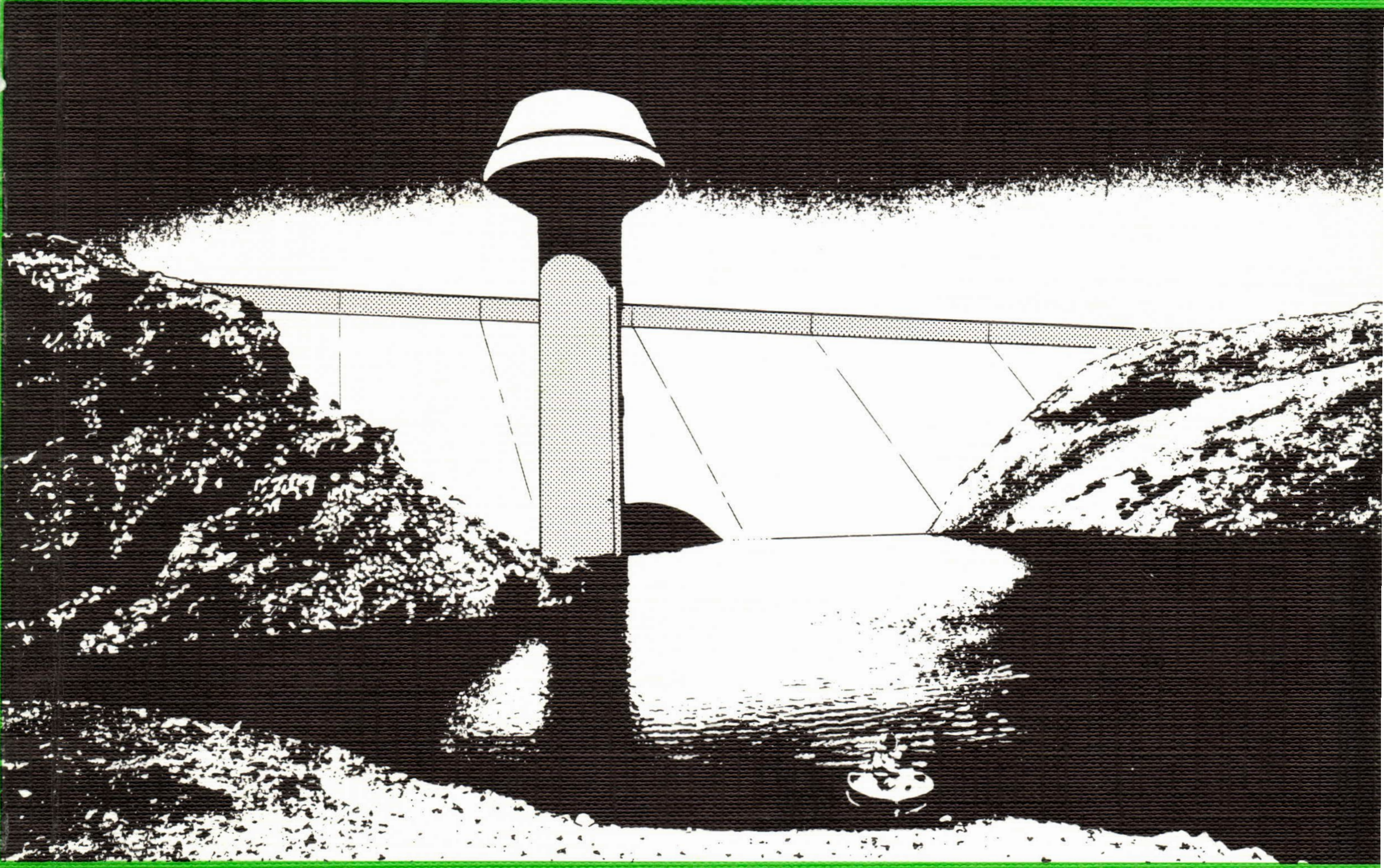
Engineering Geology of Little Para Dam

W.R.P. Boucaut and J.C. Beal



Report of Investigations 54
Geological Survey of South Australia

Department of Mines and Energy
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Frontispiece
Aerial view of Damsite looking southwest in July, 1984.

Engineering Geology of Little Para Dam

SUMMARY

The dam is built in the Little Para River 22 km northeast of Adelaide, South Australia. It is a decked rockfill structure 53 m high impounding a reservoir of 21 000 ML capacity. The reservoir acts as a balancing storage for Murray River water and helps existing reservoirs cope with increasing summertime demands. Also included in the geological investigations are a combined quarry and water treatment works site and a spillway and spillway discharge site.

The dam is founded on quartzite and dolomitic siltstone. The presence of dispersive clay in discontinuous weathered dolomitic siltstone seams in the dam foundation required a 21 m deep grout curtain to minimise leakage and possible removal of the clay by piping. This curtain was checked using point resistivity, self potential and isotopic grout methods. Isotopic grout in conjunction with a geiger probe proved the most successful method of checking the location of grouted strata. Apart from an isolated area on the right bank grout curtain, which gave losses of up to 20 lugeons, water pressure testing of the completed grout curtain gave overall lugeon values ranging from four to seven.

Slope stability analyses carried out over the spillway and quarry batters indicate stable rock mass conditions although minor falls of individual blocks are to be expected. The quarry berms will provide suitable foundations for the water treatment works.

Post-construction surveillance of the dam includes measurements of seepage values at a downstream discharge weir and groundwater level measurements in two observation wells placed adjacent to the downstream toe of the dam. Total seepage discharge was initially about 0.1 L/s at 15 m head of water, mainly from springs which existed beneath the embankment prior to the construction of the dam. Maximum total discharge measurements taken in September 1979 with 37 m of water in the reservoir indicate a total discharge of 10 L/s.

Instrumentation showed an early settlement of 500 mm during construction in the lower third of the embankment; this was caused by insufficient saturation of the fill during compaction. Total post-construction settlement at January 1979 was 20 mm. The maximum measured pressure of the embankment upon the outlet duct is 970 MPa which is within the design limits of the duct.

ACKNOWLEDGEMENTS

The Engineering and Water Supply Department, the constructing authority, has assisted with the compilation of this report by making available all relevant data collected during the investigation, design, and construction stages of the Little Para Dam Project.

Mr R. Stocker, Resident Site Engineer from 1974 to 1976, is acknowledged for his assistance with the grout and water pressure testing of the grout curtain.

PART I: INTRODUCTION, EARLY INVESTIGATION AND DAM DESIGN

Engineering Statement

The State of South Australia has always been heavily dependant upon the Murray River for its water supply. This supply is augmented by seven reservoirs founded on river systems largely confined within a 50 km radius of Adelaide (Figs 1 and 2). A comparison between Little Para Dam and two other South Australian dams at Kangaroo Creek and Myponga is shown in Table 1.

It was calculated by 1979 that the water supply to the Northern Plains area of Adelaide would be inadequate to meet demand. To overcome this a concrete-faced rockfill dam has been constructed on the Little Para River, 22 km northeast of Adelaide and approximately 7 km due east of the City of Salisbury, by the Engineering and Water Supply Department (E & WS Dept) of South Australia (Plate 1).

Apart from impounding natural waters the reservoir will receive Murray River water from a branch off the existing Mannum-Adelaide pipeline. A water treatment works is sited 300 m downstream from the dam.

Site work began on 18 April 1974. Storage of water commenced in August 1977. A summary of the construction schedule for Little Para Dam is shown in Table 2.

The principal statistics of the dam and reservoir are:

Height of dam above stream bed	53 m
Crest length of dam	255 m
Maximum width at base of dam	140 m
Spillway crest length	71 m
Reservoir capacity	21 000 ML
Reservoir waterspread	125 ha
Elevation of reservoir at full supply level	EL 149.58 m
Area of catchment	80 km^2
Area of Reservoir Reserve	1 300 ha (approx)

For most of the year the reservoir will be less than full in its main role as a balancing storage for Murray River water from the Mannum-Adelaide Pipeline. This will greatly increase the flood control capability of the dam.

