fig. 35).

Fig. 68. Loose rock remaining on pre-split batter after blasting of knob on right abutment.

DIVERSION WORKS

Diversion tunnel

Design

The 870 feet long diversion tunnel occurs within the right abutment between the base of the intake structure 40 feet upstream of the upstream toe of the embankment, and the downstream measuring weir, 30 feet downstream of the downstream toe of the embankment (Fig. 4). The tunnel is gothic arch-shaped in cross-section, 14 feet wide at the base and 15 feet high. It is straight except for a 100 feet curved section at the downstream end and a 120 feet curved section towards the upstream end. Radius of curvature of the curved sections is 107.5 feet. The floor of the tunnel is at a grade of 1 on 143 in a downstream direction.

The tunnel capacity with water stored to the crest of the coffer dam was 2,100 cusecs. Water from Millbrook Reservoir was diverted through the tunnel during the construction by means of a four feet diameter pipe, the Millbrook trunk main. This was later supplemented by a three feet diameter pipe, to convey outlet water from the reservoir.

A 30 feet long concrete plug was constructed in a section of tunnel below the grout cap (Fig. 4). The grout curtain was extended to surround this plug. The tunnel was nominally unlined, but provision was made in the design for support consisting of steel sets and concrete lining.

Geological conditions

Despite the shallow cover ranging from 0 to 40 feet normal to the slope of the natural surface, the tunnel is in mainly fresh rock. The rock types are gneiss, schist and granitic gneiss in the proportions 60 to 20 to 20 respectively. Mechanical weathering effects are limited to 10 feet normal to the ground surface at the downstream end and 20 feet at the upstream end, and consist of numerous gaping joints and seams of infilled clay.

The tunnel for most of its length penetrates the S1 foliation direction at 45 to 60 degrees and intersects sheared or crushed zones dipping in the same directions, at intervals of 20 to 200 feet. Moderately to highly weathered rock occurs within these zones and for up to 40 centimetres on each side.

Numerous flows of water up to 20 gallons per hour occurred initially, however most of them diminished and eventually dried up. Several flows up to four gallons per hour and a few seepages persisted throughout the summer periods. The seepages and flows came from sheared zones, joints and vuggy quartz veins.

Overbreak and support

Because of their favourable orientation, Set (1) defects such as sheared zones, crushed zones and joints caused very little overbreak in the tunnel walls. In the roof, overbreak was also small, due to the broad spacing of joints in other directions (Sets (2), (3) and (4)) which could combine with Set (1) defects to form potentially unstable joint blocks. The maximum overbreak to occur was three feet in the walls and four feet in the roof.

Steel sets and concrete lining were installed to support the mechanically weathered rock masses adjacent to the upstream and downstream portals for lengths of 36 feet and 21 feet respectively. A section of three steel sets, and concrete lining 25 feet long was also installed in the central part of the tunnel, adjacent to a prominent sheared partly crushed zone which was intersected by the tunnel at an oblique angle.

In several other places where "blocky" rock occurred below sheared zones in the crown of the tunnel, the rock mass was reinforced by rock bolts. Ten rock bolts totalling 60 feet in length were installed in three separate areas.

Pneumatically applied mortar (P.A.M.) was used to protect several sheared, partly weathered zones from further deterioration.

Stability of upstream area during construction

After commencement of the main contract it was realized that to convert the diversion tunnel into an outlet tunnel, work inside the tunnel would have to be carried out for a period of up to three months after completion of the embankment. During this period water would bank up in the reservoir and if a large flood occurred, the reservoir could be filled. The resulting head of up to 180 feet of water would cause hydraulic pressures up to 78 pounds per square inch in the rock mass upstream of the barrier formed by the grout curtain and tunnel plug.

In a report on this problem (Trudinger, 1967b) it was concluded that:—

1. External pressure on the downstream section of concrete lining would be that of the full static head.
2. Within 20 feet upstream of the concrete lining, joint water pressure would be close to that of the full static head.
3. As far downstream as the tunnel plug, considerable joint water pressure would be transmitted through the open-jointed zone adjacent to the surface of the tightly-jointed zone adjacent to the tunnel. This pressure would cause rock falls in closely-jointed areas.

As a result of these considerations the following extra work was carried out to support the rock mass surrounding the tunnel upstream of the tunnel plug:—

1. The installation of one and a quarter inch thick steel lining to reinforce the existing concrete lining at the upstream end of the tunnel.
2. The construction of a further 20 feet section of reinforced concrete lining adjacent to the existing section.
3. The installation of 78 by eight feet long rock bolts to reinforce the jointed portions, which amounted to about 50 per cent of the tunnel crown, between the upstream concrete lining and the tunnel plug.

Access shaft

Design

The access shaft consists of two parts:—

1. A three-sided open cut excavation in the right bank (Fig. 69) at the north end of the downstream berm of the embankment (Fig. 4). The excavation extends up to 30 feet below natural surface level with batters sloping at 70 degrees. The base of the excavation which is a 30 feet square horizontal bench at the same level as the embankment berm (RL765), houses a control building and the collar of the shaft.
2. The shaft which is rectangular (11 by 12.5 feet) in cross-section, extends vertically for a depth of 40 feet between the open cut excavation and the diversion tunnel just upstream of the downstream curve.

The shaft is lined with reinforced concrete for a minimum thickness of one foot near its junction with the tunnel, and also, in the upper 10 feet where it forms a collar which projects a foot above the bench level and supports a metal cover. The shaft houses the 2.5 feet diameter outlet main and also provides access to the diversion tunnel by means of a ladder-way.

Method of excavation

The open cut was excavated in three stages by blasting. The broken rock was removed by hand barring in the upper two stages and by front-end loader in the lowest stage.

The shaft was excavated from the top in five feet stages by conventional shaft-sinking methods, with the blasted material loaded by hand into buckets which were hauled to the surface by crane.

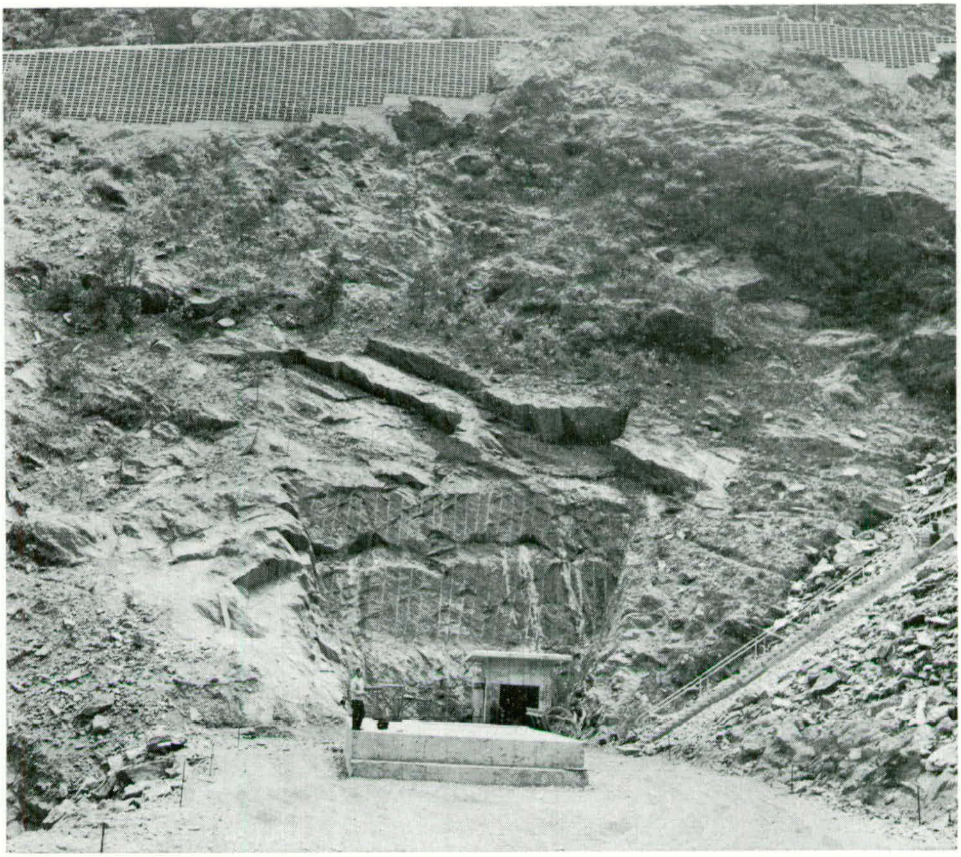


Fig. 69. A series of five near-parallel sheet joints exposed in and above the access shaft open cut.

Geological conditions

The open cut is in fresh to slightly weathered gneiss and the shaft is entirely in fresh gneiss. A series of five sheet joints near-parallel to the ground surface (Figs. 69 and 70), are exposed in the open cut and the surrounding slopes. These joints which occur in a zone up to 10 feet from the natural rock surface, are partly open and partly soil-filled up to five centimetres thick. Set (1) and (2) joints are relatively infrequent and mainly tightly closed. The shaft itself contains very few joints of any type and most of those that do occur extend for less than five feet.

Stability and support

The sheet joints within and above the open cut excavation dip downslope at approximateely 45 degrees and constitute potential failure planes. The zone of rock in which these joints occur was reinforced during construction by the installation of 15 grouted rock bolts, 10 to 15 feet in length. Set (1) joints are favourably orientated with respect to the excavation batters, and other tectonic joints are limited in extent. Following heavy rain seven

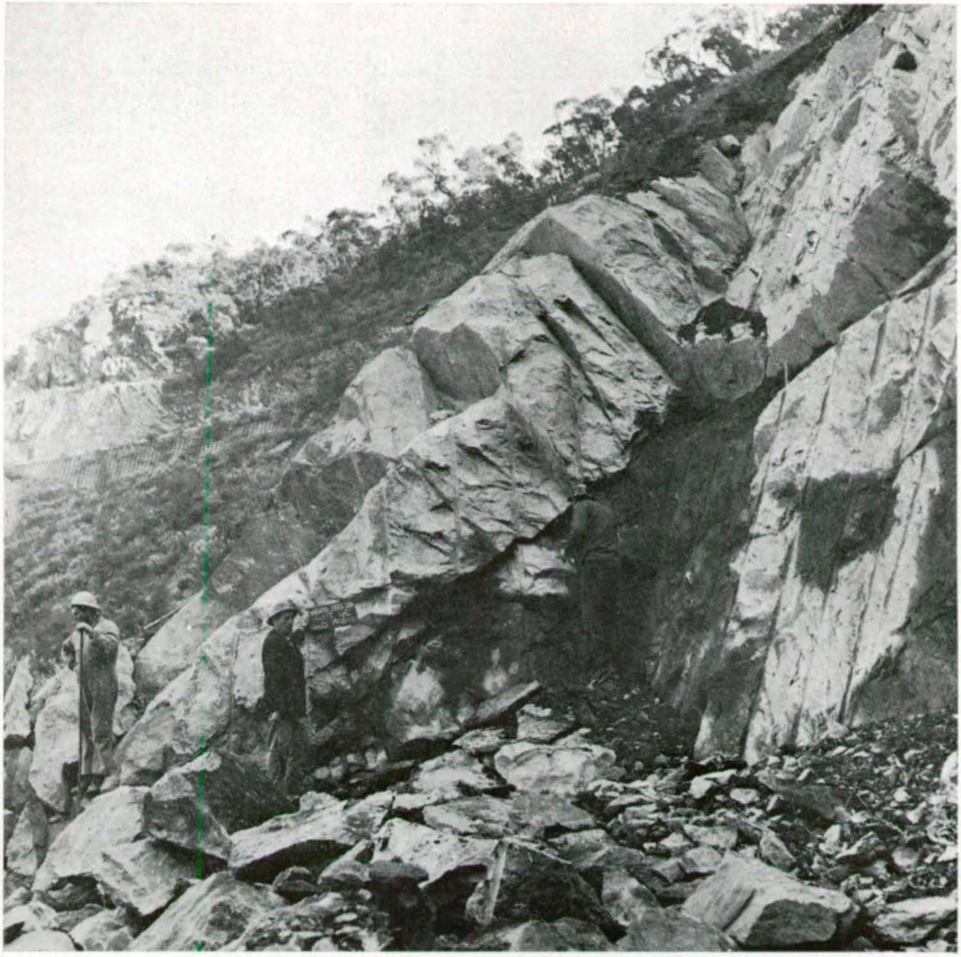


Fig. 70. Partly open sheet joints exposed in the downstream wall of the access shaft open cut.

months after excavation, there was a fall of a single joint block approximately two cubic yards in volume, which caused minor damage to the control building. The block which was located at the top of the 70 degree batter at the back of the excavation between two rock bolts, failed by sliding along a prominent sheet joint. Following this rock-fall a further nine grouted rock bolts each eight feet in length, were installed in the area.

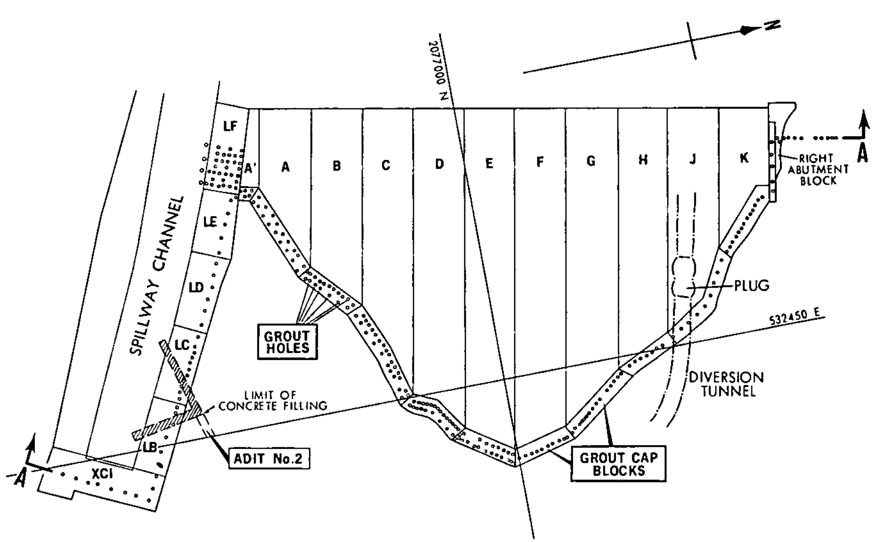
One rock bolt was installed to stabilize a ragged area at the top of the south wall of the shaft. This area has been further supported by erection of the concrete collar.

Concentration of stresses in the area where shaft excavation meets tunnel excavation have caused cracking and loosening of the rock mass in the tunnel walls. This area has been strengthened by the installation of 27 rock bolts each eight feet long in a pattern of four rows at four feet centres. The area was also concrete-lined for aesthetic reasons.

SUMMARY OF GROUTING RESULTS

BLOCK	NUMBER OF HOLES	TOTAL FOOTAGE	TOTAL TAKE (Bags)	AVERAGE TAKE PER FOOT OF HOLE (Bags)
XCI	10	400	3	0.007
LB	6	171	4 *	0.022
LC	11	414	3 *	0.007
LD	6	240	3	0.007
LE	6	240	3	0.012
LF (LEFT ABUTMENT BLOCK)	36	1420	18.5	0.020
A'	5	200	30	0.150
A	18	795	6	0.008
B	18	900	21	0.023
C	25	1420	25	0.017
D	30	1425	31	0.022
E	24	1460	57	0.039
F	10	585	30	0.051
G	16	910	31	0.034
H	7	389	10.5	0.027
J	9	521	5	0.010
K	13	560	20	0.036
RIGHT ABUTMENT BLOCK	9	360	12	0.033
ACCESS ROAD	7	318	67	0.208
TUNNEL UNDER BLOCK J	15	363	5	0.014

* EXCLUDES GROUT USED AROUND ADIT CONCRETE



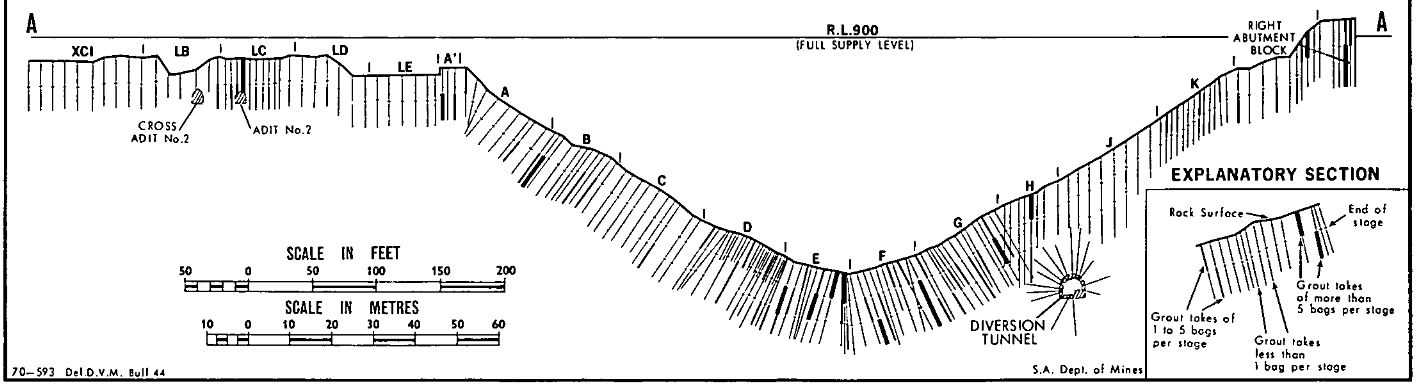


Fig. 71. Distribution of grout holes; plan and section, also summary of grouting results.

GROUTING

Design

To reduce leakage through the near-surface zone of rock around the upstream perimeter of the dam, a grout curtain was constructed consisting of 281 holes with a total length of 13,091 feet. Grouting operations were facilitated by the provision of the concrete grout cap through which most of the holes were drilled. The distribution of grout holes is shown in plan and section in Fig. 71. The holes ranged in depth from 60 feet at the toe of the dam (blocks E and F), to 40 feet in the upper levels.

On the right bank the holes were drilled in a single line and fanned out where necessary so that the distance between the ends of adjacent holes was less than or equal to 10 feet. In blocks J and K and the right abutment block, the plane of holes was also angled into the abutment at approximately 70 degrees. Holes in the upper part of the right abutment block foundations, and in the access road foundations, were collared directly into rock. Holes were also drilled from the diversion tunnel to intersect the grout curtain beneath block J.

On the left bank the holes were drilled in two rows along the grout cap and in a single row beneath the spillway overflow lip. In addition to this curtain grouting, additional "consolidation holes" were drilled in block C to ensure the intersection of specific geological weaknesses; and a pattern of "blanket grouting" was carried out in the highly disturbed rock mass at the upstream end of the left abutment block.

Procedures

The holes were drilled using drifter-type percussion drills, and grouting was by means of the downstage technique with a single packer at the top of each stage. Primary holes spaced approximately 10 feet apart were drilled in 20 feet stages. Successive holes were drilled and grouted in order so that the leading hole was a stage behind the trailing hole. Secondary holes were drilled and grouted only if the combined take of the same stages of two adjacent primary holes exceeded two bags of cement. Tertiary holes were drilled and grouted in most places where takes in the secondary holes exceeded two bags.

The grouting pressures were increased with the depth of the stage as follows:—

Stage	Pressure (lb./sq. in.)
0-20 feet	20
20-40 feet	40
40-60 feet	60

Grout consisting of water and cement in a 5 to 1 ratio was used throughout the programme, however thicker mixes were used where prolonged high leakage rates occurred. Grout injection was terminated when the take rate fell below 0.1 per bag per 20 minute period.