Reservoir discharge during periods of high natural recharge can be regulated by flood control measures incorporated into the design of the dam. These consist of a notched spillway and a 1 500 mm outlet pipe through the base of the dam to the river, bifurcating at the downstream end into two 750 mm pipes, each controlled by a fixed cone energy dispersion regulating valve. Constructed into the spillway lip are two 1.50 m wide notches, one 0.80 m deep and the other 0.52 m deep. For water supply operations the reservoir is designed to be at full supply level when the water level reaches the bottom of the lower notch. A further overall water depth of 0.8 m is thus provided for the

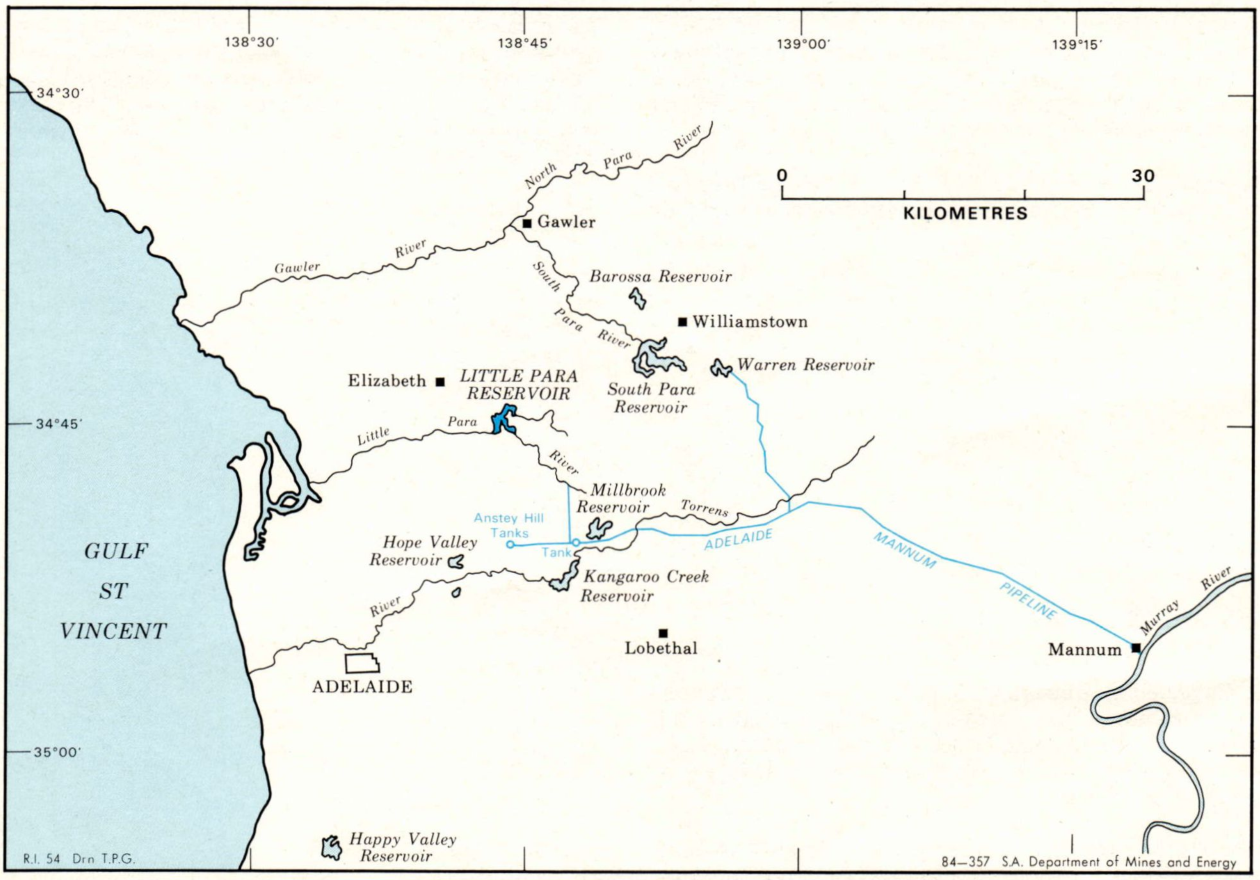


Fig. 1 Locality plan: Little Para Dam project.

Plate 1
Little Para Dam and outlet tower during initial filling of the reservoir in January, 1978 (T13609).

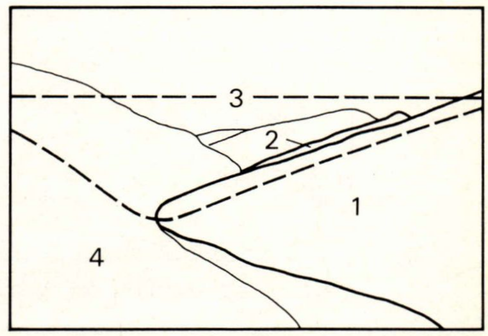
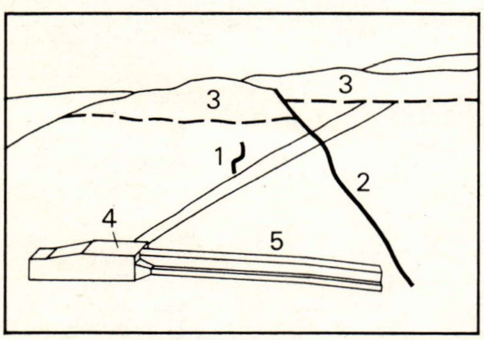


Plate 2
The dam site before construction, looking downstream in July, 1973 (T13066).

1. Quartzite stratum (Unit 12) forming dip slope on right abutment.
2. Spoil from exploration trench.
3. Dam axis.
4. Dolomitic siltstone (Unit 13) of left abutment.



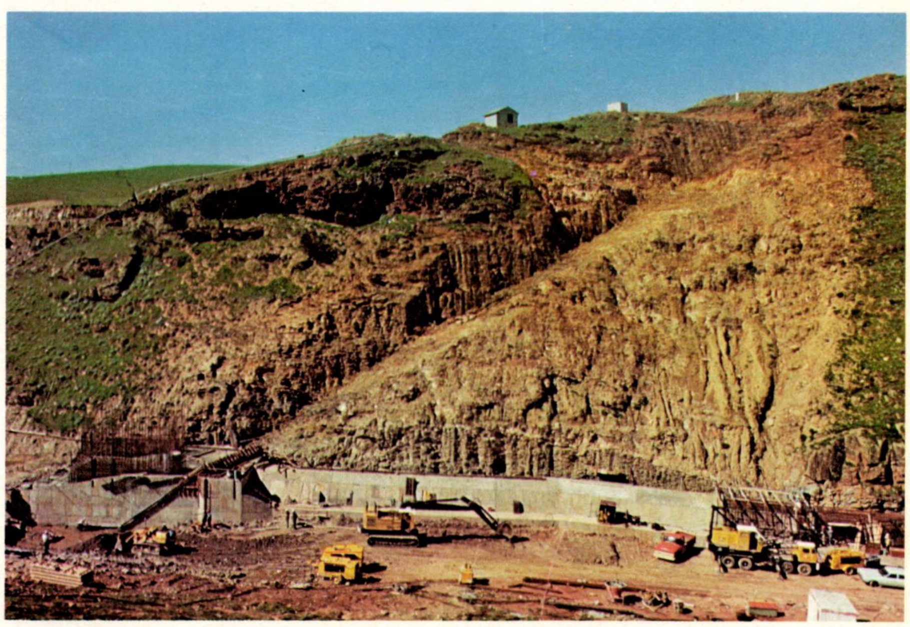


Plate 3
Left abutment and outlet duct during construction in September, 1974 (T24372)

1. Hillside stress relief fracture.
2. Shear zone.
3. Slaty dolomitic siltstone displaced by shear zone.
4. Base of outlet tower.
5. Outlet duct.

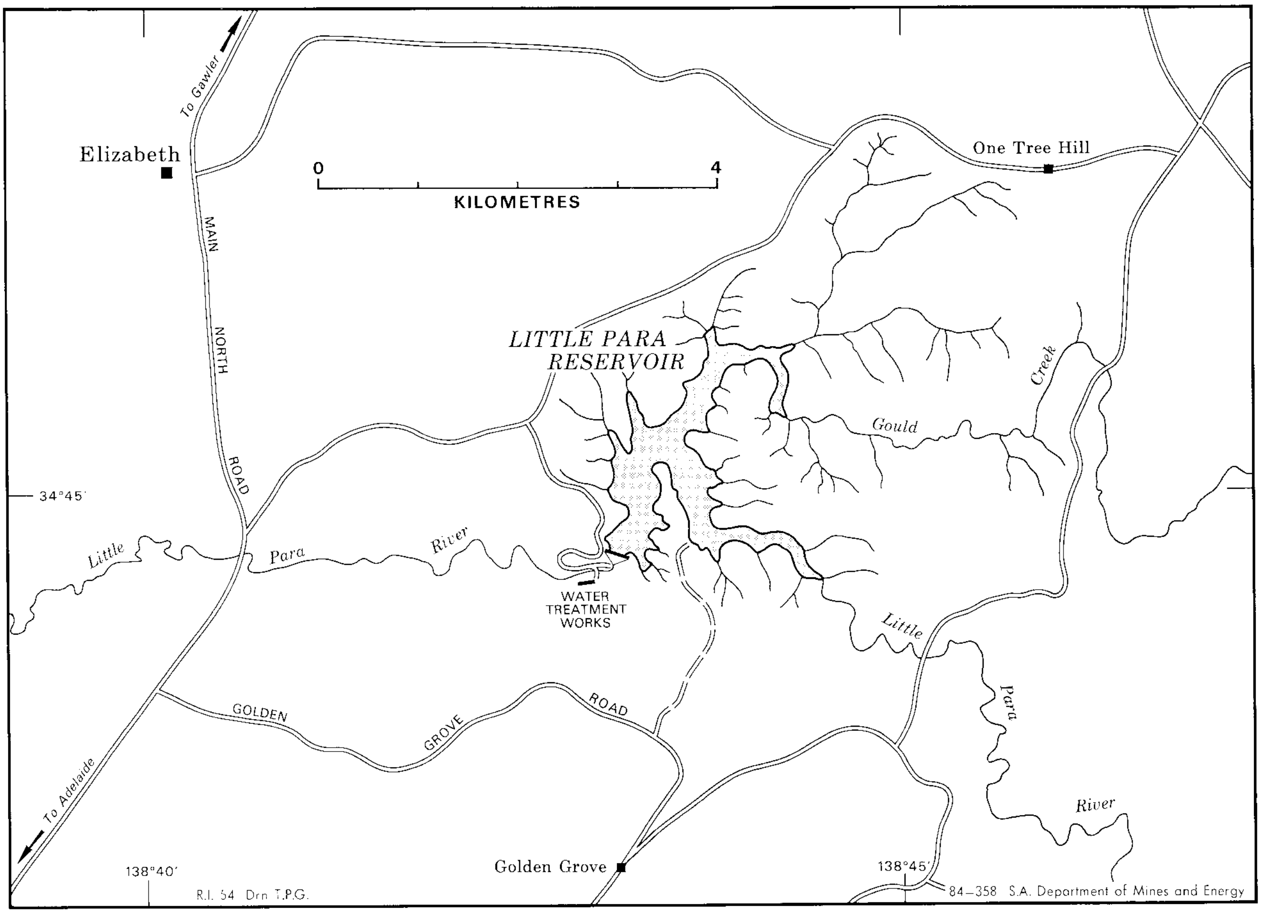


Fig. 2 Locality plan: Little Para Reservoir.

pondage of floodwaters in the reservoir. The discharge of this pondage will be regulated by the notch sizes which will restrict the outflow and reduce the flood peak. To provide additional pondage when the likelihood of an extreme flood is apparent, or to restore the flood control capacity after the spillway has been overtopped by an unexpected extreme flood, the river outlet valves can be used to lower the level of the reservoir at a controlled rate which will be well within the capacity of the lower channels of Little Para River in the northern suburbs of Salisbury and Elizabeth.

Early Investigations

Between 1959 and 1961 several rivers to the north of Adelaide were briefly investigated for potential dam sites by the SA Department of Mines, and different dam types for each discussed with the E & WS Dept. In 1964 it was decided to restrict investigations to the Little Para River, and the Department of Mines carried out a hydrogeological survey of the Little Para River, investigating its role in the recharge of aquifers downstream of the reservoir and lying beneath the North Adelaide Plains (Kingstone and Shepherd, 1973).

Table 1 Comparison of Little Para Dam with Kangaroo Creek Dam and Myponga Dam

	Little Para Rockfill 1978	Kangaroo Creek Rockfill 1969	Myponga Concrete 1957
Capacity	21 000 ML	24 400 ML	27 300 ML
Years of construction	1974-77	1966-69	1958-62
Actual cost (includes land acquisition)	$11.0 million	$5.3 million	$6.3 million
Cost on December 1973 basis	$5.6 million	$7.9 million	$12.0 million
Cost per 1000 ML on October 1973 basis	$267 000	$324 000	$440 000
Height above stream bed	53.00 m	60.95 m	47.55 m
Quantity of rockfill in the dam	288 000 m^3	354 000 m^3	
Quantity of concrete in dam, spillway, tower, etc.	14 000 m^3	17 860 m^3	57 380 m^3
Area of waterspread	126 ha	130 ha	260 ha
Elevation of stream bed	El. 100.00 m	El. 183.95 m	El. 156.66 m
Elevation of full supply level	El. 149.30 m	El. 241.86 m	El. 211.38 m
Elevation of top of dam	El. 153.00 m	El. 244.91 m	El. 213.21 m
Length of crest	225 m	131 m	226 m
Catchment area	82.5 km^2	*290.0 km^2	116.0 km^2

*Shared with Millbrook reservoir.

Table 2 Schedule of construction operations

		1974	1975	1976	1977	1978
Grout caps and abutment excavation	Access roads	90% Complete	Completed			
	Works area	20% Excavated	Completed			
	Services	Telephone	All services connected	Extended to downstream toe of dam		
	Concrete batch plant	Site selected	Completed			
Grout caps and grout curtain, pouring, drilling and grouting	Excavation left grout cap	Completed				
	Excavation right grout cap	Completed				
	Cleanup left abutment	Cleaned up as embankment placed				
	Cleanup right abutment		Completed			
	Grout curtain		Drilling & grouting		Completed and tested	
Outlet tower	Tower base excavation	Completed				
	Blanket and curtain grouting beneath tower	Commenced & Completed				
	Tension bars beneath tower	53 bars placed	Bars tensioned			
	Tower floor, walls		Floor completed, wall commenced	Walls completed to El. 111.5 m; rest of tower completed mid-1977		
Outlet duct and control house	Duct floor Duct arch Control house	Excavation completed	Floor extension to control house commenced	Completed Wing walls formed		Completed
River diversion and causeway	Temporary diversion tunnel	Completed				
	Coffer dam	Completed				
	Permanent diversion tunnel	Commenced		Completed		
	Permanent causeway		Commenced	Completed		
Spillway	Excavation and concreting	Overburden stripped	Channel excavation complete	Flip bucket excavation complete	Completed	
	Anchoring spillway lip		First rockfill placed in November	80% rockfill placed		
Embankment	Placement of rockfill			Commenced	Zones 1,2,3,4,6	Zone 5 Completed
Concrete face and parapet wall				Commenced		Completed in December
Quarry and water treatment works	Haul roads Excavation Design of water works	Continuous development throughout life of project			Commenced 1977 and continued to 1981	
	Construction of water works	Commenced mid-1980; finished late 1984			Investigation of main berm & batter stability	
Main outlet pipe					Commenced	Completed in March

Parliamentary approval to build the dam and water treatment works was given in May 1974
Site work commenced in April 1974
Reservoir in operation by August 1977
Rockfill dam and pipelines completed March 1978
Supplied with pumped Murray River water during June to November 1978

By the end of 1972 a rockfill dam site was chosen from five alternative sites on the basis of geologically suitable foundations and abutments (Plate 2).

A decision against building a concrete gravity dam on this site was influenced by the unsuitable topography of the abutments, the identification of a major shear zone traversing the left abutment, and the presence of cavities in the rock forming the right abutment. These cavities and the presence of associated seams of completely weathered* dolomitic siltstone were considered to provide unacceptable foundations for a concrete gravity dam.

In 1973-1974 the E & WS Dept carried out an ecological survey of the water catchment area and the Little Para River which concluded that, although the catchment area had been extensively modified by man, the river has remained little changed and should be left intact as much as possible during the construction of the dam.

Detailed geological investigation of the dam site commenced in 1973. This included geological mapping, diamond drilling, geophysical surveys, and high pressure water sluicing along the grout cap alignments. Possible sites for a spillway and rockfill quarry were also selected and investigated.

Ecological Survey

A limited ecological survey of the Little Para River catchment was undertaken by the E & WS Dept (Miller, 1973) to outline the biology in the Little Para Reservoir district.

Stream flows in the Little Para River catchment are extremely variable both on an annual and daily basis. The median average rainfall is between 450 mm and 700 mm.