In places where leakages of grout occurred at the surface, they were sealed by caulking with oakum.

Results

The grout "takes" are summarized on Fig. 71 which also gives the average "take" per foot of hole drilled in each area.

Two large takes of more than 20 bags occurred, one below the grout cap block A, and the other below the access road, adjacent to the right abutment block. The rest of the takes were less than one bag, apart from a few moderate takes of five to 10 bags which mainly occurred from holes drilled in the lower part of the valley through grout cap blocks E and F. Due to the small takes in the primary holes, approximately half of the proposed secondary holes were not drilled. Tertiary holes were only required in three areas.

Small leakages occurred in several areas, mainly on the left bank. The maximum distance of surface leakage from the point of injection was 30 feet. In the spillway overflow lip area where the grout holes were collared directly into the rock foundations, widespread surface leakages in the vicinity of the grout holes made it difficult to accurately estimate grout takes.

Survey measurement of the grout cap did not reveal any movement of the concrete blocks during grouting operations.

Part 4
EVALUATION

STABILITY OF LEFT ABUTMENT

The geological investigations for the proposed concrete arch dam (Stapledon, 1966) revealed within the left bank, the presence of numerous clay seams indicating that the rock mass within 100 feet of the surface had been affected by small but widespread slide movements, and that further sliding was likely in the future. Stapledon (1965) observed that " . . . it is felt that a major slide upstream from the dam (arch site) is a distinct possibility during operation."

Excavation for the grout cap and spillway foundations on the left bank exposed numerous infilled clay seams (Figs. 31 and 39), confirming that the rock mass in this area had been affected by widespread slide movements. Work carried out to study the nature of these slide movements included:—

1. Detailed logging of excavations within the affected area.
2. Installation of "tell-tale" mortar pads across joints and seams.
3. Drilling and logging of two diamond drill holes.
4. Reassessment of drilling and mapping data from the feasibility and design stages.

The presence of infilled clay seams along sheared zones, crushed seams and joints indicates that the rock mass has been disturbed in the recent past, causing relative movements of unit blocks with respect to each other.

Several separate areas of deep-seated sliding are recognized. The rock masses between these areas are relatively undisturbed except for the normal near-surface zone of mechanical weathering. Such relatively intact masses were exposed in the outer parts of the exploratory adits, the foundations for grout cap block D and the lower part of block C, and diamond drill hole KC39.

Disturbed areas upstream of embankment

Disturbed area A

Several slides in this area were delineated during exploration for a concrete arch, and two are outlined on Fig. 72. They are best explained by postulating that movement has occurred along flat-lying seams near river level. Some evidence for the presence of such seams was obtained from the core of exploratory diamond drill holes (Stapledon, 1965).

Disturbed area B

A prominent seam (seam A) dipping at 40 to 45 degrees crosses the grout cap at the boundary of blocks A and B. Another prominent seam (seam C) crosses the grout cap in the upper part of block C. The rock mass between these seams contains numerous infilled clay seams up to 100 millimetres thick (Figs. 30 and 31).

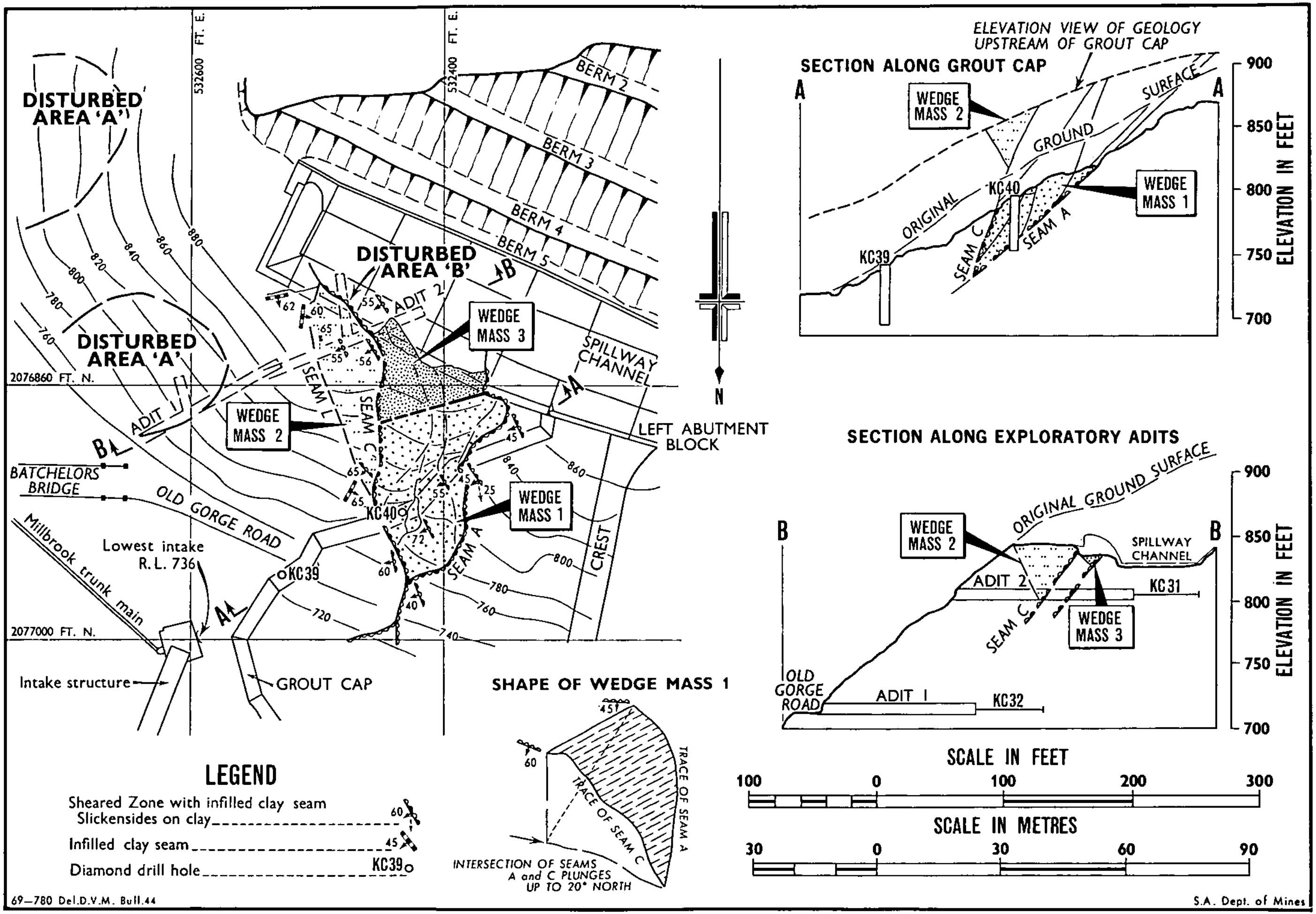


Fig. 72. Disturbed areas upstream of embankment; plan and sections.

Originally it was considered that the rock mass in this area had been disturbed by sliding movements along one or more flat-lying seams situated near the floor of the valley. However, exploratory drilling of holes KC39 and KC40 (Fig. 72) and detailed mapping of grout cap and embankment foundations, failed to reveal any such weakness. Slickensides on infilled seams (see Fig. 72) suggested that the movements had been towards the north, *i.e.* in a direction partly downslope and partly downstream. This consideration led to an understanding of the disturbances as movements of wedge-shaped masses in a downstream direction. The boundaries of the main disturbed area have been delineated by detailed mapping of exposures in the grout cap and spillway excavations, and are shown in Fig. 72.

Two types of wedges have been recognized:—

1. Open wedges formed by two seams dipping towards each other. The intersection may form an acute angle or an obtuse angle.
2. Recumbent wedges formed by two seams dipping in the same direction and intersecting at an acute angle.

Disturbed area B has been divided into three wedge masses (Figs. 72, 74 and 75).

Fig. 73. Early stage in excavation for grout cap, block B through soil and disturbed rock—part of wedge mass No. 1.

(1) *Wedge mass* 1. This recumbent wedge forms the foundation for grout cap block B and the upper half of block C (Fig. 74). The wedge lies between two clay seams, the basal seam A and the overlying seam C, which each contain up to 20 centimetres of infilled clay. The back of the wedge (seam K—see Fig. 46) contains 40 centimetres of infilled clay. Other Set (1) seams occur within the wedge mass dividing it into smaller wedge-shaped blocks.

Stereographic projection of measurements of the orientations of the seams near the toe of the wedge indicate a local plunge of 18 degrees towards the north. Slickensides on seam C near block A suggest that the mass plunges at 25 degrees toward the north. However, comparison of the elevation of intersection of the seams projected on section D (Fig. 72), and the elevation of the toe of the wedge, suggests that the overall plunge between these points is almost zero.

Fig. 74. Base of wedge mass No. 1 exposed in embankment foundation downstream of grout cap, block C.

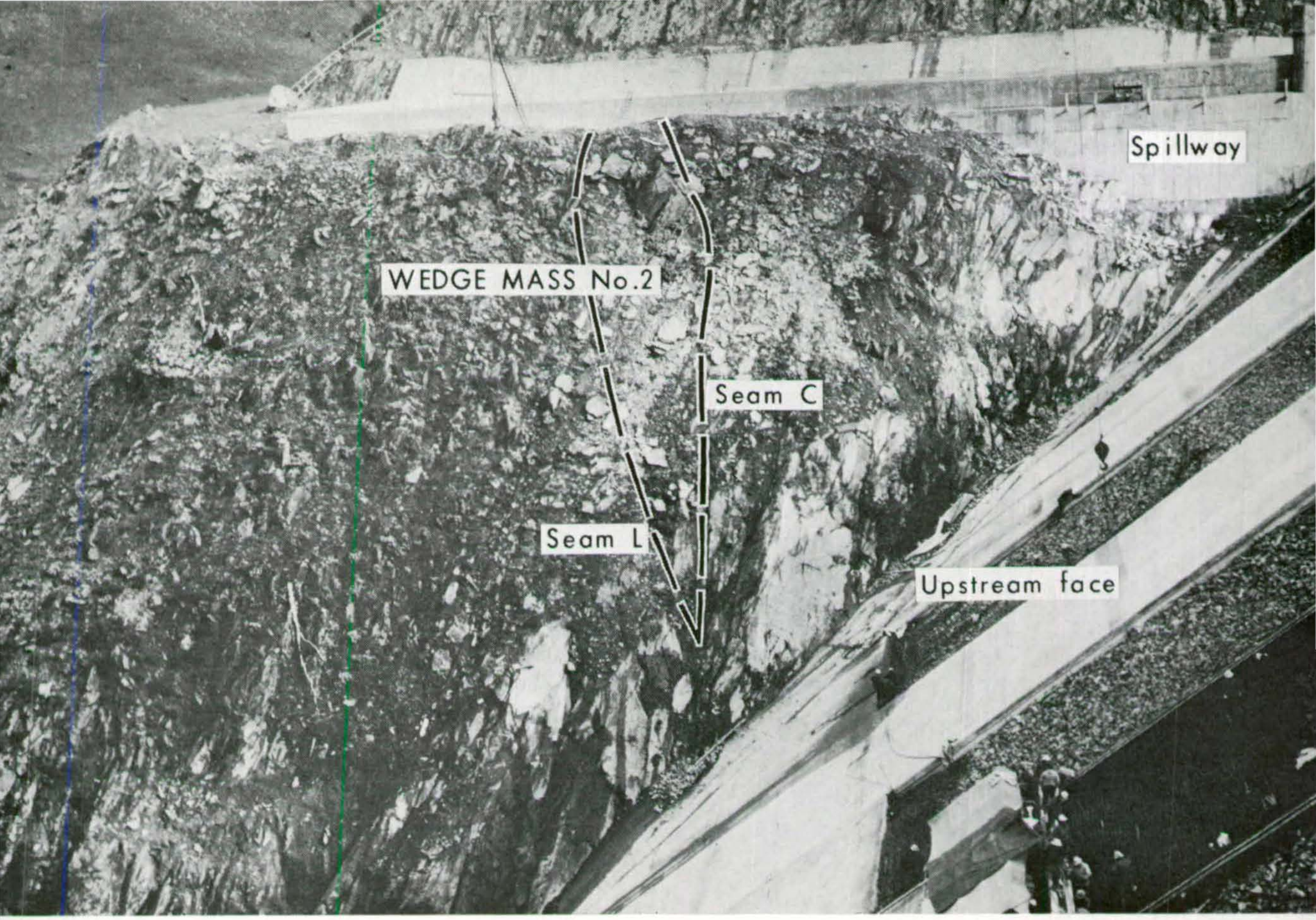


Fig. 75. View of left bank upstream of dam embankment. Wedge mass No. 2 is located between spillway block LB and grout cap, block C.

(2) *Wedge mass* 2. A disturbed open wedge of rock occurs at the upstream end of the spillway lip excavation between seam L and seam C (see Figs. 75 and 76). Examination of the grout cap excavation batter above block C revealed intersecting seams with a similar orientation (Fig. 77). Two seams in adit No. 2 also correlate with seams C and L (Fig. 72, section B-B).

These observations suggest that a continuous open wedge containing approximately 4,000 cubic yards of rock, extends between the grout cap block C and the spillway overflow lip block LB. Stereographic projection of the measurements of seam orientations indicate that the wedge plunges towards the grout cap at 25 to 40 degrees. The plunge calculated from the elevations is approximately 25 degrees. Infilled seams at the back or upper end of the wedge are less than 25 millimetres thick, indicating the past movements have been small.

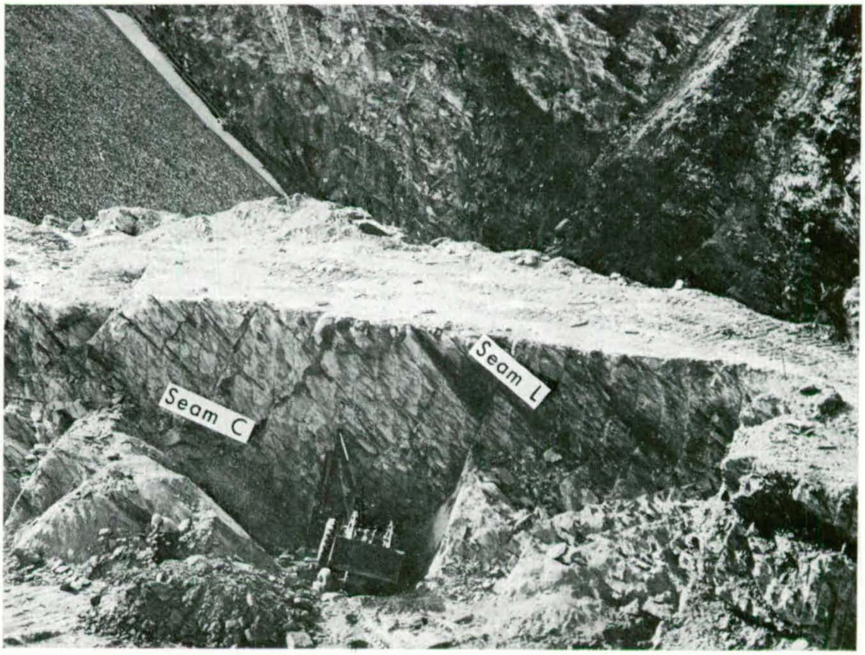


Fig. 76. Excavation of upper part of wedge mass No. 2 in foundation for spillway overflow lip, block LB.

Fig. 77. Lower part of wedge mass No. 2 in grout cap excavation. The wedge toes out 15 feet above grout cap, block C.

(3) *Wedge mass* 3. This recumbent wedge occurs behind wedge mass 1 and is bounded by the same Set (1) seams (A and C). The back of the wedge is formed by several Set (2) joints containing up to 80 millimetres of clay, indicating that past movements of the mass have been small in comparison to the movements of wedge mass 1. Several other Set (1) seams up to 100 millimetres wide, occur within the wedge mass, dividing it into smaller wedge-shaped blocks.

Disturbed areas in embankment foundation

The disturbed rock mass which occurs beneath the left abutment block (Figs. 48, 49 and 50), extends northwards beneath the embankment. Detailed observations in the left abutment block foundations indicate that small displacements have occurred between two Set (1) seams, suggesting that these seams form the sides of a large deep-seated recumbent wedge mass similar to wedge mass No. 1. The back of this mass occurs near the junction of the left abutment block and the spillway channel floor.

Clean-up of the embankment foundations was insufficient to allow delineation of the main areas of deep-seated sliding. However, the presence of numerous seams of infilled clay, some up to 50 centimetres wide, indicates that much of the foundation has been affected by slide movements. The large quantity of loose material removed from the central part of this area (see Fig. 22) was mainly debris from an old open wedge slide which extended from the upper part of the left bank down almost to river level. Fig. 19 shows the faces of this wedge exposed in the foundation for spillway block LH. The highly disturbed nature of the debris and the high proportion of soil materials suggest that most of the material had slipped for relatively large distances.

Disturbed areas downstream of embankment

Several disturbed blocks were exposed in the upper parts of the northern batter of the spillway channel excavation (Fig 39). Some of these blocks were open wedges formed by intersections of Set (1) and Set (2) defects (see Fig. 41) and others were tabular or irregular blocks underlain by sheet joints (see Fig. 40). The joints contain infilled clay seams up to 20 centimetres wide.

The valley wall downslope of spillway block LQ contains a large V-shaped gully indicating an old wedge failure extending to the floor of the valley. Other parts of the slopes also show the effects of slide movements, however the soil cover in this area has prevented delineation of the main-slide masses.

Origin of past slide movements

Movements of many of the slide masses on the left bank such as wedge mass 1, cannot be explained fully in terms of gravitational forces, as the plunges of the wedges are at angles of less than 20 degrees. It is considered that the movements have been initiated by forces associated with the relief of the high stresses which occurred during erosion of the Torrens Valley. Earthquake shocks may also have contributed to some movements.

Detailed mapping in the spillway overflow lip area has revealed displacements of up to 400 millimetres along Set (1) joints and seams (Fig. 39). The relatively small amount of infill at the back of the disturbed blocks (less than 80 millimetres) indicates that the displacements are not due to slide movements and must be reverse faults of tectonic origin.

Several "tell-tale" mortar pads installed across seams in wedge mass 1 during construction developed fine cracks open less than 0.1 of a millimetre. The nature and distribution of these cracks suggests that they were caused by vibration of individual joint blocks due to passing traffic or nearby explosions, and hence that they do not indicate that the wedge mass was in motion during this period.

Treatment of areas of instability

The attitude adopted towards treatment of the disturbed areas was influenced by several factors:—

(1) The consequences of failure, depending on the size of the instability and its location with respect to the project features.

(2) The feasibility of treatment, including practical, economic and contractual considerations.

(3) The estimated probability of failure occurring under operating conditions.

Factors (2) and (3) were difficult to assess with any degree of reliability, particularly in those areas where the nature of past disturbances were not completely understood.

Disturbed area A

It is considered that deep-seated sliding of up to 100,000 cubic yards of rock material is likely to occur in this area. However, as the area is situated well upstream of the important project features, sliding is unlikely to cause significant damage and hence the area does not warrant treatment.

Disturbed area B

The presence of the dam embankment prevents further movement of wedge masses 1 and 3. Individual wedge blocks up to 100 cubic yards in volume located above the grout cap within wedge mass 1, may be free to move and may slide on to the grout cap or upstream face.