With the exception of some 650 ha along the eastern boundary of the watershed, the plant associations have been extensively and irreversibly modified by man's activity. In the region of the proposed reservoir, modification due to clearing of certain eucalypt associations has led to the local extinction of all but about twenty-five native plant species and their replacement by aliens and noxious weeds. Of these twenty-five species, ten are confined to the stream communities and five to the refuge habitats of quartzite outcrops.

The stream communities differ markedly from the terrestrial communities in that little modification due to human occupation seems to have occurred. In the stream below the dam site a diverse aquatic fauna exists, apparently undisturbed by introduced fauna. It is from this region that the first occurrence of a particular species of psephenid beetle larvae in South Australia was established during the course of the survey.

**See Appendix 5 for definition of descriptive terms.*

The recommendations arising from the study are concerned primarily with protection of water quality and the preservation of ecologically significant areas. Requirements for the management of the reserve, precautions to be taken during the construction and operation of the dam, and recommendations for further studies are as follows:

— Afforestation of the reservoir reserve over a ten year period.
— A programme of seed collection from eucalypt species in the district and their germination for an afforestation programme.
— Rejuvenation of two small swamp areas downstream of the dam.
— Formulation of a noxious weed control programme.
— Selection of the point of discharge of supplementary Murray River water into the Little Para River should have regard for erosion and siltation effects; these effects should be further minimised by the use of baffle weirs and suitable biological communities.
— Protection of the downstream aquatic communities by the maintenance of flow in the Little Para River and by the careful siting of ancillary structures to the dam; protection during construction by diversions of stream flow around areas likely to contribute silt downstream and by preventing water from sluicing operations from entering the stream.
— Retention of as many quartzite outcrops as possible on aesthetic grounds.
— Investigation of the Little Para River estuary to determine the effect of winter discharge on the mangrove communities.

Hydrological Survey

A hydrological study of the Northern Adelaide Plains area was carried out to assess the effect of impounding water by the Little Para Reservoir on the groundwater intake from the river bed to the underlying aquifers.

In the Northern Adelaide Plains, growers rely on the continued use of groundwater for irrigation of their market gardens. Had the hydrological survey shown that building the dam would seriously affect the quantity of groundwater available for recharge of the aquifers, then either modification or abandonment of the Little Para Dam project would have been necessary.

Between 1968 and 1973 flow measurements were made at several points along the Little Para River and groundwater levels taken monthly from observation wells penetrating the Northern Adelaide Plains aquifers.

Hydrographs of wells adjacent to the Little Para River show that it is an important source of recharge to aquifers in the Salisbury area. Recharge takes place through shallow aquifers and then to deeper aquifers through semi-confining beds, and possibly through the Para Fault zone (Fig. 3).

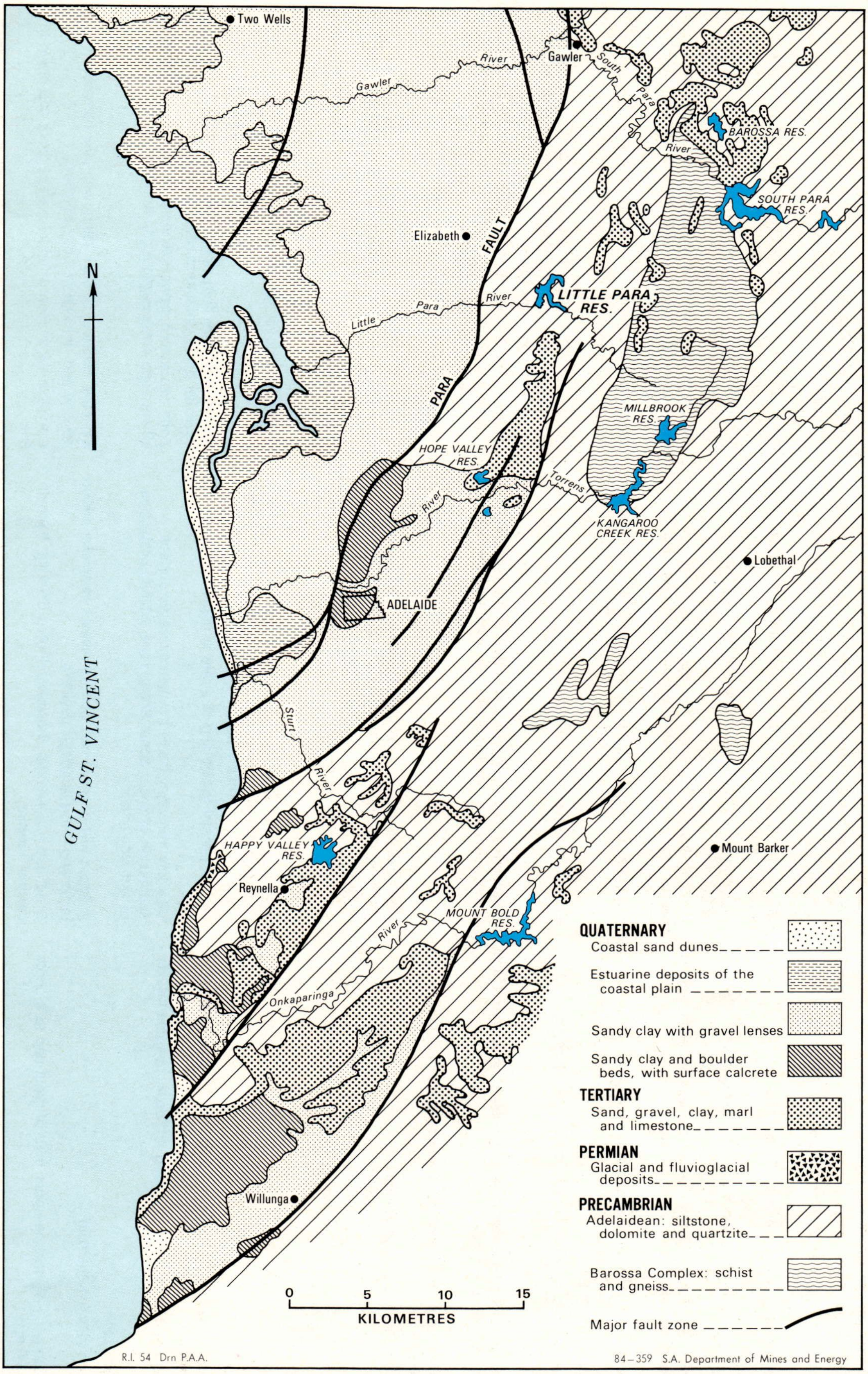


Fig. 3 Regional geology.

Table 3 Summary of investigations

Item Investigated	Description and Results	Solution Adopted
RESERVOIR AND RIVER		
Rim geology	Regional geological mapping of reservoir area.	No leakage anticipated.
Siltation of reservoir at Murray River water pipeline outlet	Low level concrete barrage to hold silt and encourage growth of plant life suggested.	Recommendation accepted.
Siltation of river downstream of dam site	Detrimental to fauna and flora in perennial rock pools and swamp habitats	River will by-pass construction work via coffer dam and outlet duct; silty water prior to this inevitable.
Discharge downstream of cold deoxygenated dam water	Suggested standing pool to allow outlet water to warm and oxygenate	Reservoir water to be taken from high outlet tower level; standing pool not necessary.
Preservation of native fauna and flora and aesthetic appeal of reservoir area	Some rare eucalypts and a rare species of beetle discovered; suggested seed collection and germination programme	Planting of reservoir reserve; limiting excavation of quartzite scarp habitats; rehabilitation of excavated faces; avoid siltation of beetle rock pool habitat.
Recharge of North Adelaide Plains aquifers	Borehole and stream gauge study along Little Para River	Recommended discharge of 3 320 ML/year.
DAM SITE AREA		
Unstable cliff boulders above outlet pipe alignment	Protection of pipe from falling boulders	Resiting of outlet pipe not possible as opposite bank to be used as access road. Pipe protected by concrete after removal of loose boulders.
Uncertain foundation conditions beneath downstream toe of dam and control house	Seismic refraction survey showed bedrock at 5 m with possible cavities. Presence of shear zone may be indicated	Bedrock conditions remain uncertain. Complete excavation of alluvial deposits recommended during construction to allow examination of exposed foundation rock.
Artesian aquifer beneath dam site intercepted in one borehole		Insignificant compared to water head of reservoir.
Completely weathered dolomite	Clays examined for expansive properties. Tests show dominant quantities of Montmorillonite, careful grouting required.	
GROUT CAP		
Left bank		
Hillside fractures	High pressure water sluicing reveals hillside fractures caused by pressure relief or Tectonic processes; produces unstable hillside rock masses	Complete removal advised.
Highly weathered phyllitic dolomite and sheared bedding planes	Occurs parallel to bedding of strata in upper half of grout cap	Removal of upper 3 m along grout cap advised plus selective dental treatment where necessary
Alignment of shear zone behind left bank; possible hillside failure	Very low frequency (VLF) traverse indicates surface expression of shear zone. Shear zone exhibits properties of wet tabular body.	Zone considered a possible leakage path. Dental treatment where zone crosses grout cap advised
Right bank		
Open joints in quartzite	In upper half of grout cap sluicing reveals trenchlike infilled joints and loose quartzite boulders	Selective cleaning of open joints with jack hammers advised and removal of loose boulders.
Weathering of strata in upper third of grout cap to a depth of 4.0 m	Grout cap examination shows weathered dolomite beneath calcrete capping	Complete excavation of upper part of grout cap down to 3 m
Possible cavities beneath quartzite	Drillhole gives core loss beneath base of quartzite at 11.5 m from surface; inspection by borehole TV camera suggests cavity	Downstage grouting techniques for grout curtain advised
Grouting procedure	Above data must be taken into consideration	Downstage grouting techniques to a depth of 21-22 m.

SPILLWAY		
Spillway batters and rock type	Detailed geological mapping	Analysis of rock discontinuities to determine angle of batters and preferred orientation of spillway centre line; bulk of spillway excavation will go into zone 3 of rock fill dam.
Unstable boulders beneath spillway discharge lip	Flip bucket of spillway terminates above jointed quartzite cliff face; falls could possibly undermine flip bucket	Rock bolting and guniting of quartzite face.
Spillway water discharge upon river alluvials in right bank	Geophysical Survey showed *maximum* depth of alluvial is 5.2 m upon sound bedrock; no cavities or shear zone indicated	No problems anticipated, however ponding of discharge water may possibly affect downstream toe of dam (see below).
Spillway design and water discharge	Hydraulic model to simulate 10 000 year flood frequency	Model tests indicate that the damming up of discharge water is not expected after erosion has removed alluvial deposits.
Spray from spillway onto right bank and access road	Soil examination suggests that saturation of opposite hillside may lead to slope movement on to access road or main outlet pipe	Installation and monitoring of hillside pegs along slope and placement of main outlet pipe along left bank recommended.
QUARRY		
Quarry site	Data on joints and bedding planes give stability report; shear tests indicate friction angles for quartzite bedding planes range from 45°-50°.	Quarry to be used as water treatment works site. Safe batters can be achieved during nearly all excavation work, particularly after removal of folded quartzite.
Quarry rock fill material	Drilling shows quartzite and dolomite with some siliceous siltstone. Rock types tested for strength, porosity, permeability, sulphate soundness, Los Angeles abrasion, water absorption, slaking test and clay mineral content	Selective rock types useful for various zones in dam embankments.

On the basis of change in storage of the shallow aquifers, average annual recharge is estimated to be 2 350 ML. It has been estimated on the basis of stream flow measurements in the Little Para River (Kingstone and Shepherd, 1973) that a total annual flow below the dam site of 3 320 ML is required to maintain the present situation, the difference being the water lost by evaporation and transpiration.

This requirement of 3 320 ML will be met as far as is practicable by natural inflows downstream of the dam combined with controlled discharges from the reservoir.

Site Geology

The dam is located on rocks of the Adelaide System (Burra Group) of Proterozoic Age (Fig. 3), having a regional strike of 005° and dipping 25-30°E.

The geological sequence in the vicinity of the dam site is interbedded quartzite, with beds up to 12 m thick, and dolomitic siltstone with beds up to 80 m thick (Fig. 4).

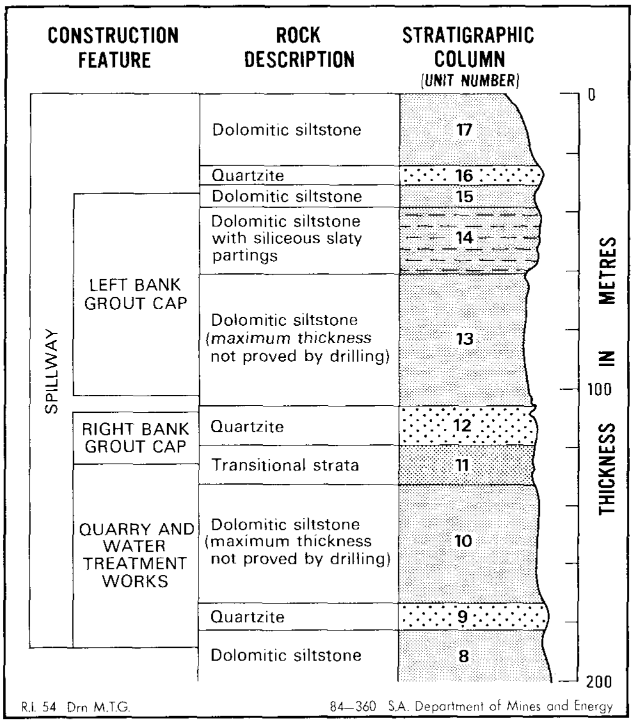


Fig. 4 Stratigraphic column and construction features.

The thickest quartzite stratum (Unit 12) is 12-15 m thick, the lower 3-5 m grading into the underlying dolomitic siltstone, with thin interbeds of quartzite and dolomitic siltstone. This gradational contact between two major rock types has its own weathering and engineering characteristics and is referred to as the *transitional strata*.

At the dam site the geology has controlled the formation of an asymmetric valley. On the left bank* of the river the dolomitic siltstone forms a steep hillside with a prominent cliff at river level where the 12 m

**Looking downstream.*

thick quartzite unit forms a dipslope hillside and is the main foundation for the rockfill dam (Fig. 5).

Regional folding is observed downstream of the dam site, where quartzite beds are folded into shallow anticlines and synclines. However, the less competent dolomitic siltstone adjacent to these folds only exhibits multiple low angled shearing, frequently associated with lens shaped vuggy quartz veins. Unlike the quartzite the siltstone is fissile, splitting along a well formed cleavage which departs from the original bedding plane by only two or three degrees. Shearing commonly follows the cleavage planes but two shear zones, the major tectonic features in the area of the dam site, cut across the cleavage at an angle of approximately 45°. One of these shear zones runs from the bottom of the left bank hillside to the top, striking at 045° and dipping approximately 50° E (Plate 3). The quartzite cliff in the left bank abruptly terminates at river level against the shear zone (Fig. 5). The zone is probably related to transverse faulting initiated during Palaeozoic times. Some of these early faults have been re-activated during and since the Tertiary Period, but there is no evidence of this later movement on the shear zone under discussion: the zone is tight and without clay development.

The presence of a second near-vertical shear zone only became evident during cleanup of the right abutment (Fig. 5). Although there is no surface evidence, downstream projection of this second shear zone would meet the first zone, described above, at about creek level beneath the Control House foundations. Excavation of these foundations exposed no evidence of the two zones.

The dominant joint directions for both rock types fall into three main groups: 025°, 080°, and 120°. The joints are vertical to 70° dipping north and south but more frequently, northwards. The joints intersect to form large blocks in the quartzite (up to 2.0 x 1.0 x 1.0 m) and smaller blocks in the dolomitic siltstone (up to 1.0 x 0.05 x 0.03 m).

Small hillside cavities were exposed in dolomitic siltstone close to the axis of the dam at the top of the right bank hillside. Completely weathered seams of dolomitic siltstone, up to 100-500 mm thick, occur in the left abutment.

The zone of saturation and aeration of successive groundwater tables in the right abutment has formed a highly weathered zone of varying thickness within the dolomitic siltstone. The zone is 3 m thick where it crops out at the top of the right abutment gradually thinning downhill. A plan and section of the geology of the dam site along the grout cap alignment is shown in Figures 6 and 7 respectively.

The criteria used for definition of rock terms are those in current use in the Department of Mines and Energy (Appendix 5). The weathering grade of a rock mass is diagnosed by colour change and loss of original rock strength and rock fabric. This is in contrast to other systems (Dearman, 1976) which use colour change, ratio of rock to soil, and breakdown of rock fabric.

During the early geological appraisal the possibility of assessing rock condition by giving values to parameters which describe rock strength and rock weathering was considered (Burgess, 1975). However this was abandoned because there were only two rock types and the condition of these could be easily assessed using current criteria already familiar to the site engineers.

Site Assessment and Dam Design

A summary of the major geological investigations is given in Table 3.