Wedge mass 2 is apparently free to slide onto the embankment, and calculations made by the Engineering and Water Supply Department Design Branch suggest that failure is likely to occur under operating conditions. It was considered that failure of this wedge would cause damage to the concrete upstream face and grout cap and that the resulting debris could possibly damage the intake structure and block the lowermost intake portal. Studies of simulated slides were carried out using a plaster model of the site, and the results (see Fig. 78) indicated that the slide debris was unlikely to reach the intake structure.

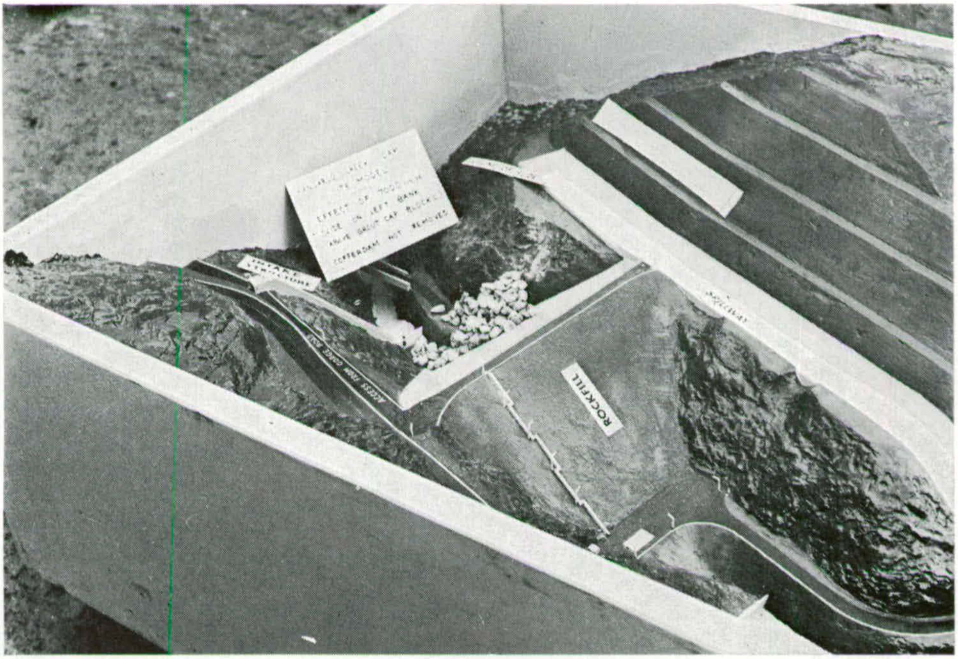


Fig. 78. Plaster model of project, with aggregate simulating a slide of 7,000 cubic yards of material from wedge mass No. 2.

A programme of stressed cables or bolts was considered to stabilize wedge mass 2. However, such a programme would have proved extremely inconvenient and expensive. It was considered that damage to the upstream face and grout cap concrete would be avoided if the impact could be "cushioned" by a protective layer of loose material. A layer of three-quarter inch aggregate, three feet thick, was placed on the upstream face below wedge mass 2 (Fig. 79).

Disturbed areas within embankment foundation

Removal of the upper parts of the wedge masses in the spillway excavation decreased their weight and so reduced the possibility of sliding. The weights of the remaining slide masses were clearly small compared with the weight of the embankment confining them. It was considered that the presence of the embankment would prevent any further movement of the wedge masses, and that no treatment was required.

In effect the disturbed masses are considered as particularly compact parts of the embankment, rather than as parts of the foundation.

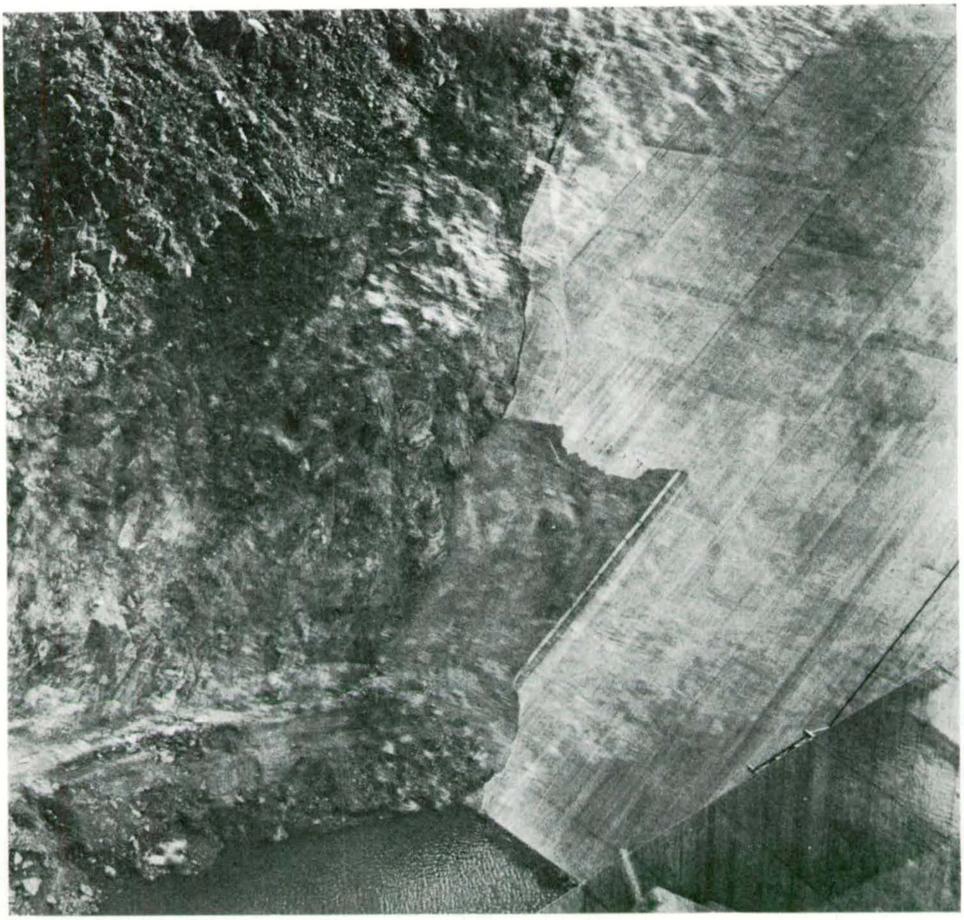


Fig. 79. Protective layer of —$\frac{3}{4}$ inch aggregate placed on upstream face of embankment below wedge mass No. 2.

Disturbed areas downstream of embankment

The stresses in this area have been considerably reduced due to the removal of material in the spillway excavation and due to the draining effect of the excavation. Nevertheless it is considered possible that some sliding may occur which could cause minor damage to the concrete lining in the upper part of the north wall of the spillway channel. Such damage would not significantly affect the operation of the project and could be easily repaired. Hence it was considered that no treatment was warranted.

RAPID DETERIORATION OF ROCK SUBSTANCES

During the investigation for the concrete arch dam it was observed that " . . . fresh, intact schist exposed in road excavations developed a surface layer of soft powdery material after several months of exposure" (Stapledon, 1966). Later a similar type of rapid deterioration was observed on the surface of the exploratory adits. This led to a study of rapid deterioration in excavations and natural exposures throughout the site area, to determine the nature and mechanism of the deterioration and hence its engineering significance.

Description of deterioration effects

(1) *Surface fretting.* This type of deterioration occurs in the exploratory adits, in several surface caves (Figs. 80 and 81) and in a few protected parts of road cuttings (Fig. 82). It consists of the progressive flaking

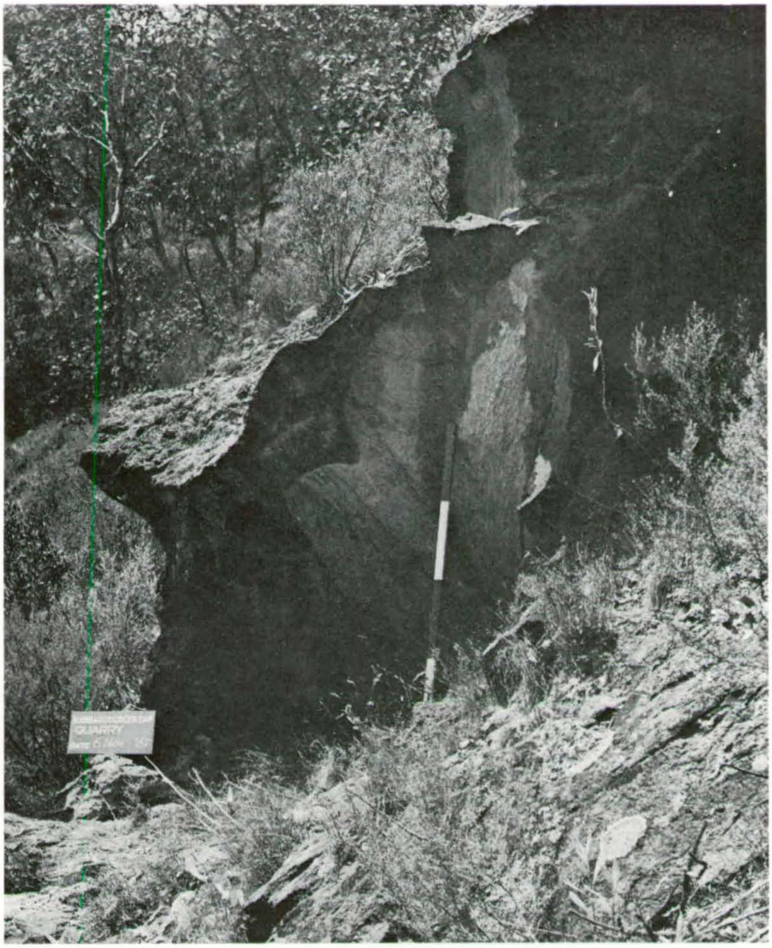


Fig. 80. Natural cave. Inner surface is actively fretting to sand and silt-sized flakes. Site of quarry No. 1.

of sand and silt-sized rock fragments. The rocks affected consist of slightly to moderately weathered schists. The fretted material has the characteristic bitter taste of sulphates. The following minerals were identified in evaporates from solutions obtained by soaking the fretted materials in water:—

Bianchite—(Zn, Fe, Mg)$SO_4.6H_2O$
Halite—NaCl

In the adits the fretting is closely associated with encrustations and crystalline growths of soluble salts, of which several different compositions have been identified:—

Gypsum—$CaSO_4.2H_2O$
Epsomite—$MgSO_4.7H_2O$
Bianchite—$(Zn, Fe, Mg)SO_4.6H_2O$
Hallotrickite—Pickeringite—$(Fe, Mg)Al_2(SO_4)_4.22H_2O$

Thin section examination of surface rock (sample 12, Appendix 2) shows that the fretted flakes are no more chemically weathered than the intact material. Limonite-filled microfractures which occur in the intact material appear to be related to the weathering of sulphides.

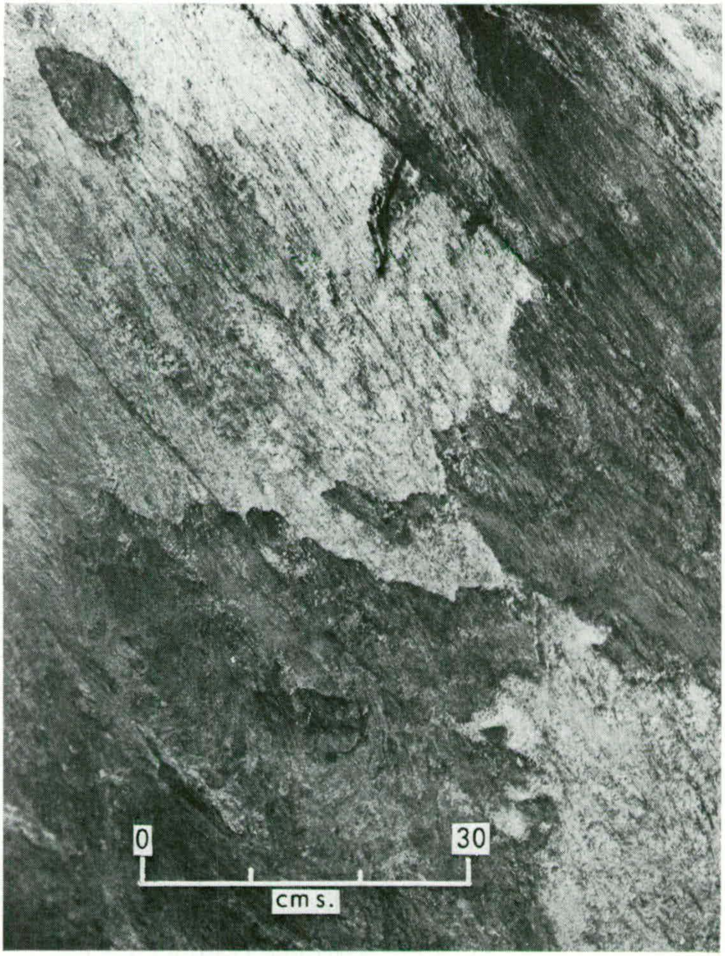


Fig. 81. Close-up view of inner surface of cave shown in Fig. 80. Lighter areas are actively fretting. Dark areas are case-hardened and show no signs of fretting.

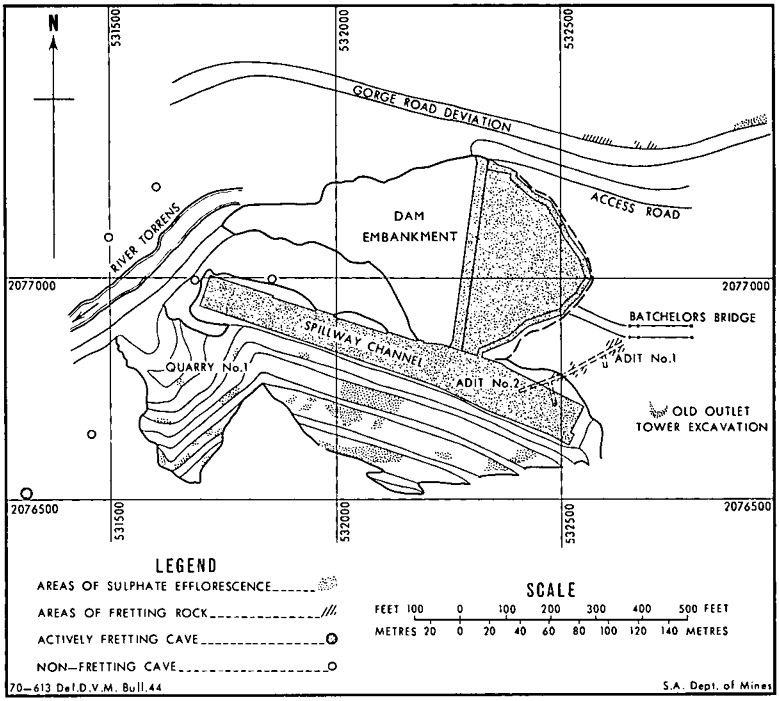


Fig. 82. Rapid deterioration of rock surface; plan showing affected areas.

(2) *White encrustations.* This type of deterioration occurs in parts of road excavations and spillway and quarry open cuts (Figs. 82, 83 and 84). It consists of white crystalline salts filling microfractures in the rock (Fig. 84) and encrusting parts of the exposed surface. Schist and gneiss rock substances ranging from almost fresh to slightly weathered, have been affected in this way. These substances are considerably weaker than similar unaffected rock substances. A typical sample of this material was treated with distilled water and the resulting solute evaporated. The salts obtained by this evaporation were bianchite and halite which together comprised 6 per cent by weight of the sample.

The areas of most pronounced encrustation are associated with relatively high amounts of sulphides in the rock, up to 5 per cent pyrite and chalcopyrite in some localized bands. Thin section examination of affected rock substances (sample 13, Appendix 2) indicates that the sulphides are partly weathered to limonite and that the weathering is associated with microfracturing of the rock. In spite of the fact that no water was used in the preparation of thin sections, no sulphates were identified.

(3) *Yellow powdery encrustation.* This effect occurs in parts of the diversion tunnel and adit No. 2. The affected rocks were mainly gneisses containing pyrite. The yellow mineral was identified as carpho-siderite—(H_3O) Fe_3, $(SO_4)_2$ $(OH)_6$, which is closely related to natro-jarosite, a

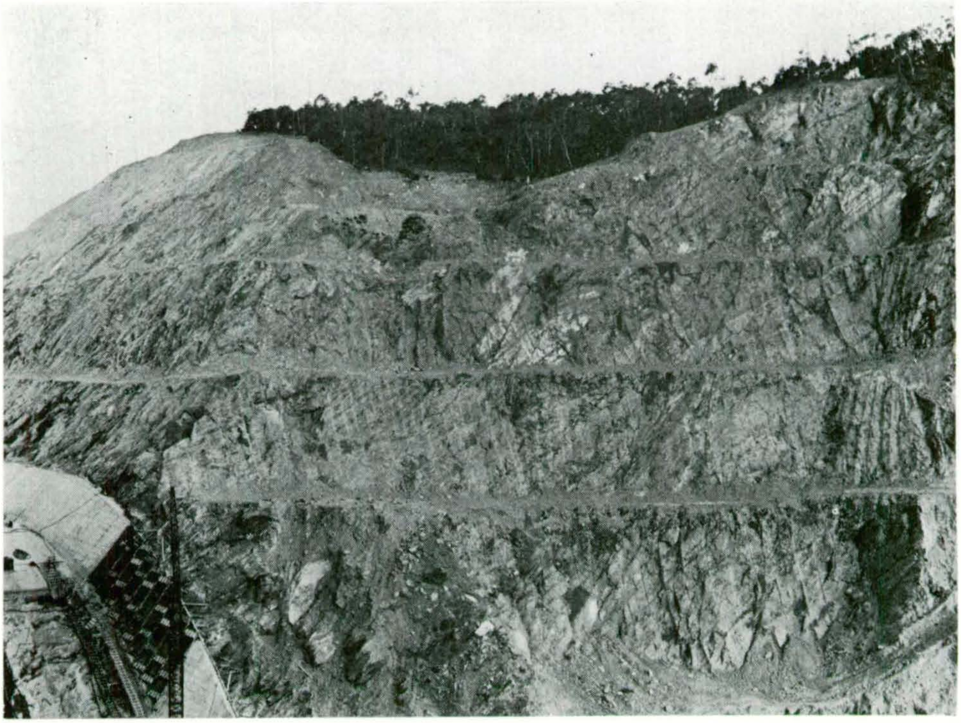


Fig. 83. White patches of efflorescent sulphate salts encrusting surface of weathered schist, upper part of quarry No. 1.

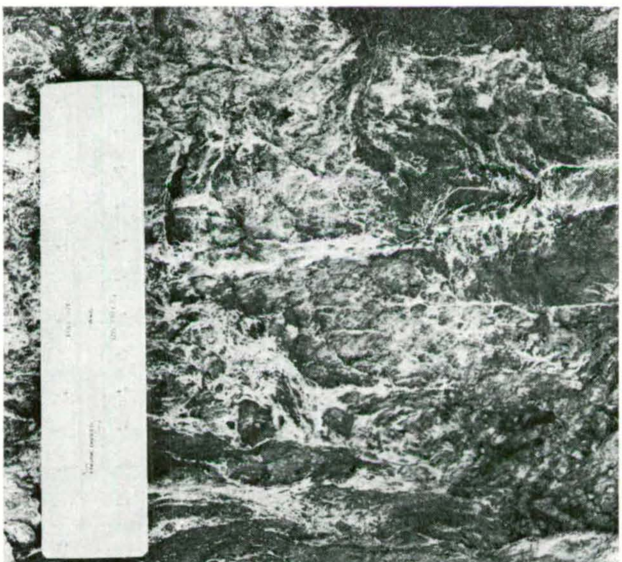


Fig. 84. Efflorescent sulphate salts concentrated in microfractures at the surface of weathered gneissic schist; Gorge Road deviation.

mineral commonly formed by weathering of pyrite. In the diversion tunnel this jarositic mineral occurs as a powdery encrustation on the surface of the rock. The rock substance itself shows no signs of weathering or weakening.

In the embankment foundation several localized zones occur, in which the rock substance contains jarosite to a considerable depth. This rock is considerably weaker than adjacent non-jarositic material. However there is no evidence that this deterioration was particularly rapid.

Mechanism of deterioration

In all of the many occurrences of rapid deterioration in the site area and also in other occurrences in rocks in the Mount Lofty Ranges, the deterioration has occurred in rocks containing sulphide mineralization. This fact plus the fact that most of the products of deterioration contain substantial amounts of hydrated sulphate salts, leads to the conclusion that the deterioration is related to the oxidation of sulphide minerals.

The probable mechanism is as follows:—

1. In the near-surface zone, ground moisture gains access to the rock substance along microfractures which form due to stress relief and in some areas as an effect of blasting.
2. The sulphides particularly pyrite, are oxidized to form sulphate salts.
3. With seasonal decreases in moisture content the sulphates crystallize in the form of hydrated salts. The expansion which accompanies crystallization results in opening of the microfractures in which the crystals are forming. This mechanism is identical to the mechanism of breakdown in the sulphate soundness test (Minty, 1965; U.S. Bureau of Reclamation, 1963).
4. At the surface the sulphate may crystallize as an encrustation or may form larger crystal growths.

The fretting type of deterioration is restricted to schistose rocks, and the size of the flakes suggest that the sulphates are crystallizing along all the foliation planes for a distance of up to five millimetres from the surface.

The white encrusting type of deterioration occurs in stronger rock substances and results in considerably less weakening of the rock fabric.

The yellow powdery encrustation appears to be derived only from sulphides present at the surface, and the adjacent rock is unaffected.

Halite (sodium chloride) which was identified from the deterioration products in surface cuttings and natural cavities, has probably contributed to deterioration in the same way as the sulphate salts. However, the halite probably originates as cyclic salt rather than as a weathering product of constituent minerals.

Dragovich (1967) describes a similar type of cavernous deterioration in South Australia, which she attributes to hydration of silicate minerals. It is suggested that soluble salts, either derived from constituent minerals or cyclic in origin, may also be responsible for these effects.

The results of research into the effects of the sodium sulphate soundness test (Minty, 1965; Minty and Monk, 1966) show that the breakdown of rock substance is directly proportional to the absorption of the material, which is related to the presence of voids, microfractures, *etc.*

It was also found that the amount of breakdown was inversely proportional to the size of the fragments. This appears to be partly a function of surface area and also partly related to the stresses of fragmentation. A similar effect was observed in the exploratory adits where fretting was most pronounced adjacent to traces of drill holes.

Sulphate soundness tests on samples of rock substances (Appendix 5) from severely affected areas did not show significantly higher degrees of breakdown than tests on rocks of comparable degree of weathering from other parts of the site. This confirms that the deterioration is initiated by the presence of pyrite, and that the physical state of the rock substance only serves to determine the type and degree of deterioration.

In many parts of the site area, rocks containing sulphides show no trace of deterioration. It is probable that the physical characteristics of these rocks, particularly the low absorptions, make them resistant.

A detailed examination was made of rockfill materials exposed by excavation of the Millbrook trunk main. The rock which consisted of slightly to moderately weathered schist, had been buried for approximately 40 years. No signs of fretting were observed, however a minor proportion of the rock fragments showed slight crystalline gypsum or powdery jarositic encrustations at the surface. These rocks contained pyrite mineralization which was also present in some unaffected rocks.

Engineering significance

It is evident that if rapid deterioration such as the fretting which occurs in adit No. 1, was to occur throughout the embankment, it would result in substantial settlement with consequent damage to the upstream face.

However the observations and tests that have been made suggest that rapid deterioration of rock substances within the embankment will be insignificant. The main considerations leading to this conclusion are:—

1. Rocks in which deterioration has been observed all contain sulphide minerals. Sulphide minerals occur only in some parts of the site and are absent from an estimated 70 per cent of the materials placed in the embankment.

2. Many of the rock fragments which do contain sulphide minerals would not be susceptible to deterioration.

3. Deterioration is a direct result of evaporation of moisture. This process is not likely to occur within the embankment, except in a narrow zone adjacent to the downstream face.

Examination up to 18 months after placement of rockfill near the surface of the downstream face, has not revealed any traces of rapid deterioration.

PERFORMANCE OF EMBANKMENT

Operational characteristics

Settlement. There are several ways in which settlements may occur:—

1. Compression of soil materials in the foundations.
2. Crushing of points of contact between rock fragments.
3. Erosion or compression of soil materials separating rock fragments.
4. Rearrangement of rock fragments with a consequent reduction in porosity.

It is considered that none of these effects are likely to occur to a significant extent, for the following reasons:—

1. Soil materials were removed from the foundations to make them virtually incompressible.
2. The loads within the bulk of the embankment are relatively low, and insufficient to cause significant crushing of the rock substances. Towards the base of the dam where higher loads do occur, stronger rock was used.
3. The sluicing of the rockfill during placing washed the soil materials into voids. The resulting fill consisting of rock fragments in contact, and soil materials in the voids, was observed in the ring density tests.
4. The vibratory rolling process resulted in a high degree of compaction. The porosity of the bulk of the dam ranges from 4 per cent to 20 per cent. Settlements measured to May, 1970 are less than 0.18 foot or 0.09 per cent of the height of the embankment.

Leakage. Two types of leakage past the embankment are possible:—

1. Through the concrete face and/or grout cap. No cracks in the face, or ruptures in the water stops separating the face slabs, have been observed. Shrinkage cracks occurred in most of the grout cap blocks, but appear to have been sealed during grouting operations.

2. Through the abutments and under the dam. Experience at numerous other sites suggests that grouting is seldom, if ever, completely successful and hence that some leakage of this type is likely to occur.

Up to May, 1970, a maximum reservoir head of 100 feet has produced only minor leakages up to 7.5 gallons a minute.

Drainage. Sluicing waters applied to the completed surface of zone 1 layers during construction, penetrated rapidly into the fill. In zone 4 drainage was less rapid, however surface ponding was not widespread or prolonged. It is therefore considered that if high leakages occurred in the concrete face, they would be readily drained through zones 1 and 4.

Vertical drainage in zone 3 is very slow due to the presence of numerous "skins" of low permeability. However it is considered that the later permeability of the layers is sufficient to drain any water leaking through the abutments, without significant build-up of internal water pressure.

Stability. Tri-axial tests have been carried out to find the shear strength of representative samples of rockfill materials.

The sample of "skin" material from zone 3 which is representative of the weakest material in the embankment, gave test values for shear strength parameters:—

Angle of internal friction (ϕ') 39.5 degrees
Cohesion (C') 12 lb/sq. in.

Samples of crushed schist and dolomite typical of zone 3 material were tested in a tri-axial cell by the Snowy Mountains Hydro-electric Authority under varying degrees of lateral pressure. The following values were obtained for the angle of internal friction within the range of confining pressures occurring within the embankment:—

	Angle of internal friction (degrees)	Cohesion (lb/sq. in.)
Schist	39-49	0
Dolomite	39	29

These values are all well above the minimum shear strength required for the embankment to be stable, with a safety factor of 1.2.

Comparison with other concrete-faced rockfill dams

The type of dam constructed at Kangaroo Creek has only recently become popular, due mainly to the substantial improvement in techniques for handling and placing rockfill. It is now widely used in situations where the lack of conveniently located source of core materials makes an earth and rockfill dam undesirable. Steele and Cooke (1960) and Wilkins (1968) review the history of the development of faced or decked embankments. A summary of the main features of some of these dams is given in Table 15.

TABLE 15

Concrete Faced Rockfill Dams

Name of dam	Year of completion	Slope of upstream face	Height	Method of placement	Leakage	Comments
			(ft.)			
Salt Springs (U.S.A.)	1931	1 on 1 to 1 on 1.4	328	Dumped rock	High	Leakage reduced by repairs to face concrete
Lower Bear River (U.S.A.)	1952	1 on 1.3	245	Dumped rock	Low	—
Paradela (Portugal)	1958	1 on 1.3	361	Dumped rock	High	Leakage reduced by repairs to face concrete
Exchequer	1966	1 on 1.4	500	Rock rolled in layers	High	Leakage reduced by extensive repairs to face concrete
Cabin Creek	1966	1 on 1.3	260	Rock rolled in layers	Low	—
Risdon Brook	1968	1 on 1.5	117	Rock rolled in layers	Low	—
Kangaroo Creek	1969	1 on 1.3	200	Rock rolled in layers	Not filled	—

The early embankments (prior to 1965) were constructed by dumping the rock from a height. The degree of compaction obtained by this method was substantially less than that obtained in embankments constructed after 1965 in which the rockfill has been compacted by rolling in layers. Consequently the settlement in the more recent dams has been much less than for the dumped rockfills, and there have been fewer problems with leakage due to cracking of the upstream face membrane, the main problem in these dams.

The high strength of the Kangaroo Creek Dam fill as determined from laboratory tests, and the very small settlements, compare very favourably with the strength and settlements of other dams (Wilkins, 1968; Sowers, Williams and Wallace, 1965). Schistose rocks have been largely ignored as sources of rockfill for major embankments, however the embankments where they have been used (Sowers, Williams and Wallace, 1965), have all shown comparatively minor settlements. The schists used at Kangaroo Creek are much weaker than rock materials commonly accepted for use in dams of comparable height. However it is this comparatively low strength which is the main factor contributing to the high densities obtained and the consequent high shear strength of the embankment. There is no doubt that the use of similar weak materials will be more widespread in the future.

EFFECTIVENESS OF EXCAVATION TECHNIQUES

A total quantity of almost 600,000 cubic yards of soil and rock material was excavated for the project. The excavation items accounted for more than 40 per cent of the total cost of the contracts. In order to assess their effectiveness and efficiency some aspects of the excavation procedures are discussed below.

Stripping of loose materials

The method of stripping adopted by the contractor depended on the slope of the ground surface and the distribution of loose materials and not on the nature of the surface materials. Table 16 summarizes the main methods in order of increasing efficiency, and the excavations in which they were used:—

TABLE 16

METHODS OF EXCAVATION OF SOIL MATERIALS

Method used*	Feature
1. Hand tools (pelican picks, picks, shovels, crowbars, wheelbarrows)	Grout cap (left and right banks)
2. Drag-line	Intake structure part of quarry No. 1
3. Small excavating machinery (back-hoe, front-end loader, etc.)	Spillway, quarries Nos. 1 and 2, Embankment foundation (part of left bank)

* The actual plant used is given in Appendix 1.

A considerable amount of inefficient stripping on the left bank could have been avoided if a programme of bulldozer stripping from top to bottom had been carried out in the initial stages of construction. However on the right bank the use of the less efficient techniques was dictated by the steep slopes.

Fragmentation*

The degree of fragmentation aimed at during excavation was governed by two opposing considerations:—

1. To produce a minimum of waste rock.
2. To produce rock of a grading which could be readily handled by the equipment available at any particular time.

In the smaller excavations the latter consideration was the more important but in the large excavations (spillway and quarry No. 1) it was necessary to compromise between the two considerations.

The following geological factors were found to have a considerable effect on the degree of fragmentation of the blasted rock:—

1. Size of unit blocks.
2. Strength and anisotropy of rock substance.
3. Presence of weak seams which tend to absorb much of the shock of a blast.
4. Orientation of geological structures with respect to the direction of drill holes.

The fragmentation also depended on the following aspects of blasting technique:—

1. The pattern of burden holes (angle, spacing etc.).
2. The quantity and distribution of explosive charge.
3. The type of explosive used.
4. The delays between firing of holes.

The wide variation of the geological factors at the site resulted in substantial differences in the degree of fragmentation between one part of an excavation and another. Attempts were made to overcome this problem by varying the blasting technique according to geological conditions. This approach proved successful in several instances, however in general it was limited by the complexities of the geological conditions. A more successful approach was the adoption of a standard technique developed by trial and error, which was found to produce acceptable fragmentation in most situations.

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* The term "fragmentation" includes the size (grading) and also the shape of the fragments.

The problem could have been more efficiently solved during the investigation stage, by a comprehensive programme of research by geologist and mining engineer, into blasting techniques most applicable to the site conditions. Such a programme in the form of one or more trial quarries, could include tests on drilling rates and bit wear, which with the other details of blasting technique would allow tenderers to make accurate cost estimates, and would minimize wasteful errors during construction.

The highly flaky shape of the rock fragments produced by blasting of weak schist materials, caused considerable concern due to the tendency of this material to break down during handling and placing. Fig. 54 shows this flaky material and Fig. 55 shows the more bulky fragments typical of the excavation of gneissic rock. It is considered that the shape of the fragments could have been improved with consequent decrease in fines content, if blast holes had been drilled near-normal to the foliation direction as shown in Fig. 85.

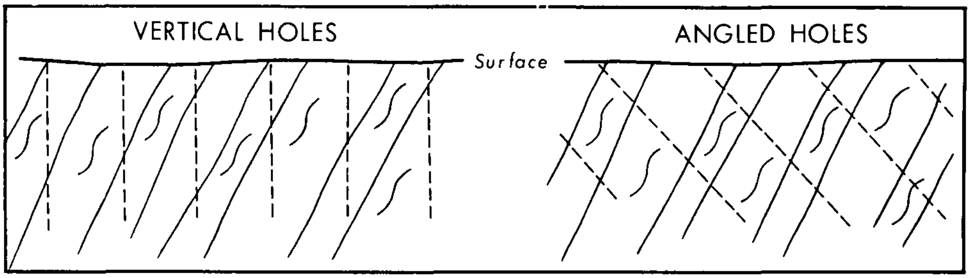

Fig. 85. Blast hole directions, showing how fragmentation of gneissic rock could have been improved.

Handling

The loading of blasted rock was carried out efficiently for small bench heights (less than 20 feet) and where steep batters were used. However in the higher benches with flatter batters such as spillway excavation, south wall, it was necessary to use D8 bulldozers to transport materials from the upper parts of the bench so that they could be loaded from below. The bulldozers were also used to trim the batters. These operations resulted in considerable breakdown of the rock materials, increasing the amount of waste rock and lowering the quality of rockfill.

This problem could have been reduced in two ways:—

1. A gradall machine could have been used to trim the batters.
2. Each bench which was higher than 20 feet could have been excavated in two vertical stages.

The latter method would have been expensive but would have resulted in a much higher yield of usable rock.

Wastage of rock

A total quantity of 190,000 cubic yards of material was sent to disposal from the spillway and quarry compared with an estimated total of about 130,000 cubic yards. The difference was mainly due to the contamination of otherwise suitable rock by soil and/or highly and completely weathered rock.

This contamination could have been reduced by the following procedures:—

1. More efficient stripping to remove all soil materials except those occurring in localized zones and patches.
2. Ripping and/or ripping with light blasting, in areas of highly and completely weathered rock which could not be removed by stripping techniques.
3. Direct loading of rock at the face, thereby avoiding the severe breakdown which occurs when the rock is subjected to repeated contact with excavation machinery.

Pre-splitting

Theory and practice of pre-splitting

The technique of pre-splitting has been developed during the past 10 years to control overbreak in rock excavation, and is now used for most permanent excavations in large civil engineering projects.

The theory of pre-splitting is briefly that when closely-spaced drill-holes are fired almost simultaneously, the stress waves surrounding each hole interfere causing a crack to propagate between the holes.

The advantages of pre-splitting are:—

1. The occurrence of blast-damaged rock in the walls of the excavation is almost completely eliminated. This contributes to greater stability, and vastly reduces the amount of hand-barring required for foundation excavations.
2. The amount of back-fill concrete required in foundations is reduced.
3. Irregularities which cause stress concentrations are avoided.
4. The comparatively smooth surface obtained is aesthetically pleasing.

The techniques of pre-splitting are described by Paine, Holmes and Clark (1962). Holes are usually spaced between one and four feet apart depending on geological conditions and the quality of pre-split required. At Kangaroo Creek a spacing of four feet was found quite satisfactory in quarry No. 1, but spacings as close as one foot were used in the intake structure and grout cap excavations.

The amount of explosive per drill hole used in pre-splitting is much less than for burden blasting. Powder factors between 0.5 and 1.5 pounds per square foot are commonly used. Stemming is used between the charges in the holes to distribute the explosive as evenly as practicable throughout the plane.

Results of pre-splitting on Kangaroo Creek Dam project

The pre-splitting technique was used to form the batters for most of the excavations involved in the project. Table 17 relates the direction of the batters and the relevant geological factors, to an empirical assessment of the quality of the pre-split at each location.

TABLE 17

QUALITY OF PRE-SPLIT SURFACES

Locality	Direction of batter		Angle to mean foliation direction	Predominant rock types	Chemical weathering	Mechanical weathering	Pre-split quality index *	Notes on overbreak
	Trend	Plunge						
	(degrees)	(degrees)	(degrees)					
Spillway, north batter	199	60	64	Schist and gneiss	Mainly slight	Severe to slight	5 (3 at down-stream end)	—
Spillway, south batters 2 and 3	019	45	28 to 32	Schist and gneiss	Mainly moderate	Slight	4	
Spillway, south batter 4	019 to 026	45	26 to 30	Schist and gneiss	Mainly slight	Slight	4	—
Spillway, south batters 5 and 6	019 to 026	45	28 to 32	Schist and gneiss	Slight to fresh	Slight	4	—
Spillway, channel upstream batter	297	60	64	Schist	Fresh to slight	Slight	5	—
Quarry, south batters 1 and 2	320	76	47	Gneiss and schist	Mainly moderate	Slight	5	Some overbreak along sheet joints in upper 10ft.

TABLE 17—QUALITY OF PRE-SPLIT SURFACES—*continued*

Locality	Direction of batter		Angle to mean foliation direction	Predominant rock types	Chemical weathering	Mechanical weathering	Pre-split quality index *	Notes on overbreak
	Trend	Plunge						
	(degrees)	(degrees)	(degrees)					
Quarry, batters 3 and 4	320	76	47	Gneiss and schist	Slight to moderate .	Slight.........	5	Some overbreak along sheet joint in upper 5ft.
Quarry, batters 5 and 6	320	76	47	Gneiss and schist	Fresh to slight	None	5	—
Quarry, downstream batters	007 to 045	63	17 to 34	Gneiss	Moderate to fresh..	Slight to severe	1 and 4	Batters 4 and 6 have broken mainly along Set (1) joints
Access road, north batter	119	71	55	Gneiss	Slight.............	Slight.........	5	—
Access road, downstream batter	086	71	22	Gneiss	Slight.............	Slight.........	1	Batter has broken almost entirely along Set (1) joints
Intake structure vertical walls	112	90	40	Gneiss	Mainly slight to moderate	Severe to slight	5—2	Upstream wall broke partly along Set (2) joints. Downstream wall broke partly along Set (1) and sheet joints
Intake structure floor	202	46	77	Gneiss..........	Slight to fresh	Mainly slight ..	5—2	Minor overbreak occurred along sheet joints
Knob excavation	194	50	71	Gneiss and granitic gneiss	Slight to fresh	Mainly slight ..	5—2	Minor overbreak occurred along sheet joints

* Pre-split quality index—

1. Breakage is mainly along one or more joints behind the plane of drill holes. Less than 20 per cent of drill holes are visible.
2. Face contains localized areas of overbreak along joints but more than 60 per cent of drill holes are visible.
3. Breakage is mainly along joints close to the plane of the batter. Most of the drill holes are visible.
4. Breakage between drill holes is concave or convex but there is less than 10 per cent overbreak along joints.
5. Planar breakage between holes. Less than 10 per cent overbreak.