The Reservoir

Mapping of the rim geology showed no areas of potential leakage and any of the five dam sites initially chosen for investigation would have impounded water within a watertight reservoir. It was therefore not necessary to modify in any way the natural state of the reservoir site.

The Dam Site

The building of a concrete arch dam across the Little Para River at the present site was originally considered but soon dismissed owing to the obvious poor rock quality of the abutments. However, a mass concrete dam was originally considered in more detail. Geological investigations showed that the shear zone had displaced the thick quartzite stratum (Unit 12) from the left hillside onto the right hillside giving an apparently sound right abutment bank suitable for a mass concrete dam.

The dolomitic siltstone on the left bank was however considered too fractured and of too poor a quality to support a mass concrete dam. Another reason for abandoning the idea of building a mass concrete dam was because of the asymmetrical topography of the valley at this site.

A concrete deck rockfill dam was chosen as the most suitable type and its layout together with other associated works is shown in Figure 5. The upstream grout cap and base of the deck was constructed as a series of blocks labelled from 'A' at the top of the left abutment to 'X' at the top of the right abutment.

The outlet pipe passes beneath the spillway lip on the left bank of the river and connects the outlet duct via the control house to the water treatment works.

Originally it was planned to run the outlet pipe along the right river bank but spillway discharge spray could saturate the bank leading possibly to a soil-slip failure of the hillside and dislocation of the outlet pipe. To assess this possibility, survey pegs were placed along the hillside and monitored over a period of eighteen months. The results showed no hillside movement and, although the outlet pipe was finally located on the left bank, the knowledge was useful in assessing the potential stability of the hillside above the batters

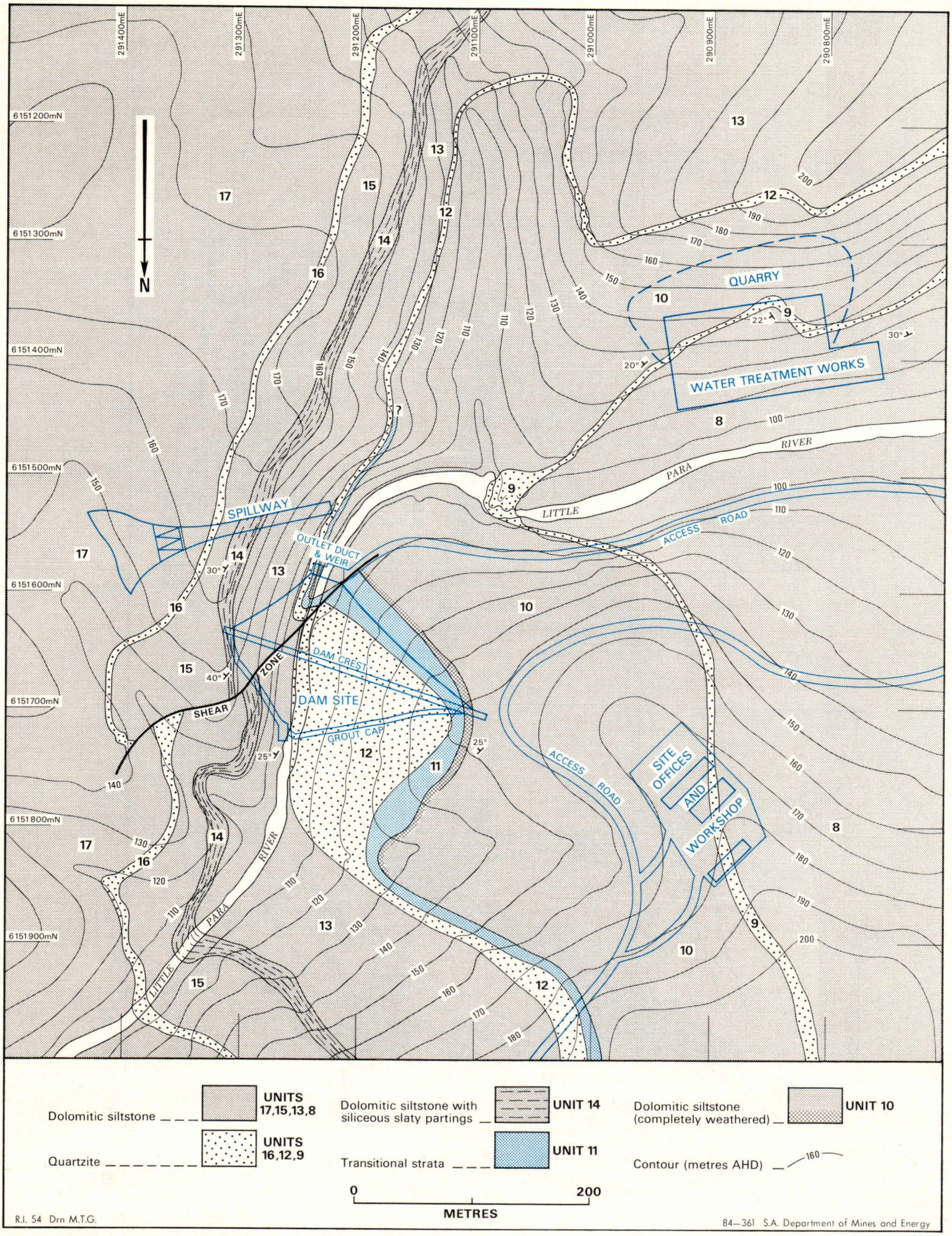


Fig. 5 Geological plan.

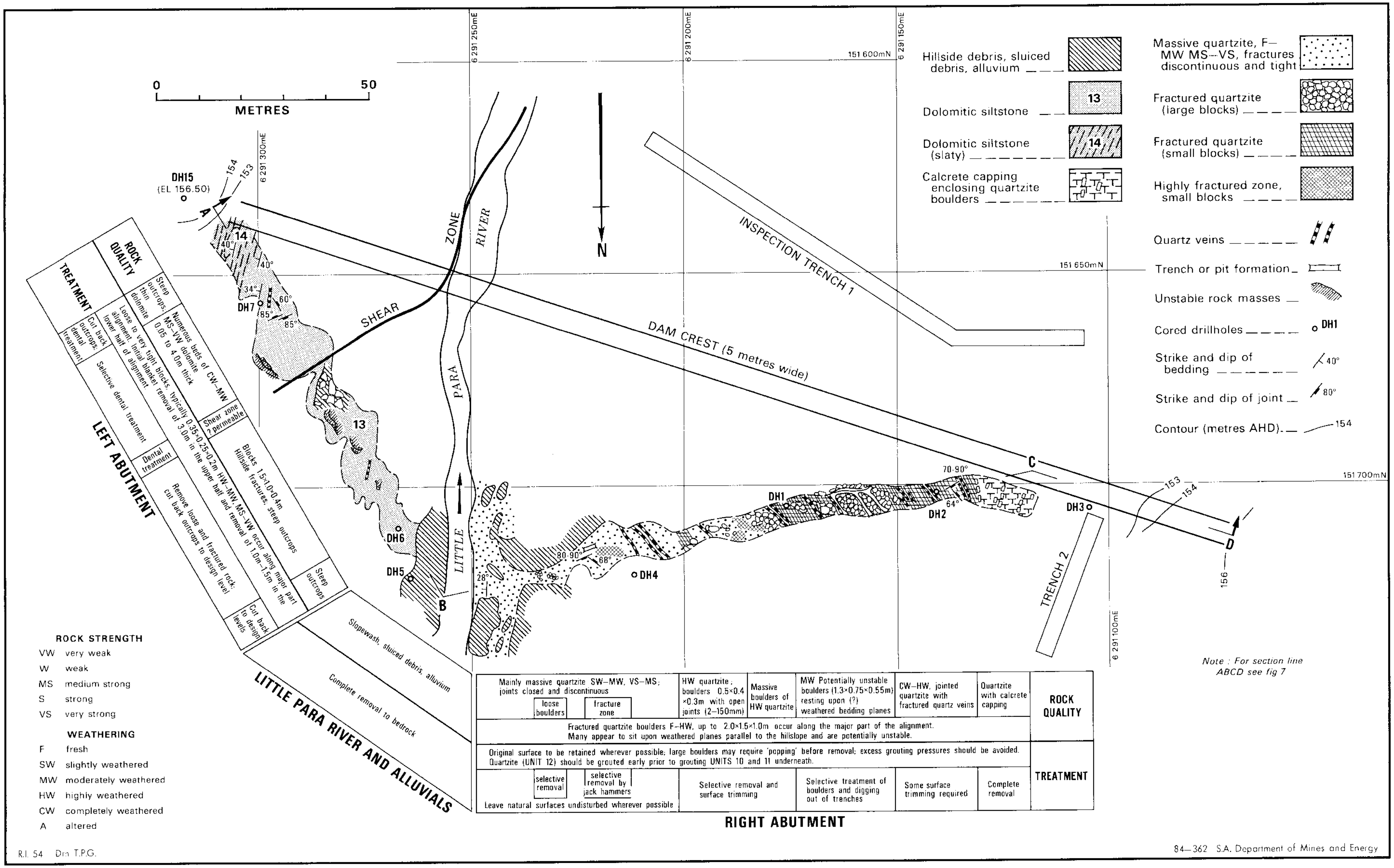


Fig. 6 Grout cap: Geological plan.

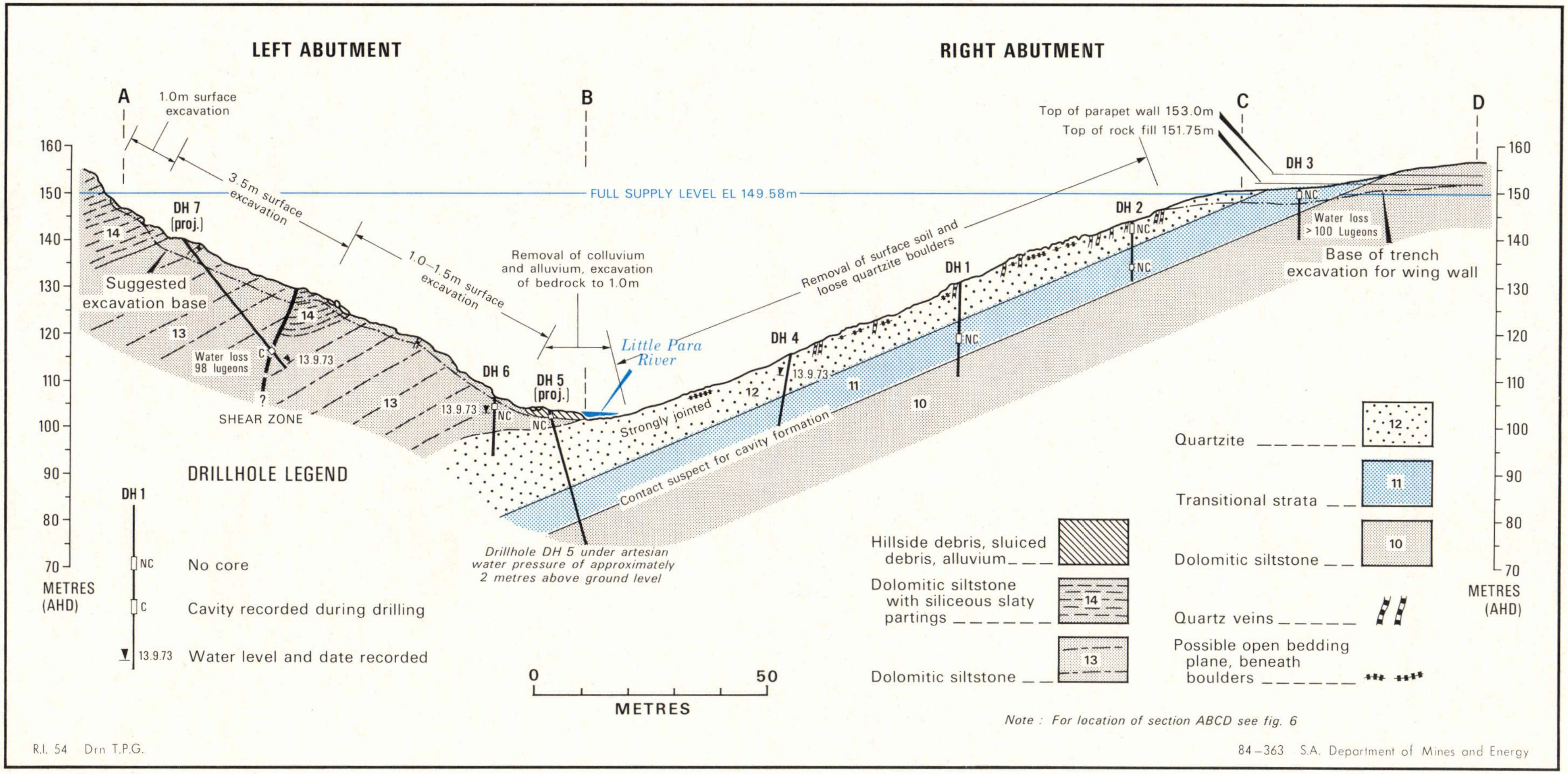


Fig. 7 Grout cap: Geological cross section.

of the permanent access road located along a similar route to that initially proposed for the outlet pipe.

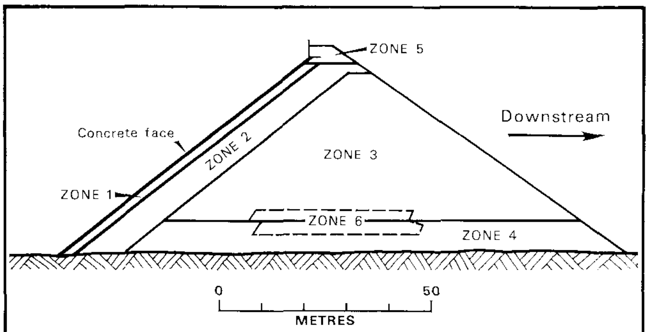
The Grout Cap and Grout Curtain

Diamond core drilling and water pressure testing down to 20 m along the grout cap alignment showed rock conditions which would require a grout curtain for watertightness.

The cap for the grout curtain (Figs. 6 and 7) on the left bank (Plate 3) traverses dolomitic siltstone (Units 13 and 14) and the right bank (Plate 4) traverses for most of its length massive quartzite (Unit 12). Where the cap passes into the parapet wall, at the top of the right bank, it traverses dolomitic siltstone (Unit 11).

High pressure water sluicing along the left bank alignment exposed hillside stress relief fractures, seams of completely weathered dolomitic siltstone, vuggy shattered quartz veins, open joints, and a shear zone. These features were examined during mapping of the dam alignment and some necessitated redesign of the grout cap during construction.

Water sluicing along the right bank alignment exposed large blocks of massive quartzite separated by wide infilled joints, quartz veins, zones of closely jointed and weathered quartzite, and weathered dolomitic siltstone and calcrete. None of these geological features required specific design treatment but constant assessment was required during grout cap excavation.



DESIGN CRITERIA FOR ROCKFILL ZONES

ZONE 1 – well graded durable rock compacted in horizontal layers not exceeding 0.5m in thickness when compacted and graded as specified

ZONE 2 – up to 1000mm diameter rock with not greater than 20% passing a 26.5mm sieve compacted in horizontal layers not exceeding 1.0m in thickness when compacted

ZONE 3 – up to 1000mm diameter rock with not greater than 30% passing a 26.5mm sieve compacted in horizontal layers not exceeding 1.0m in thickness when compacted

ZONE 4 – up to 1300mm diameter rock with not greater than 20% passing a 26.5mm sieve compacted in horizontal layers not exceeding 1.3m in thickness when compacted

ZONE 5 – up to 500mm diameter rock with not greater than 40% passing a 26.5mm sieve compacted in horizontal layers not exceeding 0.5m in thickness when compacted

ZONE 6 – up to 300mm diameter rock with not greater than 40% passing a 26.5mm sieve. Compacted parallel to duct centreline in horizontal layers approximately 2.0m thick when compacted

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Fig. 8 Rockfill zones.

The Spillway

Two locations for the spillway were originally considered: one to be excavated into the left bank hillside adjacent to the rockfill dam and another—the final choice—to broach the hillside on the left bank, 100m downstream of the dam. The first site considered was unacceptable mainly because the relatively shallow excavations for the spillway would have been in dolomitic siltstone already known to show hillside pressure relief fractures and seams completely weathered to some considerable depth—both features which would require extensive treatment. Also, work on a spillway located along the left bank hillside would interfere with construction of the dam immediately below. Finally, a sharp bend would have been required to enable the spillway to discharge flood water into the river at a safe distance away from the downstream toe of the dam.