Fig. 86. Part of batter for quarry No. 1, bench 3. Pre-split is almost planar except towards the top of the batter where some overbreak has occurred.

It is evident that the main geological features affecting the quality of the pre-split surfaces are the S1 foliation and joints and seams in the same direction. Defects in other directions were responsible for minor overbreak. Chemical weathering of the rock substance does not significantly affect the quality of the pre-split surface and satisfactory pre-splits were obtained even in zones of severe mechanical weathering (see Fig. 51).

In general, all pre-split batters were successful, except those where the batter direction was almost parallel to or within 25 degrees of the main S1 foliation direction in the rock, in which case overbreak usually occurred to the nearest near-parallel joint or seam behind the face. In those batters where the angle to the foliation was more than 40 degrees the pre-split followed an almost planar path between holes (Fig. 86). However, where the angle to the foliation was between 25 and 40 degrees, the pre-split followed a zig-zag path, partly along the foliation and partly across the fabric. This effect was most pronounced in those cases where the pre-split holes were drilled within the plane of the foliation (Fig. 87). These batters were quite satisfactory from the view-point of stability but relatively unsatisfactory for foundation surfaces, as extra concrete was required to fill the irregularities.

Fig. 87. Part of the upslope (south) wall of the spillway excavation. The pre-split surfaces are uneven, due partly to overbreak along joints and partly to V-shaped breakage between holes.

Deviation of blast holes

Much of the inaccuracy of pre-split planes is attributable to the deviation or "wandering" of drill holes. This phenomenon occurs to some extent in all holes drilled with rotary/percussion drills such as the Air-trac machines used at Kangaroo Creek. The deviations resulted in substantial overbreak and underbreak in many of the batters at Kangaroo Creek Dam, necessitating in some cases the use of extra concrete and in others, extra excavation at the toes of the batters. In the lower benches of the spillway, the holes were steepened in the belief that they would flatten out, however they tended to steepen, resulting in a substantial volume of extra concrete in the spillway channel wall.

In order to understand the nature of the deviations a survey of blast hole orientations was carried out. The orientations were measured at the top and bottom of each hole, and the deviation index which is defined as the angular deviation per 100 feet of hole, was calculated. The results of these measurements are given in Appendix 7, and are summarized in Fig. 88.

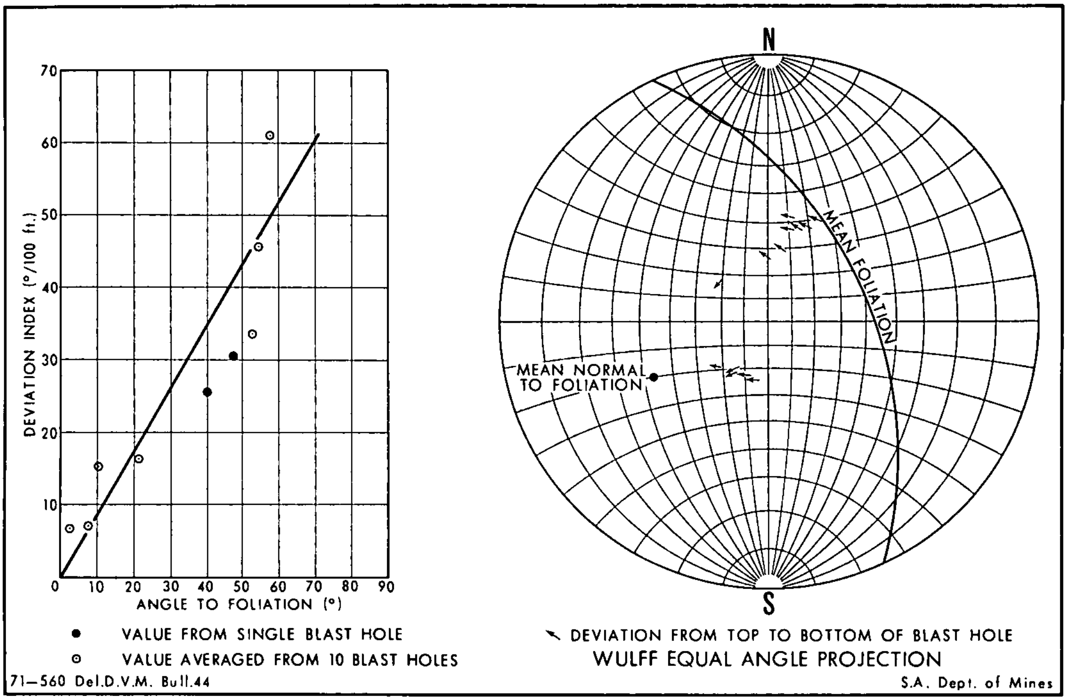

Fig. 88. Deviation of blast holes; stereographic projection. Graph showing relation between foliation and deviation of blast holes.

It is shown on Fig. 88 that the deviation tends towards the pole of the foliation or foliation normal and that the curvature as given by the deviation index is proportional to the angle between the drill hole and the foliation. Although the broad trends are evident from this work, a considerable amount of further work of this type is necessary to establish definite relationships and to determine the characteristics of blast hole deviations in other rock types. Such work would provide a basis for the accurate prediction of the deviation to be expected for any particular hole direction at any site. Research is also necessary into the effects of various drilling factors such as pressure, bit sharpness, rate of penetration, *etc.* The results to date suggest that these factors are not of prime importance, however they may explain the wide scatter of results.

The phenomenon of drill hole deviation has also been observed in diamond drilling, and the nature of the deviations appears to be very similar for both types of drilling. A theory for the mechanism of deviation of diamond drill holes which effectively explains why holes tend toward the foliation normal, has been postulated by Pritchard-Davies (1970).

Briefly the theory is that as the drill bit penetrates obliquely through a band of weak rock to a band of strong rock, a pivotal movement occurs in the weak rock. However the opposite pivotal movement that tends to occur when the drill penetrates through strong rock into weak rock, is partly restrained by the stronger rock and thus the deviations do not cancel out.

Hence in a foliated rock with numerous hard and soft bands a deviation tends to develop away from the foliation direction. This theory applies equally well to blast hole drilling.

Summary of observations

Pre-splitting is a relatively simple technique for producing batters of high quality in most geological circumstances. The following observations emerge from a study of the pre-split surfaces at Kangaroo Creek Dam:—

1. The quality of a pre-split surface depends to a large degree on its orientation with respect to geological structural features such as foliation, jointing and bedding, *etc.*
2. The quality of a pre-split also depends, but to a far less degree, on the spacing and charging of drill holes.
3. The quality of a pre-split is almost independent of chemical weathering effects and of rock type, except for anisotropic rocks.
4. Pre-splitting is usually unsuccessful if the desired pre-split plane intersects the foliation or any other closely spaced planar weakness in the rock, at an angle less than 25 degrees.
5. The deviation of drill holes is a major cause of inaccuracy in pre-split planes. The approximate amount and trend of the deviation may be calculated in advance so that corrections to the initial drilling angles can be made to give the required overall batter angle.

Rock-bolting

The technique of rock-bolting has been developed during the past 15 years for temporary and permanent stabilization of rock faces. The theory and practice is comprehensively described by Pender, Hoskings and Mattner (1963) and Burman (1969). Rock bolts are commonly installed for two types of conditions:—

1. To attach or pin individual blocks of rock, which are partly detached from the main rock mass. One or two bolts are usually sufficient in these conditions.
2. To reinforce a zone of disturbed rock adjacent to an excavation and to bind it to the relatively unaffected rock mass. In such cases a pattern of bolts is chosen according to the following factors.
   (1) The orientation of geological weaknesses.
   (2) The size of unit blocks.
   (3) The height of the excavation.
   (4) The presence of stress concentrations in the rock mass.

Table 18 summarizes the use of rock bolts at Kangaroo Creek Dam. As far as possible the length, direction and pattern of bolts was determined according to the empirical rules devised by Pender, Hoskings and Mattner (1963). All bolts were installed as soon as possible after excavation.

Wherever possible they were each tensioned to a torque of 250 foot-pounds. In the highly disturbed rock mass in the access road excavation and in parts of the intake structure excavation, the torques achieved were less than 150 foot-pounds.

TABLE 18

USE OF ROCK BOLTS

Locality	Number of bolts	Length of bolts (feet)	Reason for bolting	Notes
Intake structure excavation and foundation	44	6, 8, 10, 12	To stabilize disturbed rock underlain by sheet joints	Difficulties in tensioning some bolts. Grouted
Diversion tunnel ..	14	6, 8	To pin blocks of rock above seams intersecting the roof of tunnel	Grouted
	78	8...........	To form a pattern to reinforce the jointed rock mass to resist high external pressures during construction	Not grouted 4ft. x 4ft. pattern
Knob excavation	12	8, 20	To stabilize large joint blocks underlain by sheet joints	Grouted
Grout cap excavation (right bank)	31	8, 10, 12, 15	To stabilize large joint blocks underlain by sheet joints, and to reinforce the jointed rock mass in areas where the vertical batter was more than 25ft. high	Some of the bolts were installed in a 10ft. x 10ft. pattern. Grouted
Right abutment block excavation	11	8, 10, 20	To stabilize disturbed joint blocks during excavation	Grouted
Access road cutting	28	6, 8, 10, 12, 15	To stabilize highly disturbed rock mass below Gorge Road deviation	Difficulties in achieving tension and in grouting
Spillway ski-jump north wall foundation	5	10, 20	To reinforce the loose blocks above several sheet joints	Grouted
Spillway ski-jump floor foundations	16	8, 10, 20	To stabilize large disturbed joint blocks	Grouted
Access shaft excavation	20	8, 10, 12, 15.	To stabilize disturbed rock underlain by sheet joints	Grouted
Access shaft	27	8...........	To support highly stressed rock at junction of shaft and tunnel	4ft. x 4ft. pattern. Grouted

All bolts were grouted except those installed in the upstream portal area of the diversion tunnel, which were only required to give temporary support. Difficulty in the grouting of bolts in open-jointed areas required the use of caulking materials, and thickening of the grout mix.

None of the rock-bolted areas has shown any signs of movement since installation of the bolts, and it is considered that these areas are stable under operating conditions. However, there is slight doubt as to the long term stability due to the possibility of corrosion of the bolts over long periods.

A total number of 286 rock bolts with a total length of 2,713 feet were installed on the project. Of these bolts approximately 25 per cent were not required for permanent support and hence were not grouted.

THE VALUE OF FOUNDATION GROUTING

Although the grouting results show several broad trends from one part of the rock mass to another, many of the leakages are difficult to explain in terms of the geological conditions. The average grout take for the entire grouting programme, of 0.03 bags per foot of hole, was very low considering the highly disturbed nature of parts of the foundation. This was undoubtedly due to the fact that most of the rock separations contained infilled clay materials. Most of the significant takes which did occur were in the lower stages of the holes. This is thought to be partly due to the increased pressures used in these stages but mainly because less infilling had occurred at these depths.

Moderate takes which occurred in the foundations below the river bed grout cap blocks E and F, reflect the absence of infilled clay from geological weaknesses in this area. Seams exposed in the foundations of these blocks were mainly sand-filled.

The highest takes occurred adjacent to the two abutment blocks and it is considered that most of this grout found its way into the open-jointed areas which occurred in the foundations of these blocks. In the right abutment block and in adjacent grout cap block K, the grout curtain was angled into the abutment to avoid the open-jointed area, as shown on page 84. The low grout takes in this area indicate that the change in design was highly successful.

On the left bank, grouting of the downstream row of holes in grout cap blocks A, B, C and D, was completed before grouting of the upstream rows five feet away, was commenced. The results are summarized in Table 19.

TABLE 19

RESULTS OF GROUTING DOWNSTREAM ROWS OF HOLES IN GROUT CAP BLOCKS A, B, C AND D

Block	Grout take in downstream row Bags	Grout take in upstream row Bags
A	31	10
B	18	8
C	18	8
D	20	14
Total	87	40

These figures suggest that the grout "curtain" formed by a single row of holes was incomplete, and it is therefore considered that the second row of holes was well-justified.

Although the takes recorded from the majority of holes were small (a quarter to a full bag), the grout injected was probably very significant in reducing leakage by sealing small openings adjacent to clay seams. Openings of this type are subject to considerable enlargement by piping of the soil materials under the high operating pressures. A take of 0.5 bags of cement could fill an opening one millimetre wide over an area of approximately 80 square metres (1,000 square feet).

Prediction of grouting results on the basis of geological conditions, proved to be a difficult task. In two parts of the grout curtain geological considerations resulted in variations to the arrangement of grout holes as designed. In grout cap blocks C and D, additional "consolidation holes" were drilled specifically to intersect joints which were near-parallel to the primary and secondary holes. In the spillway overflow lip block LC, additional holes were drilled to provide extra intersections of the major seams which occur in the foundation. In both these areas the grout takes recorded from the extra holes were small, due presumably to the presence of infilled clay.

Seventeen core holes were drilled through the rock mass close to the line of grout holes. Details of the holes are given in Table 20.

TABLE 20

CORED HOLES IN VICINITY OF FOUNDATION GROUTING

Hole No.	Area	Depth into rock	Total depth	Distance from nearest grout holes	Grout recovered	* Rate of infiltration after filling
		(feet)	(feet)	(feet)		(gal. per min.)
KA39A	Grout cap, block D	43	45	8	None	—
KA40	Grout cap, block C	42	45	5	None	—
KC41	Left abutment, block LF	20	36	4	None	0.86
KC42	Left abutment, block LF	20	33	1	None	2.25
KC43	Left abutment, block LF	20	35	3	None	0.19
KC44	Left abutment, block LF	20	34	1	None	0.12
KC45	Left abutment, block LF	20	39	2	Seam of grout 8 mm wide, 6ft. into rock. Coating at 10.5ft.	0.07
KC46	Spillway overflow lip (block LE)	20	34	5	None	0.39
KC47	Spillway overflow lip (block LE)	20	34	5	Seam of grout 12 mm wide, 4ft. into rock	0.44
KC48	Spillway overflow lip (block LE)	20	33	6	None	0.66
KC49	Spillway overflow lip (block LE)	20	32	5	None	1.18
KC50	Spillway overflow lip (block LE)	20	31	2	None	0.32
KC51	Spillway overflow lip (block LE)	20	34	2	Coating of grout up to 2 mm thick, 19ft. into rock	0.06
KC52	Spillway overflow lip (block LE)	20	33	3	None	0.59
KC53	Spillway overflow lip (block LE)	20	32	6	None	Could not fill
KC54	Spillway overflow lip (block LE)	20	28	5	None	0.59
KC55	Spillway overflow lip (block LE)	20	26	6	Coating of grout 1 mm thick, 8ft. into rock	0.56

* The groundwater level in the holes at the time of testing was approximately 25 feet below the floor of the gallery.

The two cored holes through the grout cap were drilled using "NMLC" coring equipment, and resulted in almost 100 per cent core recovery. No grout was recovered from either of these holes.

The fifteen cored drain holes from the drainage gallery were drilled using "NX" coring equipment, and resulted in 95 per cent core recovery. The material recovered in the core confirmed the disturbed, open-jointed nature of the rock mass in this area. Detailed examination of the core revealed several seams and coatings of grout. However many of the partly open joints in the core did not show any traces of grout despite the close proximity of grout holes.

The most likely explanation for the absence of grout from many open joints is that it was eroded by drilling water. This suggests that the grout could also be vulnerable to erosion by reservoir water during operation of the dam.

The drain holes were tested by filling and measuring the rate of infiltration. Several holes showed moderate leakage rates and hole KC53 could not be filled. Periscope observation of the sides of this hole revealed the presence of two joints open up to two centimetres at a depth of 14.5 feet or approximately four feet into rock.

The following two possible methods of approach to the problem of possible leakage into drain hole KC53 were considered:—

1. Leave the drain hole untreated and observe the leakages as the reservoir fills. If leakage rates are low and if the water is clear (*i.e.* contains no suspended soil materials) then no treatment is necessary.
2. Inject cement grout into drain hole KC53 to fill the gaping joints which intersect the hole at a depth of 14.5 feet. This would impair the function of this hole as a drain and may also reduce the effectiveness of adjacent holes.

   A packer should be located immediately below the open joints, and a thick grout mix injected under low pressure from the hole collar.

The latter approach was adopted.

EFFECTS OF FILLING THE RESERVOIR

Stability of slopes

The bedding of the Torrensian rocks has resulted in the formation of dip slopes on the west side of three gullies towards the southern end of the reservoir. These dip slopes show the effects of numerous slides, deep-seated and superficial.

The gully adjacent to the dam contains numerous old landslides and several relatively young ones in which movements have occurred within historical times. A large apparently deep-seated slide located near the head

Fig. 89. Large landslide at the head of the gully to the south of the dam. The disposal area in the foreground is mainly above full supply level and supports the toe of the slide.

of this gully just above the reservoir full supply level (Figs. 5 and 89) may contain up to three million cubic yards of disturbed material. Several soil scarps up to four feet high reflect the occurrence of recent superficial movements. Measurements carried out during construction indicate that the slide mass was stationary during this period. The base of the slide movements probably coincides with a seam along the bedding, however it is possible that initial movements were caused by seismic shocks related to the nearby Kitchener Fault.

A quantity of approximately 200,000 cubic yards of waste rock which was placed at the toe of the slide should prevent further movement of the mass.

The gully in which Kangaroo Creek is located (Fig. 5) also contains several slides on the west bank. These slides appear to be due to movement along bedding planes, however it is possible that collapse due to solution of calcareous rocks also contributed to the instability.

The stability of slopes on the left bank immediately upstream of the dam, is discussed in detail on pages 141 to 150. On the right bank immediately above Batchelor's Bridge there are several indistinct scarps which probably reflect the occurrence of ancient slide movements. The nature of this sliding has not been determined but the most likely explanation is that movements have occurred along flat-lying seams near river-bed level.

Of the slides described above, that at the head of the gully to the south of the dam is the only one so large that further sliding could create a wave capable of overtopping the dam. The reservoir level will not reach the toe of this slide, but it will have the effect of raising the groundwater table in the area. It is considered that the extra stability provided by the large mass of waste rock at the toe of the slide more than compensates for the effect of raising the water table.

Leakage through reservoir rim

Observations of groundwater in drill holes and excavations at the site and in parts of the reservoir area, indicate that the pre-construction groundwater table sloped towards the Torrens Valley. The occurrence of seepages in the valley walls well above river level suggests that for large parts of the year the level of groundwater in the reservoir rim was above the proposed full supply level of the reservoir. Experience at other dam sites has shown that under these conditions, significant leakage from the reservoir is unlikely.

The shortest leakage path through the reservoir rim is through a saddle in the southwest corner to a gully on the western side, a distance of 2,800 feet at full supply level. Significant leakage along this path is considered unlikely, as the water would have to flow through a succession of rock types, including substantial thicknesses of slates which have a characteristically low permeability.

The dolomitic rocks which occur on the eastern side of the saddle were investigated during the feasibility stage by a 450 feet long drill hole, which indicated the presence of numerous small solution cavities. However, the dolomite does not intersect the ground surface below full supply level outside the reservoir, and hence although some leakage of water may occur as the reservoir fills, the water would return to the reservoir during draw-down.

THE CONCRETE ARCH DESIGN IN RETROSPECT

The design stage investigations for the arch dam (Stapledon, 1966) indicated that the rock mass on the left bank is intersected at intervals of five feet to 20 feet by seams of compressible clay, some of which are up to 15 cm wide.

The arch design was finally rejected after consideration of two possible causes of failure:—

1. Consolidation of seams in the rock mass due to the thrust of the dam.
2. Movement in a downstream direction of wedge-shaped blocks (obtuse angled, open wedges), due to a component of the arch dam thrust which acts in this direction.

It was realized at this stage that the exploration had revealed only a very small sample of the rock mass and that this sample was not necessarily typical of the rock mass as a whole. In addition, the three-dimensional geological picture contained considerable interpretation. The possibility of other undiscovered defects occuring in the area also influenced the decision to reject the arch design.

Geological conditions revealed during construction

The excavations for the grout cap and spillway confirmed the great depth of mechanical weathering effects as revealed by the earlier investigations. The excavations intersected several large infilled clay seams up to 40 cm wide which were considerably thicker than any intersected by the exploration.

Detailed examination of the left bank grout cap foundations between RL720 and RL870 revealed a total true thickness of 100 cm of compressible soil seams of which 70 cm consisted of highly plastic infilled clay (CH).

Detailed examination of the larger seams revealed that infilling had occurred following small movements of wedge-shaped rock masses in a downstream direction (see pages 143 to 147).

Excavations for the intake structure, grout cap and right abutment block indicated the presence of a zone of severe mechanical weathering on the right bank up to more than 25 feet thick. Within this zone the rock mass contains numerous gaping sheet joints, some of which are open to 20 cm, and infilled seams up to 15 cm wide.

Discussion of concrete arch versus rockfill dam

The geological conditions revealed by the construction stage investigations confirmed the correctness of the judgment that the site was unsuitable for a concrete arch dam.

The successful construction of an arch dam at the site would have required one of the following procedures.

1. Excavation of the foundation rock in the form of a slot or trench to remove mechanically disturbed rock, and replacement of the excavated rock by concrete in the form of a pulvino. This would entail the removal of a thickness of up to 80 feet measured normal to the surface of rock on the left bank and 25 feet on the right bank. Experience with smaller excavations of this type has shown that the shape of the excavation is very difficult to control, particularly in directions close to that of the rock foliation. An excavation of this magnitude on the left bank would be highly dangerous and would require a large programme of support during construction, particularly on the downstream side.

Such a construction programme would have been highly expensive and very difficult to estimate economically with reasonable accuracy. Furthermore, even with such large foundation excavations, the possibility of failure by wedge sliding of part of the left abutment, would still remain.

2. Moderate excavation to remove the most severely disturbed rock and an extensive programme of treatment to reinforce the foundation rock mass. The most likely forms of treatment would be the installation of post-stressed anchored cables, and the removal by washing and replacement by grout, of seam materials.

Apart from the expense and practical difficulties associated with these techniques, their effectiveness is suspect, particularly in the long term. Experience with grouting at the site indicates that the replacement of the non-dispersing seam materials by grout would be at best very difficult, and probably impossible.

The most likely type of failure for a concrete arch dam at the Kangaroo Creek Dam site would be one in which the abutments could not develop sufficient reaction to the thrust forces exerted by the dam. This could be due to compression of the foundations or to displacement of foundation rock in the direction of thrust. Either would cause rupture of the concrete arch and catastrophic failure. This would be particularly disastrous at the Kangaroo Creek Dam site, located as it is, upstream of the city of Adelaide.

The fact that construction of the rockfill dam in a form essentially as designed, was achieved without any serious problems, confirms that the design was appropriate to the site.

Unlike concrete arch dams, rockfill dams are not susceptible to catastrophic damage due to foundation failure. The main problem to which concrete faced rockfill dams are susceptible, is leakage resulting from rupture of the concrete face due to settlement of the fill. However, this can be readily recognized and repaired before the damage becomes catastrophic.

Rockfill dams may also be severely damaged by overtopping due to blockage of the spillway, or a wave induced by a rockslide in the reservoir. However, measures were undertaken as part of the construction programme to ensure that there is no significant risk of such events.

To sum up, the large amount of foundation treatment and/or excavation which would have been required for an arch dam, would have been considerably more expensive than the cost of construction of the rockfill dam. The excavations involved for the arch dam would also have caused major safety hazards during construction, whereas the excavations associated with the rockfill design have been comparatively safe. Finally, the geological situation is such that even with extensive excavation and/or foundation treatment, the stability of an arch dam could not be completely assured. This fact alone provides ample vindication for the rejection of the concrete arch dam in favour of the rockfill dam.

GENERAL CONCLUSIONS

Even the most comprehensive programme of design stage investigations will not reveal all the geological factors and weaknesses which could influence the construction of a large project. In most cases the investigations show the approximate nature and distribution of geological substances and weaknesses at a site, however it is necessary to realize that the geological exploration reveals only a very small sample of the total soil and rock masses. This was particularly the case with the Kangaroo Creek project in which most of the exploration was for the concrete arch dam investigation and there was insufficient time to carry out a systematic investigation of the rockfill dam site. For this reason some of the geological conditions revealed during construction were outside the ranges which were assumed in the design stage. In many cases this necessitated modifications to the design of parts of the structures.

As many of the factors which influence the design and construction of large civil engineering projects such as the Kangaroo Creek Dam are related to the geology, it is important that the construction team for such a project includes an engineering geologist or someone with geological insight. The role of the geologist is to ensure that the engineers understand the relevant geological conditions and their significance, and to inform them where geological conditions differ from those assumed in the design.

In the Kangaroo Creek Dam project, geological considerations resulted in modifications to the design of many features including grout cap, spillway channel, right abutment block and grout curtain. In addition, geological information was extensively used in assessing the quality of foundations, stability of batters, and standard of rockfill, which comprised most of the day-to-day problems of the project.

The detailed record of the geology proved useful in interpretation of the grouting results and in assessing the basis of contractual claims. These records would also be invaluable in the event of failure of any of the structures.

The experience gained from construction of the project was extremely valuable to the author. In the first place the contact with the construction team resulted in an increased understanding of engineering principles and methods. Secondly, the detailed examination of large exposures has led to a better understanding of the nature and behaviour of rock masses and geological processes. The resulting "feedback" will be of direct benefit in geological investigations for other projects, particularly in interpreting the significance of geological features where the exposures are limited. The information will also contribute to improvements in the design and contract specification of future dams.

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Appendix 1
PROJECT OPERATIONS

SCHEDULE OF MAIN OPERATIONS

Operation	Date commenced	Date completed
Diversion tunnel excavation	26th August, 1966	24th October, 1966
Intake structure excavations	31st July, 1967	4th December, 1967
Spillway excavation	Early March, 1968	27th June, 1969
Spillway concrete	28th April, 1969	Late August, 1969
Quarry No. 1 excavation	1st March, 1968	Late April, 1969
Quarry No. 2 excavation	25th February, 1969	Late April, 1969
Grout cap excavation	26th February, 1968	January, 1969
Grout cap concrete	28th April, 1969	19th March, 1969
Embankment	Mid August, 1968	August, 1969
Upstream face concrete	April, 1969	Late August, 1969
Closure of tunnel plug		8th September, 1969
Official opening of project		5th December, 1969

CONTRACTORS

Diversion tunnel—Citra Australia

Intake structure excavations—McMahon Constructions

Embankment, spillway, *etc.*—Dumez (Aust.)

CONTRACTORS' WORK FORCE ON SITE

1. Professional

   Project engineer, one to three civil engineers, one mechanical engineer and two surveyors.

2. Clerks, tradesmen and unskilled workmen.
   Mainly from 60 to 120 employed.

SUPERVISING STAFF ON SITE

1. Professional

   Resident engineer, assistant resident engineer, resident geologist, one to two civil engineers, engineering assistant (chief inspector), two surveyors and an engineer (quantity calculations).

2. Inspectors, assistants, *etc.*

   Five to seven inspectors, one to two concrete testing officers (part-time), one geological field assistant, one courier, one to two surveyor's assistants, one typist/clerk and one draftsman.

PLANT AND EQUIPMENT USED BY CONTRACTOR FOR EXCAVATION AND PLACING OF ROCK

Category	Item	Feature
Excavation	3 x Gardner Denver air-trac drills with 600 cu. ft./min. compressors	Spillway Quarry Zone 1 quarry
	5 x pneumatic hand drills	Grout cap, intake structure
	2 x air legs	Grout cap, diversion tunnel
	1 x pavement breaker	
	1 x 71B Ruston Bucyrus face shovel (4.25 cu. yd.)	Spillway, quarries Nos. 1 and 2
	1 x 988 front-end loader (5.5 cu. yd.)	Spillway, quarries
	1 x 977 front-end loader crawler type (4.5 cu. yd.)	Spillway, quarries
	1 x 966 front-end loader (3.5 cu. yd.)	Zone 1 quarry
	2 x front-end loader backhoes	Embankment foundations
	1 x D4 Caterpillar bulldozer	Spillway, embankment zone 1
	2 x D8 Caterpillar bulldozer	Spillway, quarries
	1 x Caterpillar grader	Haul roads
	1 x P and H crane, fitted as a dragline	Quarry No. 1
Hauling equipment	4 x haulpak tip trucks (27 cu. yd.)	Spillway, quarries Nos. 1 and 2
	3 x Euclid tip trucks (21 cu. yd.)	Spillway, quarries Nos. 1 and 2
	2 x 12 ton International tip trucks	Zone 1 quarry
	1 x 7 ton International tip truck	Zone 1 quarry
Placing equipment	1 x D8 Caterpillar bulldozer	Embankment zones 2 to 4
	1 x D7 Caterpillar bulldozer	Embankment zones 2 to 4
	1 x large monitor mounted on D7 bulldozer as above	
	2 x small hand monitors	Embankment zone 1
	1 x Pacific 9.7 ton vibrating roller—connected to 6 inch deisel pumps at stilling basin	Embankment—all zones

Appendix 2

PETROLOGICAL DESCRIPTIONS OF SOME TYPICAL ROCK SUBSTANCES

FRESH ROCK SUBSTANCES

Sample 1. *Gneiss—(Site area—Diversion tunnel)*

Hand specimen—The rock is composed of bands five to 20 millimetres wide consisting mainly of coarse-grained quartz and feldspar, separated by bands up to 20 millimetres wide of very fine-grained, foliated minerals. Overall appearance of the rock is dark greenish-grey. Maximum size of mineral grains is eight millimetres.

Thin section—Coarse-grained bands consist of quartz (40 per cent), feldspar (30 per cent) and fine-grained sericite (30 per cent). The feldspar crystals contain numerous microfractures which are filled with fine-grained sericitic material, and are optically strained. Chlorite occurs in fibrous aggregates elongated along foliation. Chlorite fibres commonly orientated at an angle to the foliation. The fine-grained bands consist of aligned fine-grained sericite, chlorite and quartz with some coarser chlorite and quartz. The foliae are bent around irregularities in the coarser bands. Pyrite in crystals up to 0.3 millimetre constitutes 1 per cent of the rock.

Sample 2. *Schist—(Site area—Spillway channel)*

Hand specimen—The rock consists of fine-grained micaceous minerals with a well-developed foliation. Foliation planes have a soapy appearance. Medium and coarse-grained quartz crystals occur in places, often in bands but more commonly separated by fine-grained material. Maximum size of mineral grains is two millimetres. Overall appearance of rock is light grey.

Thin section—The bulk of the rock consists of fine-grained sericite and chlorite with porphyroblasts of quartz and remnant feldspar. Coarser grained quartz crystals which constitute 15 per cent of the rock, have been highly stressed and fractured. The fractures are filled with sericite. Occasional grains of pyrite and biotite also occur.

Sample 3. *Granitic gneiss—(Site area—Diversion tunnel)*

Hand specimen—The rock consists mainly of coarse and very coarse-grained quartz and feldspar, and has a granitic texture. Schistose stringers of chlorite occur in places. Numerous irregular veins of quartz less than one millimetre in diameter also occur. The maximum diameter of mineral grains is 20 millimetres. The overall colour is greenish-white.

Thin section—The rock consists of 40 per cent quartz, 40 per cent feldspar (mainly oligoclase) and 20 per cent sericite. The quartz and feldspar are mainly coarse-grained and contain numerous microfractures. The sericite occurs around the boundaries of feldspar crystals and filling cracks. Traces of chlorite occur in places.

Sample 4. *Quartz gneiss—(Zone* 1 *quarry)*

Hand specimen—The rock consists of medium to fine-grained quartz, feldspar and chlorite. Two distinct foliations are present. One consists of the parallel alignment of chlorite flakes and the other is a coarse layering of lighter (quartz rich) bands alternating with darker (chlorite rich) bands. Layers are 20 to 40 millimetres wide. Overall appearance of rock is grey, speckled. The maximum diameter of mineral grains is eight millimetres.

Thin section—The rock consists of 60 per cent quartz, 20 per cent feldspar, 10 per cent chlorite and 10 per cent sericite with occasional crystals of tourmaline, garnet and hematite. The sericite occurs surrounding and penetrating the feldspar crystals. The quartz crystals are elongated parallel to the alignment of biotite crystals and there is also some quartz/feldspar layering in this direction. (The other foliation was parallel to the plane of the slide and hence not observed.) Quartz and feldspar crystals are interlocking.

Sample 5. *Dolomite—(Quarry No.* 2)

Hand specimen—The rock is crystalline with a medium-grained "sugary" texture. The maximum grain size is 1.5 millimetres in diameter. The overall colour of the rock is translucent light grey-brown.

Thin section—The rock consists of sub-angular grains of quartz (25 per cent) and feldspar (10 per cent) in a matrix composed of interlocking fine- to medium-grained, re-crystallized dolomite. Boundaries of quartz and feldspar grains are pitted, but microfractures are rare. No bedding visible. Contains vein of calcite.

WEATHERED ROCK SUBSTANCES

Sample 6. *Schist—Completely weathered*

Hand specimen—Consists mainly of fine-grained sericite, limonite and medium-grained quartz. Foliation distinct. Outlines of original feldspar crystals visible. Colour—pale grey and yellow-brown.

Thin section—Consists mainly of felted groundmass of sericite and clay up to 0.1 millimetre diameter (silt). Approximately 50 per cent of this groundmass is stained with limonite. Quartz ranging in grain size from 0.1 to two millimetres comprises 30 per cent of sample. Most of the grains contain numerous micro-fractures. Limonite and goethite comprise 20 per cent of sample and occur in numerous veins mainly less than 0.2 millimetre in diameter, and orientated mainly along the foliation. Vague outlines of original feldspar crystals are common, but feldspar is completely decomposed to sericite.

Sample 7. *Schist—Highly weathered*

Hand specimen—Consists mainly of very fine-grained sericite and fine-grained quartz. Flaky texture. Colour—mainly pale grey with some brown patches.

Thin section—Consists mainly of felted, fine-grained sericite and clay, with 15 per cent highly microfractured quartz grains up to 1.5 millimetres in diameter. Microfractures also penetrate the groundmass and are open to 0.2 millimetre. Slight limonite staining. Outlines of weathered feldspars are common. Fine-grained opaques (not sulphides or hematite) form 2 per cent of sample.

Sample 8. *Gneiss—Moderately weathered*

Hand specimen—Bands up to 20 millimetres wide consisting mainly of quartz and weathered feldspar grains up to four millimetres diameter, alternate with schistose sericitic bands. Colour—pink to grey. Texture—granular to flaky.

Thin section—Alternating coarse and very fine-grained (lepidoblastic) bands. Quartz and feldspar crystals up to 10 millimetres wide in coarse bands contain numerous fractures filled with limonite—up to one millimetre wide. Feldspars are mainly decomposed (20 to 80 per cent) to sericite. Limonite and goethite occur as weathering products of chlorite, only traces of which remain. The approximate mineralogical composition is:

Quartz 15 per cent
Feldspar 10 per cent
Limonite 30 per cent
Sericite 40 per cent
Chlorite 5 per cent

The composition of the rock before weathering was approximately:

Quartz 15 per cent
Feldspar 30 per cent
Sericite 20 per cent
Chlorite 35 per cent

Sample 9. *Schist—Moderately weathered*

Hand specimen—Coarse-grained quartz and feldspar porphyroblasts in a fine-grained flaky groundmass. Light grey, brownish in part.

Thin section—Groundmass consists of lepidoblastic sericite, quartz, chlorite and limonite. The limonite is from weathering of chlorite—approximately 50 per cent of the original amount remains. Limonite staining extends out from larger chloritic areas into sericitic bands. Feldspars are 50 per cent weathered to sericite. Weathering has spread outward from microfractures. Quartz grains also contain numerous microfractures, many filled with limonite. Several hematite cores occur, surrounded by goethite. The approximate mineralogical composition is:

Quartz 30 per cent
Feldspar 10 per cent
Limonite and goethite 20 per cent
Chlorite 15 per cent
Sericite 25 per cent

Sample 10. *Schist—Slightly weathered*

Hand specimen—Occasional quartz and feldspar porphyroblasts in fine-grained sericite/chlorite. Flaky texture. Greenish-grey colour.

Thin section—Sericite (50 per cent) and chlorite (20 per cent) form a lepidoblastic groundmass. Porphyroblasts of quartz up to 0.3 millimetre (15 per cent), contain numerous microfractures filled with sericite, and a few unfilled. Grains of feldspar up to one millimetre (15 per cent) are partly decomposed, mainly by metamorphism but partly by weathering. Occasional crystals of unweathered hematite occur, and pyrite shows only slight traces of weathering at grain boundaries. Chlorite is partly weathered to limonite, (approximately 20 per cent affected) which occurs in veins less than 0.5 millimetre wide.

Sample 11. *Gneiss—Slightly weathered*

Hand specimen—Bands up to 10 millimetres wide of coarse-grained quartz and feldspar alternating with bands up to 30 millimetres of chlorite and sericite, with porphyroblasts of quartz and feldspar. Occasional pyrite flakes.

Thin specimen—Coarse bands consist of anhedral quartz and feldspar grains up to five millimetres diameter separated by fine-grained quartz, sericite and aggregates of chlorite. Fine-grained bands consist of chlorite, sericite and quartz. Approximately 25 per cent of the chlorite has been weathered to limonite. Opaques (hematite and pyrite) comprise 2 per cent of the sample. Feldspar crystals are partly weathered to sericite. Pyrite crystals are partly weathered to limonite, and hematite crystals are slightly affected at boundaries.

ROCK SUBSTANCES AFFECTED BY RAPID DETERIORATION

Sample 12. *Schist—Severely affected by rapid deterioration* (*Adit No.* 1)

Hand specimen—Slightly weathered. Consists of quartz porphyroblasts in flaky chlorite/sericite groundmass. Occasional pyrite grains. Manly greenish-grey with some light grey (sericite) bands.

Deterioration at the surface consists of flaking concentrated in bands along the foliation trace. The zone of disintegrated rock ranges from two to 10 millimetres in thickness. Worst affected bands are chloritic.

Thin section—(through material adjacent to the surface and including some fretting rock.) Porphyroblasts of fractured quartz elongated along the foliation direction (up to 10 millimetres long and three millimetres wide) occur in sericite groundmass. Elongated fibrous chlorite aggregates occur in bands up to 15 millimetres wide. Fractures in the direction of the foliation are spaced between 0.5 millimetre and 15 millimetres apart, more closely in chloritic bands. Fractures are filled with limonite but some are open to two millimetres. Some chlorite aggregates are unaffected. Occasional grains of sulphide (not pyrite) occur, unaffected by weathering. Several patches of limonite approximately one millimetre in diameter occur—possibly weathering products of sulphides. Fretted flakes are mineralogically the same as the intact rock substance. No sulphate identified.

Sample 13. *Schist (somewhat gneissic) with efflorescent sulphate—Gorge Road deviation*

Hand specimen—Slightly weathered with some moderately weathered areas. Rock consists of flaky sericite and chlorite with numerous porphyroblasts of quartz and some feldspar concentrated in bands. Pyrite occurs mainly in quartz bands.

Thin section—(does not extend to fretting surface). Numerous elongated coarse-grained quartz and feldspar porphyroblasts up to 15 millimetres long and four millimetres wide, occur throughout sample. Bands of fine-grained quartz and sericite up to 20 millimetres wide alternate with bands of fibrous chlorite and sericite. Pyrite occurs in quartz/sericite bands where it forms 10 per cent of the material. Limonite surrounds pyrite veins and extends outwards along microfractures. Occasional elongated hematite grains are present in chloritic bands but show no sign of deterioration. No sulphate detected.

INFILLED SOIL SEAMS

Sample 14. *Infilled seam (mottled grey and red). The sample was taken from a prominent seam (seam A) along the foliation direction, in the excavation for grout cap block A.*

Hand specimen—A seam of clay 40 millimetres wide occurs within a zone of completely weathered rock containing roots up to eight millimetres diameter. The seam contains numerous fragments up to 10 millimetres in length (mainly less than three millimetres), of weathered schist and quartz. Approximately 40 per cent of the clay is mottled red-brown, the rest grey. Much of the adjacent rock is stained yellow-brown.

Thin section—The seam consists of fragments of quartz and schist (50 per cent) in a matrix of clay, much of which is stained with goethite. In places in the central part of the seam there is a banding perpendicular to the seam boundaries. The boundaries are distinct in places due to limonite staining along the contact, and indistinct in other places. Many of the rock fragments near the boundary appear to have been rotated into near parallelism with the seam direction.

Sample 15. *Infilled seam (red). The sample was taken from a prominent seam along a Set* (2) *joint in the embankment foundation.*

Hand specimen—The seam consists of red-brown clay containing numerous fragments and grains of schist and quartz. The seam occurred between blocks of slightly weathered schist and the boundaries were sharp.

Thin section—The seam consists of 60 per cent schist fragments and quartz grains up to eight millimetres in length, mainly sub-angular to sub-rounded, in a matrix of clay. The clay is stained by numerous veinlets of limonite which also surround the rock fragments. Elongated fragments have a wide range of orientation although areas of local parallelism occur in parts of the sample.

Sample 16. *Infilled seam (grey). The sample was taken from a seam in the foundation for grout cap block A.*

Hand specimen—The seam of clay is 8 to 15 millimetres wide and occurs in a zone of highly weathered schist, containing roots up to four millimetres diameter. The seam is mainly grey-green in colour with a few patches of brown. It appears to consist mainly of clay with a few quartz grains.

Thin section—The seam consists of 40 per cent quartz grains and schist fragments up to two millimetres diameter, in a matrix of clay containing numerous voids up to one millimetre diameter. The clay is greenish-grey and contains no limonite except in isolated patches. The rock fragments are mainly subrounded to subangular and appear to be randomly orientated except near the boundaries of the seam where they are near-parallel to the seam direction.

Appendix 3

THE UNIFIED SOILS CLASSIFICATION

COARSE-GRAINED SOILS More than 50% of material is larger than No. 200 B.S. sieve size	**FIELD INVESTIGATION PROCEDURES** Excluding particles larger than 7·5cm and basing fractions on estimated weights		
	GRAVELS More than 50% of the coarse fraction is larger than 2mm. (retained on B.S.7 sieve)	**CLEAN GRAVELS** Little or no fines	Wide range in grain sizes, and substantial amounts of all intermediate particle sizes
			Predominantly one size or a range of sizes, with some intermediate sizes missing
		DIRTY GRAVELS Appreciable amount of fines	Non-plastic fines—for indentification see ML below
			Plastic fines—for identification see CL below
	SANDS More than 50% of the coarse fraction is smaller than 2mm. (passing B.S.7 sieve)	**CLEAN SANDS** Little or no fines	Wide range in grain sizes, and substantial amounts of all intermediate particle sizes
			Predominantly one size or a range of sizes, with some intermediate sizes missing
		DIRTY SANDS Appreciable amount of fines	Non-plastic fines—for indentification see ML below
			Plastic fines—for identification see CL below

FINE-GRAINED SOILS More than 50% of material is smaller than No. 200 B.S. sieve size	**FIELD INVESTIGATION PROCEDURES** on fraction smaller than 0.4mm. (passing B.S. 36 sieve)						
		SOIL CAST (soil wet)	SOIL THREAD	SHINE	DILATANCY	ODOUR	DRY STRENGTH
	SILTS AND CLAYS Liquid limit less than 50	Forms fragile cast Cracks form when kneaded while moist	Thick crumbly thread, easily broken	None to very dull	Distinct	Not significant	None to slight
		Cast maybe handled freely without breaking Can be kneaded moist without cracking Material adheres to the hand	Thread can be pointed as fine as a lead pencil but is fragile	Moderate	None to slight	Not significant	Moderate
		Cast fragile to cohesive material will adhere somewhat to the hand	Soft, weak thread	None to very dull	Slight to distinct	Decayed organic matter	Low
	SILTS AND CLAYS Liquid limit more than 50	Moderately plastic and cohesive Material adheres somewhat to the hand	Weak to medium thread May be crumbly	Dull	None to slight	Not significant	Moderate Powdered soil feels floury
		Very plastic and cohesive Material very sticky to the hand Greasy to touch	Very tough thread, can be rolled to a pin point	Very glossy	None	Strong earthy	High to very high Cannot be powdered by finger pressure
		Plastic and cohesive Feels slightly spongy Greasy to touch	Weak to medium thread Often soft and fibrous	Moderate to very glossy	None	Decayed organic matter	Moderate to high Powdered soil may be fibrous
		Readily identified by colour, odour, spongy feel and frequently by fibrous texture					

NOTE: BOUNDARY CLASSIFICATIONS: Soil possessing characteristics of two groups are shown as a combination of two group symbols, eg. GW-GC, well graded gravel with clay binder.

Bull.44 70–641

SOILS CLASSIFICATION

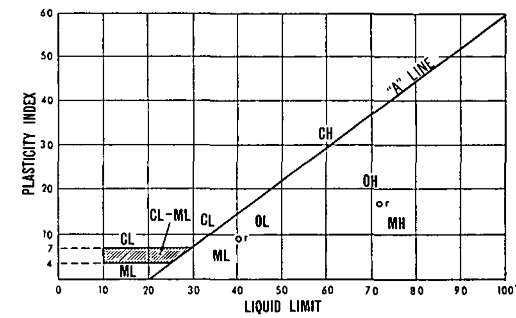
GROUP SYMBOL	GROUP NAME and typical materials	LABORATORY CLASSIFICATION CRITERIA	
GW	**GRAVEL, well graded;** gravel sand mixtures, little or no fines	$Cu = D60/D10$ Greater than 4 $Cc = (D30)^2/D10 \times D60$ Between 1 and 3	
GP	**GRAVEL, poorly graded;** gravel sand mixtures, little or no fines	Not meeting all gradation requirements for GW	
GM	**GRAVEL, excess silty fines;** poorly graded gravel-sand-silt mixtures	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
GC	**GRAVEL, excess clayey fines;** poorly graded gravel-sand-clay mixtures	Atterberg limits below "A" line or PI greater than 7	
SW	**SAND, well graded;** well graded sands, gravelly sands, little or no fines	$Cu = D60/D10$ Greater than 6 $Cc = (D30)^2/D10 \times D60$ Between 1 and 3	
SP	**SAND, poorly graded;** poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW	
SM	**SAND, excess silty fines;** poorly graded sand-silt mixtures	Atterberg limits below "A" line or PI less than 4	Above "A" line with PI between 4 and 7 are borderline cases requiring use of dual symbols
SC	**SAND, excess clayey fines;** poorly graded sand-clay mixtures	Atterberg limits below "A" line or PI greater than 7	

GRAIN SIZE CURVES to be used to identify soil fractions

Coarse-grained soil classified on basis of percentage of fines, as follows

PERCENT OF FINES	GRAVELS	SANDS
Less than 5	GW GP	SW SP
More than 12	GM GC	SM SC
5 to 12	Borderline cases, use 2 symbols	

GROUP SYMBOL	GROUP NAME and typical materials
ML	**SILT SOIL, low plasticity;** inorganic silts and very fine silty or clayey sands, rock flour
CL	**CLAY SOIL, low plasticity;** inorganic clays of low to medium plasticity, gravelly clay, sand, clays, silty clays, lean clays
OL	**ORGANIC SOIL, low plasticity;** organic silts and silt clays of low plasticity
MH	**SILT SOIL, high plasticity;** inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
CH	**CLAY SOIL, high plasticity;** inorganic clays of high plasticity, fat clays
OH	**ORGANIC SOIL, high plasticity;** organic clays of medium to high plasticity
Pt	**PEATY SOIL;** Peat and other highly organic soils



PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE-GRAINED SOILS

Based on "The Unified Soil Classification System" United States Department of the Interior, Bureau of Reclamation "Earth Manual" First Edition, Denver COLARADO 1960. 70–641 Bull.44

Appendix 4
SCHMIDT REBOUND HAMMER TESTS

SCHMIDT REBOUND HAMMER TESTS

The Schmidt rebound hammer was originally designed for testing concrete. Dixon (1969) has shown its applicability for measuring the coefficient of restitution of *in situ* rock substance. Dixon also confirmed the manufacturers' claim that there is an approximate relation between coefficient of restitution and unconfined compressive strength.

At the Kangaroo Creek Dam site the Schmidt hammer has been used extensively on rock faces to determine its value in determining the quality of rockfill materials. A selection of the results is given in Table 4-1.

Factors influencing results

The tests on rocks have shown that the reading recorded by the hammer depends on several factors:—

1. Tests carried out on joints give rebound numbers considerably higher than tests carried out across the rock fabric (see Table 4-1).
2. Tests carried out on surfaces coated with loose materials give lower rebound numbers than tests on uncoated surfaces.
3. The presence of moisture results in lower rebound numbers (see Table 4-1).
4. Tests carried out on foliation surfaces usually give lower results due to flaking.
5. Abnormally low values are recorded when the impact of the hammer causes fracturing of large mineral grains on the surface.
6. The roughness of the surface does not significantly affect the results, except that on very rough surfaces the projections tend to fracture on impact.

To standardize the tests as far as possible, the following rules were adopted for selection and preparation of test surfaces:—

1. Only artificial surfaces, across the fabric of the rock, are tested.
2. Only dry surfaces are tested.
3. The surfaces are prepared by brushing with a wire brush.
4. Tests are not carried out within 30 centimetres of significant discontinuities.
5. As far as possible large crystals are avoided.
6. Abnormally low values obviously due to fracturing or crushing, are ignored.
7. Very rough and fractured surfaces are avoided.

Figure 4-1 is an example of the test result form designed for use at Kangaroo Creek Dam.

Value of Schmidt rebound hammer

The hammer was used extensively at Kangaroo Creek Dam site and it was found that the results could be related to the degree of weathering for a particular rock type (see pages 201 and 202). Wide ranges of test values were obtained for some rocks, due to differentiated mineral banding within the test areas.

The suitability of the hammer for use by inspection staff in determining the suitability of rock for fill was considered. However, it was considered to be impractical for the rock conditions at the Kangaroo Creek Dam site due to the wide range of rock types and the numerous factors affecting the test results. It is evident that the hammer would be suitable for this type of use in quarrying in situations where geological conditions are more simple, *e.g.* many granite quarries.

	TEST No. 58
PROJECT: *KANGAROO CREEK DAM*	
LOCATION: *SPILLWAY EXCAVATION BATTER 2. Ch. 1 + 40*	

NATURE OF SUBSTANCE		OBSERVATIONS OF REBOUND NUMBER	DEVIATION	EFFECT
ROCK TYPE	*Gneiss*	24	0	*Slight crushing*
DESCRIPTION	*Alternating bands of coarse-grained quartz/feldspar and fine-grained chlorite etc.*	25	+1	,,
		26	+2	,,
WEATHERING	*Moderately weathered*	25	+1	,,
		18	-6	,,
NATURE OF SURFACE		25	+1	,,
TYPE	*Across fabric*	26	+2	,,
		25	+1	,,
COATING	*Surface veneer of clay*	24	0	,,
		22	-2	,,
TEXTURE	*Hackly D=0 to 2mm W=0 to 15mm*	TOTAL 240		
MOISTURE	*Dry*	MEAN 24·0	1·6	

ANGLE OF HAMMER TO HORIZONTAL	−30°	CORRECTED MEAN REBOUND NUMBER	26·0

SKETCH

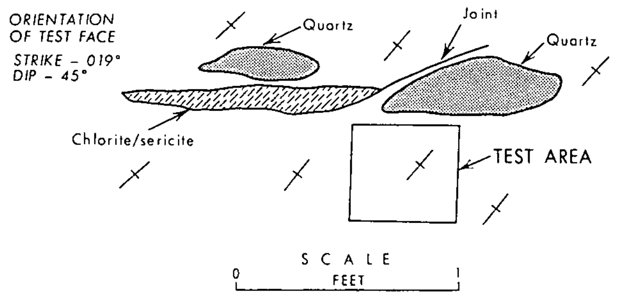


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Fig. 4-1. Rebound hammer test results sheet.

APPENDIX 4. TABLE 4-1

Schmidt Rebound Hammer Test Results

Test location number	Rock type and degree of weathering	Nature of surface: Type	Texture	Coatings, moisture, *etc.*	Schmidt hammer readings	Hammer angle	Corrected mean rebound number	Notes
						(degrees)		
57	Schist, slightly weathered	Across the fabric.	Hackly	No coating, dry	27, 33, 39, 32, 34, 34, 32, 30, 24, 29	0	31.5	Partial crushing of surface irregularities
58	Gneiss, moderately weathered	Across the fabric.	Hackly	Thin clayey coating, dry	24, 25, 26, 25, 18, 25, 26, 25, 24, 22	—30	26.0	Slight crushing of surface
59	Gneiss, moderately weathered	Across the fabric.	Hackly	No coating, dry..	18, 23, 22, 18, 19, 19, 25, 23, 17, 16	—60	23.0	Slight crushing of surface with partial indentation
60	Gneiss, highly weathered	Across the fabric.	Hackly	No coating, dry..	14, 11, 13, 16, 12, 13, 12, 12, 13, 12	—70	15.5	Considerable crushing and indentation of surface
61	Schist, moderately weathered	Across the fabric.	Hackly	No coating, dry..	23, 24, 22, 22, 25, 20, 21, 18, 21, 23	—10	22.5	Partial indentation of the surface
62	Schist, slightly weathered	Partly across the fabric, partly along the foliation	Hackly to scaly	No coating, dry..	26, 24, 20, 32, 24, 20, 26, 25, 32, 27	—30	27.5	Partial crushing of surface irregularities
63	Schist, moderately to highly weathered	Across the fabric.	Smooth with isolated granular projections	No coating, dry..	<10, <10, <10, 10, 11, 10, <10, 10, <10, 11	—20	11.0	Indentation of the surface
64	Schist, moderately weathered	Along the foliation	Smooth	No coating, dry..	20, 22, 22, 23, 23, 23, 24, 23, 21, 18	—30	23.0	Some flaking of surface, slight crushing
65	Gneiss, moderately weathered	Across the fabric.	Hackly	No coating, dry..	24, 20, 23, 19, 28, 24, 31, 22, 22, 30	—45	26.5	Fracturing of larger grains
66	Schist, fresh to slightly weathered (affected by rapid deterioration)	Across the fabric.	Hackly	Sulphate salts coating (removed before test), dry	28, 24, 31, 26, 25, 31, 24, 24, 31, 22	—30	27.5	Slight crushing of irregularities

APPENDIX 4. TABLE 4-1—*continued*

SCHMIDT REBOUND HAMMER TEST RESULTS—*continued*

Test location number	Rock type and degree of weathering	Nature of surface: Type	Nature of surface: Texture	Nature of surface: Coatings, moisture, *etc.*	Schmidt hammer readings	Hammer angle (degrees)	Corrected mean rebound number	Notes
67	Gneiss, slightly weathered	Across the fabric.	Hackly to corrugated	No coating, dry. .	27, 25, 24, 27, 25, 28, 25, 28, 25, 28	—20	28.0	Slight crushing of irregularities
68	Gneiss, slightly weathered	Across the fabric.	Hackly	No coating, dry	30, 30, 26, 33, 38, 30, 40, 27, 27, 31	—25	32.5	Fracturing of large protruding crystals
69	Gneiss, slightly weathered	Across the fabric.	Hackly	No coating, dry. .	37, 30, 30, 34, 30, 43, 42, 37, 30, 28	0	34.0	Crushing of larger crystals
70	Gneiss, fresh	Across the fabric.	Hackly to corrugated	No coating, dry. .	40, 30, 43, 32, 39, 39, 40, 37, 40, 45	—20	39.5	None
71	Gneiss, fresh (same rock as Test 70)	Across the fabric.	Hackly to corrugated	No coating, wet.	35, 30, 32, 32, 37, 40, 32, 42, 37, 28	—20	35.5	Slight crushing
5	Schist, slightly weathered	Across the fabric.	Corrugated. . . .	Fretted material on surface, removed by brushing, dry	23, 17, 26, 24, 20, 20, 26, 24, 23, 25, 20, 22	—30	24.5	Slight indentation
7	Granitic gneiss, slightly weathered	Across the fabric.	Rough, uneven	No coating, dry	45, 22, 27, 27, 31, 30, 34, 25, 30, 28	—25	31.0	Cracking of larger crystals
9	Schist, fresh	Across the fabric.	Rough.	No coating, dry	24, 25, 31, 23, 32, 36, 32, 29, 28, 29	+8	28.5	None
20	Schist, fresh	Set (4) joint	Hackly to scaly	Limonite coated, dry	40, 34, 46, 50, 38, 38, 31, 43, 51, 39	—35	42.0	Some flaking of surface
18	Schist, fresh	Set (2) joint	Smooth	No coating, dry	40, 53, 40, 43, 53, 51, 48, 43, 58, 55	0	48.0	None
13	Gneiss, slightly weathered	Set (2) joint	Smooth to scaly	Limonite coated, dry	50, 49, 58, 58, 57, 61, 54, 60, 55, 57, 55, 62	—5	56.0	None

Appendix 5

DURABILITY TESTS ON ROCK SUBSTANCES

DURABILITY TESTS ON ROCK SUBSTANCES

Three types of durability tests were carried out on rock substances from various parts of the site area. The results are given in Tables 5-1, 5-2 and 5-3.

Sodium sulphate soundness test

This test was carried out according to the procedures outlined in the A.S.T.M. Standards, part 10, test designation C88-63. The test which was carried out on crushed aggregate, consists of five cycles of saturation in concentrated sodium sulphate solution, followed by evaporation. The results, expressed as a percentage loss of the weight of the fractions in the original grading, are given in Table 5-1.

D.M.R. weathering test

This test, also carried out on crushed aggregate, consists of 10 cycles of alternate wetting and drying, followed by compaction. Sieve analyses and plasticity tests were carried out before and after the weathering cycles. The results are given in Table 5-1. The test also includes the chemical analysis of the wash waters, the results of which are given in Table 5-2.

Tests using temperature and weak salt solution

The test carried out was a modified version of the A.S.T.M. test designation C218-48T (test for building stones). The samples tested were in the form of drill cores and cubes cut from spalls. The test consisted of 50 cycles of immersion in 8 per cent gypsum solution, followed by drying to constant weight. The results are given in Table 5-3.

These tests were carried out by officers of the Engineering and Water Supply Department, Concrete Laboratories, Sassafrass.

TABLE 5-1

SULPHATE SOUNDNESS AND D.M.R. WEATHERING TESTS

Sample number	Type of material	Source	Sodium sulphate soundness (5 cycles) percentage loss	Sieve analysis before and after D.M.R. test 3/4	3/8	3/16	7	14	25	52	100	200	Liquid limit	Plastic limit	Plasticity index	Linear shrinkage
KD1 ..	Gneiss, mainlv fresh	Quarry area. Drill hole, depth 2.5ft. to 15ft.	5.7	100	52.6	36.8	31.0	22.4	15.4	10.2	7.4	5.6	15.3	11.6	3.7	0.2
				100	61.4	41.8	33.0	23.5	16.5	11.3	8.2	6.2	14.3	11.6	2.7	0.1
KD2 ..	Schist, slightly weathered	Quarry area. Drill hole, depth 20ft. to 30ft.	67.3	100	53.1	34.0	25.5	18.4	13.3	9.0	6.3	4.5	15.3	11.1	4.2	0.3
				100	69.1	44.5	31.1	22.1	16.2	11.9	9.0	6.9	16.4	13.8	2.6	0.1
KD3 ..	Schistose gneiss, slightly weathered	Quarry area, trial excavation	39.4	100	45.5	26.0	18.3	11.8	7.6	4.7	3.2	2.4	16.5	11.8	4.7	0.5
				100	65.0	42.6	32.2	23.1	14.9	9.5	6.7	5.0	17.7	14.3	3.4	0.3
KD4 ..	Gneiss, moderately weathered	Quarry area, trial excavation	44.0	100	48.0	27.4	19.9	14.8	9.5	5.9	3.9	2.8	15.5	11.6	3.9	0.1
				100	68.8	44.0	34.9	26.9	16.3	14.4	10.8	8.6	17.2	14.0	3.2	0.3
KD5 ..	Gneiss, moderately weathered	Quarry area, trial excavation	43.7	100	52.7	29.7	19.1	11.2	5.8	3.1	2.0	1.5	14.5	11.8	2.7	0.2
				100	74.3	54.1	41.5	26.8	15.7	9.1	6.4	4.6	15.1	15.1	—	—
KD6 ..	Gneiss, slightly to moderately weathered	Quarry area, trial excavation	42.3	100	83.8	67.4	30.2	24.8	19.5	14.6	11.4	9.2	17.0	12.3	4.7	0.3
				100	84.1	69.8	51.9	40.7	31.6	23.4	17.8	13.8	16.2	12.7	3.5	0.3
KD7 ..	Schist, moderately weathered	Quarry area. Cave	62.1	100	70.7	47.9	37.8	28.0	21.0	16.0	13.3	11.2	18.7	14.6	4.1	0.0
				100	72.3	55.1	43.8	33.4	25.7	19.1	15.1	12.4	17.3	13.1	4.2	0.2
KD8 ..	Gneiss, slightly weathered	Quarry area, trial excavation	13.5	100	63.3	42.2	31.8	22.1	15.3	10.7	8.3	7.0	16.2	13.2	3.0	0.5
				100	72.1	49.9	38.3	27.7	20.3	14.4	10.8	8.6	15.5	11.9	3.6	0.2

KD9 . .	Gneiss and schist, slightly weathered	Quarry area. Drill hole, depth 20ft. to 40ft.	38.0	100	57.4	39.4	30.1	21.6	15.1	9.6	6.7	5.0	16.0	13.3	2.7	0.2
				100	72.1	55.7	43.7	31.7	23.4	17.6	13.4	10.9	14.4	11.6	2.8	0.1
KD10 .	Schist, slightly weathered	Quarry area. Drill hole depth 5ft. to 15ft.	45.3	100	56.6	35.3	26.0	19.0	14.5	11.3	9.6	8.5	19.9	14.5	5.4	0.6
				100	65.3	46.9	39.8	32.0	24.4	18.3	13.7	11.0	18.2	12.3	5.9	0.5
KD11 .	Gneiss, slightly weathered	Quarry area. Drill hole, depth 70ft. to 90ft.	20.8	100	57.2	36.2	26.5	18.1	12.0	7.7	5.5	4.2	14.3	11.8	2.5	0.2
				100	70.1	50.3	38.2	26.7	18.2	12.9	9.3	7.4	14.7	10.9	3.8	0.2
KD12 .	Gneiss, slightly weathered to fresh	Quarry area, road cutting	21.9	100	61.5	43.4	31.9	22.5	16.1	11.4	8.9	7.3	17.0	12.8	4.2	0.4
				100	67.5	50.6	39.1	29.3	22.2	17.5	14.0	11.7	16.2	11.7	4.5	0.4
KD14 .	Gneiss, fresh	Diversion tunnel	—	100	44.2	25.6	18.2	11.9	7.9	5.2	3.8	2.9	—	—	—	—
				100	52.5	32.2	21.8	14.2	9.6	6.8	4.7	3.5	—	—	—	—
KD16 .	Gneiss, slightly weathered	Intake structure	—	100	52.9	30.6	20.9	13.3	8.7	5.5	3.9	3.0	—	—	—	—
				100	65.0	40.2	28.1	18.0	11.6	7.9	5.3	3.9	—	—	—	—
KD18 .	Schist, slightly weathered	Old Gorge Road excavation	—	100	60.1	44.2	32.6	21.6	14.5	10.4	8.1	7.3	20.3	16.0	4.3	0.5
				100	69.4	49.5	38.6	26.0	17.1	11.9	9.2	7.7	18.5	12.8	5.7	0.4
KD19 .	Schist, fresh to slightly weathered	Adit No. 1. 40ft. inside portal	—	100	64.6	44.7	33.8	23.5	16.0	11.5	9.3	8.5	18.3	14.0	4.3	0.5
				100	71.6	49.4	37.1	26.0	18.2	12.9	10.1	8.4	17.1	12.1	5.0	0.5

TABLE 5-2

D.M.R. WEATHERING TEST

CHEMICAL ANALYSIS OF WASH WATERS

Composition in grains per gallon	KD1	KD2	KD3	KD4	KD5	KD6	KD7	KD8	KD9	KD10	KD11	KD12	KD14	KD16	KD18	KD19
Calcium carbonate	3.92	2.66	1.12	2.10	2.80	1.48	8.41	4.38	2.63	0.56	1.40	1.75	3.85	—	—	—
Magnesium carbonate	3.48	1.89	0.55	1.48	1.05	0.56	—	—	—	0.65	1.03	—	—	—	—	—
Magnesium sulphate ..	0.34	0.08	0.08	0.59	1.85	0.35	13.48	6.78	0.34	—	0.88	1.18	5.56	28.7	13.5	39.6
Sodium sulphate ..	7.66	6.76	4.88	4.67	2.59	3.15	—	—	1.44	4.28	3.73	2.68	5.12	9.94	—	—
Sodium chloride ..	7.16	7.61	5.32	4.62	5.11	2.31	96.16	25.94	11.99	13.38	9.70	6.30	1.68	11.5	2.2	0.9
Calcium sulphate ..	—	—	—	—	—	—	6.67	1.86	0.81	—	—	1.62	3.72	17.6	11.6	30.5
Magnesium chloride ..	—	—	—	—	—	—	40.0	1.43	—	—	—	—	—	—	0.3	6.8
Sodium nitrate ...	—	—	—	—	—	—	1.19	—	—	—	—	—	—	—	—	—
Sodium carbonate	—	—	—	—	—	—	—	—	—	0.26	—	—	—	—	—	—
Free sulphuric acid.......	—	—	—	—	—	—	—	—	—	—	—	—	—	—	3.4	22.0

TABLE 5-3

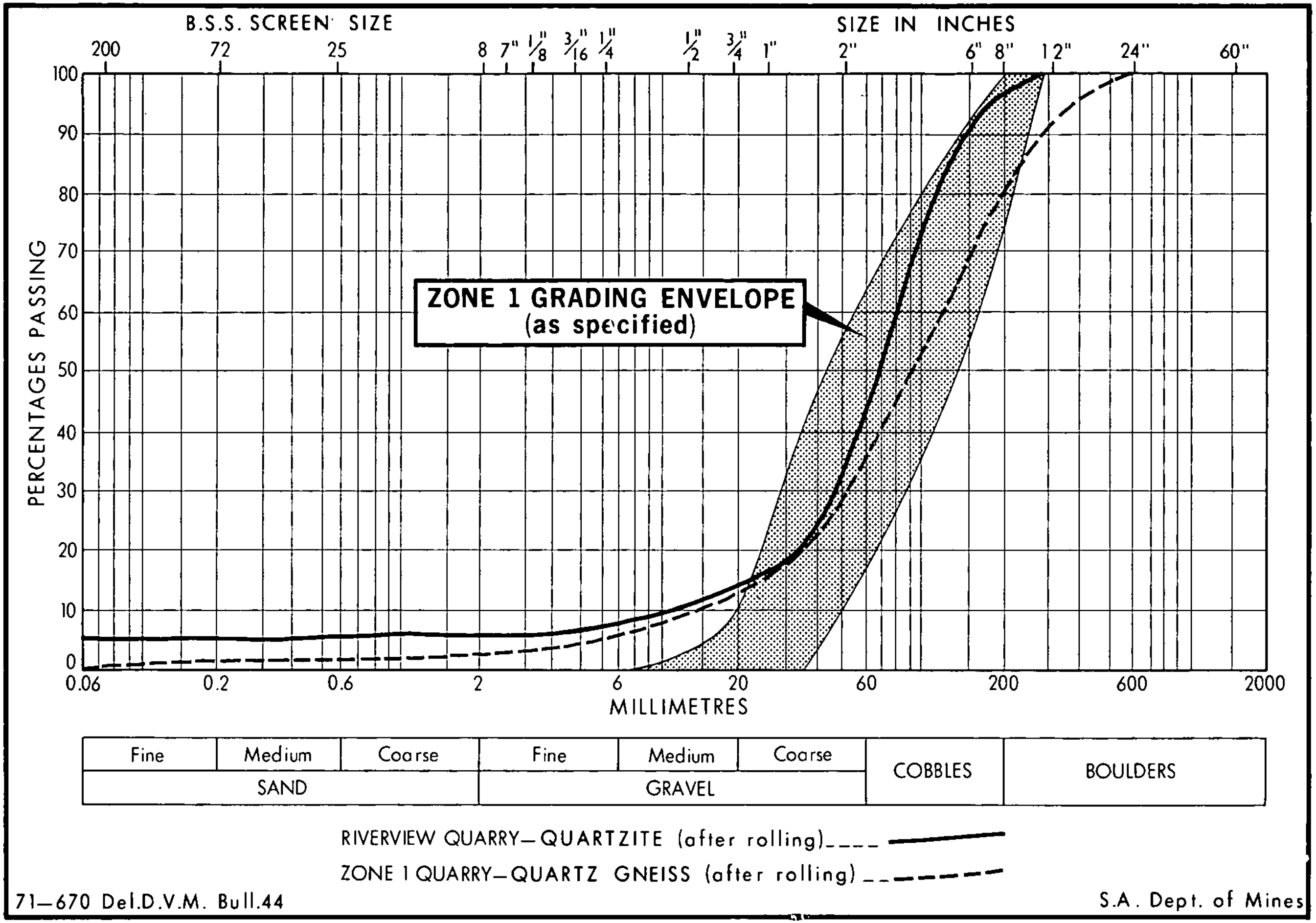
Effects of Combined Temperature and Immersion on Cubes in Weak Salt Solution after 50 Cycles

Sample No.	Original wt. of sample oven dried	Wt. of sample SSD	Wt. of sample immersed in water	Percentage absorption dry wt.	SG. SSD. basis	Oven dry wt. after 10 cycles	Oven dry wt. after 20 cycles	Oven dry wt. after 30 cycles	Oven dry wt. after 40 cycles	Oven dry wt. after 50 cycles	Remarks
		gm	gm								
KD 5 ...	722.2	727.8	469.1	0.78	2.81	722.2	722.1	722.1	722.1	722.1	Not affected
KD 6 ...	703.6	719.7	452.9	2.29	2.70	703.6	703.6	703.6	703.6	703.6	Surface cracks opened slightly
KD 8 ...	719.6	725.1	469.3	0.76	2.83	719.5	719.5	719.5	719.5	719.5	Not affected
KD 9* ..	258.4	263.0	166.1	1.78	2.71	258.4	258.2	258.2	258.2	258.2	Surface cracks opened slightly
KD10* ..	422.6	434.8	273.7	2.89	2.70	422.6	422.1	421.9	421.9	421.9	Surface cracks opened slightly
KD11* ..	239.6	242.1	155.5	1.04	2.80	239.6	239.4	239.2	239.2	239.2	Surface cracks opened slightly
KD12* ..	687.7	699.6	434.7	1.73	2.64	687.7	687.4	687.4	687.4	687.4	Surface cracks opened slightly
KD13 ...	770.8	772.9	503.3	0.27	2.87	770.2	770.0	770.0	770.0	770.0	Surface cracks opened slightly
KD14 ...	794.6	796.0	513.1	0.18	2.81	794.6	794.5	794.5	794.5	794.5	Not affected
KD15 ...	703.6	710.9	450.3	1.04	2.73	701.4	701.4	701.4	701.4	701.4	Surface cracks opened slightly
KD16 ...	1,025.8	1,031.9	652.5	0.59	2.72	1,023.6	1,023.3	1,023.3	1,023.3	1,023.3	Surface crack opened slightly on edge of sample
KD17 ...	645.6	666.5	410.6	3.24	2.60	643.4	643.4	639.0	639.0	638.4	Surface cracks extremely affected causing pieces to break off
KD18 ...	899.4	907.2	568.9	0.87	2.68	894.8	894.8	894.8	894.8	894.8	Medium surface cracks, one crack split sample in two
KD19 ...	671.9	676.9	434.1	0.74	2.79	670.4	670.4	670.4	670.4	670.4	Surface cracks opened moderately

* 9, 10, 11, 12 cores. The rest cubes cut from spalls.

Appendix 6
GRADING OF ZONE 1 ROCKFILL

GRADING OF ZONE 1 ROCKFILL



Appendix 7
DEVIATION OF BLAST HOLES

DEVIATION OF BLAST HOLES

SITE No. 1 Access shaft open cut (gneiss, fresh)

Hole No.	Top of hole Trend	Top of hole Plunge	Bottom of hole Trend	Bottom of hole Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	207	71	207	67	8	50
2	209	71	211	69	8	26
3	210	69	211	68	9	13
4	206	71	211	68	9	43
5	206	70	202	71	8	21
6	198	72	194	72	8	15
7	197	72	187	70	10	48
8	201	71	197	72	7	20
9	201	72	190	68	10	68
10	197	70	195	68	10	27

Average deviation index—33 degrees/100ft.
Average angle to foliation—52 degrees.
Some of these readings are suspect due to the proximity of magnetic metal.

SITE No. 2 Access road (gneiss, slightly weathered)

Hole No.	Top of hole Trend	Top of hole Plunge	Bottom of hole Trend	Bottom of hole Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	206	71	214	65	16	42
2	210	69	221	64	18	42
3	210	69	221	62	21	52
4	208	69	227	61	18	70
5	209	71	222	68	15	34
6	191	67	218	66	18	67
7	215	70	225	66	18	34
8	198	67	216	64	18	50
9	210	68	214	64	15	27
10	206	68	217	65	17	33

Average deviation index—45 degrees/100ft.
Average angle to foliation—54 degrees.

SITE No. 3 Quarry No. 1, batter 4 (gneiss, moderately weathered)

Hole No.	Top of hole Trend	Top of hole Plunge	Bottom of hole Trend	Bottom of hole Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	1	63	351	58	45	16
2	6	65	351	60	50	19
3	12	61	3	59	50	11
4	18	61	3	57	50	19
5	18	60	5	58	50	15
6	1	61	350	56	50	17

Average deviation index—16 degrees/100ft.
Average angle to foliation—21 degrees.

SITE No. 4 Gorge Road deviation (gneiss, slightly weathered)

Hole No.	Top of hole Trend	Top of hole Plunge	Bottom of hole Trend	Bottom of hole Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	211	67	227	61	15	61
2	198	66	224	63	12	77
3	226	67	236	60	18	55
4	205	68	218	62	18	54
5	223	66	236	61	17	51
6	212	66	239	61	18	86
7	221	68	241	59	18	84
8	225	65	237	58	20	60
9	222	63	232	59	11	42
10	228	60	234	55	15	40

Average deviation index—61 degrees/100ft.
Average angle to foliation—21 degrees.

SITE No. 5 Spillway, batter 3 (schist, moderately weathered)

Hole No.	Top of hole Trend	Top of hole Plunge	Bottom of hole Trend	Bottom of hole Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	013	46	007	46	55	8
2	016	44	013	47	52	7
3	014	46	010	46	35	5
4	017	44	008	45	36	12
5	019	46	009	47	40	13
6	016	47	013	47	50	4
7	014	46	008	45	55	8
8	018	47	001	46	45	9
9	020	47	018	47	52	2
10	020	46	016	46	42	5

Average deviation index—7 degrees/100ft.
Average angle to foliation—7 degrees.

SITE No. 6 Spillway, batter 4 (gneiss and schist, slightly weathered)

Hole No.	Top of hole Trend	Top of hole Plunge	Bottom of hole Trend	Bottom of hole Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	018	50	010	50	40	13
2	019	50	002	51	55	27
3	012	48	000	53	56	23
4	015	50	006	53	48	16
5	018	51	008	52	45	16
6	017	51	013	52	48	7
7	018	49	010	49	50	13
8	014	51	003	51	35	17
9	020	50	016	51	43	7
10	021	49	012	47	50	16

Average deviation index—15 degrees/100ft.
Average angle to foliation—8 degrees.

Site No. 7 Spillway, batter 5 (schist, fresh)

Hole No.	Top of hole Trend	Plunge	Bottom of hole Trend	Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	021	44	021	44	25	0
2	021	44	020	44	24	3
3	025	45	024	44	22	5
4	023	43	019	44	26	12
5	023	45	021	44	22	7
6	024	45	021	44	24	9
7	024	44	021	44	20	9
8	026	42	023	43	20	10
9	020	44	018	43	20	7
10	022	45	020	44	20	7

Average deviation index—7 degrees/100ft.
Average angle to foliation—2 degrees.

Site No. 8 Quarry No. 1, batter 6 (gneiss, fresh)

Hole No.	Top of hole Trend	Plunge	Bottom of hole Trend	Plunge	Length (ft.)	Deviation index (deg/100ft.)
1	316	65	301	63	22	30

Deviation index—30 degrees/100ft.

Angle to foliation—47 degrees.

Site No. 9 Access shaft (trial hole) (gneiss, fresh)

Deviation index—approximately 25 degrees/100ft.

Angle to foliation—40 degrees.