The Quarry

The main materials quarry had to supply sufficient quantities of Zone 2, Zone 3 and Zone 4 rockfill (Fig. 8 and Plate 5). Because Zones 2 and 4 require strong, fresh rock, investigations for a quarry site were restricted to areas where good quartzite could be extracted.

The final site was chosen (Fig. 5) to meet these requirements and also so that the quarry excavation could be used as a site for a water treatment works.

Access Roads and Other Works

Locations of access roads and the site offices were governed by topography and location of the dam and related structures. Excavation through strong quartzite was unavoidable and at some locations roadside batters adjacent to the site offices needed to be assessed for possible slope instability. Results indicated that although all road batters are stable, individual falls of small boulders will occur during the lifetime of the batter faces.

General site safety involved removal of all obviously unstable and doubtfully stable boulders lying on steep hillsides above all access roads, offices and construction areas.

PART II: MATERIAL TESTS

Introduction and Summary

Testing of rock, clay seams and soil in the vicinity of the dam site was carried out both during the investigation stage and the construction period.

Of major importance was the assessment of the various rock types for possible inclusion in the various rockfill zones of the dam embankment and their suitability as aggregate for on-site concrete. A total of 29 diamond cored holes were drilled during the investigation and construction periods and core was taken from some of these holes and used for testing.

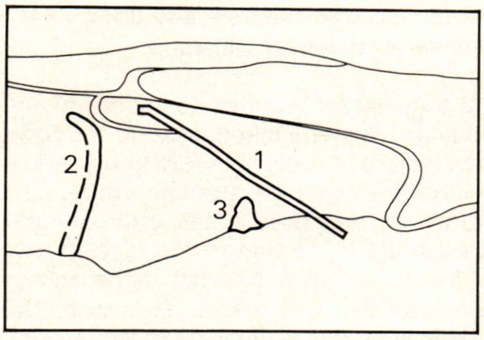


Plate 4
Right abutment in December, 1976 (T24373).
1. Right bank grout cap.
2. Early investigation trench.
3. Over-excavation along a minor shear zone backfilled with zone 1 material.

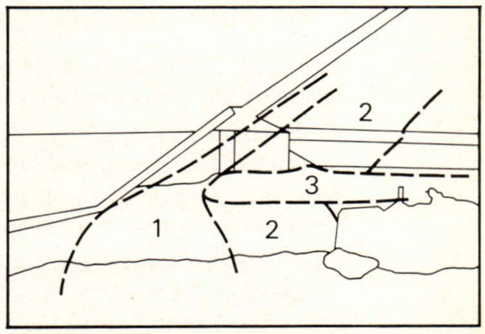


Plate 5
Looking along dam embankment towards left abutment—November, 1976 (T24374).
1. Zone 1 rockfill.
2. Zone 2 rockfill.
3. Zone 6 rockfill.

Plate 6
Cavity exposed in right abutment inspection trench—November, 1972 (N23112).
Note: Scale marked in 10 cm increments.

The material investigation included petrographic description of rock types, analyses of clay, concrete aggregate testing, rock strength testing of quartzite and dolomitic siltstone, ring density tests, and permeability tests on weathered seams of dolomitic siltstone. The results of material testing are summarised below:

- Two main rock types occur over the dam site and quarry area, namely quartzite and dolomitic siltstone. The quartzite is feldspathic and the dolomitic siltstone shows a variable carbonate content; the siltstone is also more siliceous when occurring beneath the base of the quartzite.
- Only the quartzite is suitable for concrete aggregate.
- The quartzite and the siliceous dolomitic siltstone are suitable for rockfill in Zones 1 and 4.
- Fresh to weathered dolomitic siltstone is suitable for Zone 3.
- Breakdown of the dolomitic siltstone due to weathering produces a sandy silt with a 10-20 per cent clay content. The clay may be nearly all montmorillonite or montmorillonite and vermiculite.
- The quartzite shows an unconfined compressive strength within the range 85-296 MPa.
- Unconfined compressive strength values for the dolomitic siltstone fall within the range 30-105 MPa.
- Axial and diametral point load tests on the dolomitic siltstone gave values of from 1.7 to 13.8 MPa.
- Permeability tests on samples of completely weathered dolomitic siltstone give values ranging from 1×10^{-5} to 5×10^{-7} cm/s.
- Dry densities from ring density tests carried out on Zone 3 material in the dam embankment gave values of from 1 840 to 2 150 kg/m^3 which lie mainly within the required specifications.

Petrological descriptions and a summary of physical properties of the two major rock types are given in Appendix 4.

Table 4 Summary of petrological descriptions

Sample	Analysis
Quartzite	Quartz 55-95% Feldspars 5-45% Opaques, Sericite, Chlorite } Traces only
Dolomite	Dolomite 90-95% Quartz 3-6% Opaques 2-4%

Rock Tests

Physical Properties

A summary of the physical tests carried out on specimens of dolomitic siltstone and quartzite is given in Appendix 4.

Values of unconfined compressive and point load strengths* related to drilling speed in the quarry site are also given. Drilling rates appear to vary from 10-20 m/hr in quartzite to 25-35 m/hr in fresh to moderately weathered dolomitic siltstone and up to 78 m/hr in highly weathered dolomitic siltstone.

The axial point load strength index for some of the samples of dolomitic siltstone taken close to the base of the quartzite where it becomes siliceous, show values in the extremely high rock strength range. Low values (0.5 MPa) for diametral point load strength indices are considered to be due to the high fissility of the dolomitic siltstone in a direction approaching normal to the axis of the core tested. True diametral tests are very difficult to carry out owing to this fissility. Unconfined compressive strength tests for moderately weathered to strongly weathered dolomitic siltstone fall within the range 30-105 MPa.

Low unconfined compressive strength values for the quartzite are accounted for by the high feldspar content of some of the samples. The feldspar is rarely fresh and shows breakdown into kaolin and sericite.

It was noted during core testing that core samples showed an increase in their unconfined compressive strength with depth owing to a decrease in weathering.

Shear Tests

Using the Hoek portable shear testing machine, peak and residual angles of friction were derived in the laboratory for bedding planes in the quartzite and dolomitic siltstone on samples taken from the proposed quarry site (Attewell *et al.*, 1976; Hoek and Bray, 1974). The work was carried out by the Consulting Engineers, Coffey and Partners Pty Ltd and results of the tests are included in Appendix 3.

Folding of the quartzite has caused the bedding planes to dip into the quarry at approximately 22° and samples taken were tested across this plane. Values for the shearing angle of from 45-50° were recorded for the quartzite reflecting the interlocking nature of the surface of the bedding planes. Bedding planes within the dolomitic siltstone are far smoother than in the quartzite and friction angles of 25-30° were recorded. These values were useful as a guide when assessing batter safety (Hazeldene, 1975) during excavation of access roads and the quarry (Appendix 3).

Assessment for Rockfill Zones and Concrete Aggregate

Samples of quartzite and dolomitic siltstone were tested in the E & WS Dept laboratory for their suitability for use in the free draining zones of the rockfill dam (Zones 1, 2 and 4) and for use as on-site concrete aggregate. Results of these tests together with associated petrological descriptions are summarised in Appendix 4.

Accelerated weathering test results (Appendix 4) show that the dolomitic siltstone is acceptable for Zone 3 rockfill, the fines produced from these tests having low plastic clay properties. However petrological examination (Appendix 4) carried out by the Australian Mineral Development Laboratories (AMDEL) showed

*Definitions of rock strength are given in Appendix 5.

Table 5 Occurrence of cavities and seams of completely weathered dolomitic siltstone

Occurrence	Investigation	Description	Treatment
RIGHT ABUTMENT			
Early investigation trench down right abutment.	Trench 2-3 m deep excavated downstream of dam axis from El. 145 m to El. 110 m.	Three cavities in transitional strata all approx. 1.0 × 0.5 × 2.5 m lateral extent with associated weathered seams.	Cavities backfilled with rockfill.
Trench for parapet wall foundation, Blocks T,U,V,W.	Trench 3-4 m deep at top of right abutment; transitional strata exposed.	(CW)* dolomitic siltstone seam up to 3 m thick with three small 10 cm deep (area 2 cm^2) caused by solution or piping effects.	Trench infilled with construction concrete to form parapet wall foundation.
Outcrops upstream and downstream of the dam axis.	Eight backhoe pits 2 m deep located along outcrops of the transitional strata.	(CW) seams from 0.3 to 2.8 m thick exposed.	No treatment of upstream outcrops. (CW) seam assumed to pass beneath the right abutment and reverse sand filter placed over downstream outcrop.
Beneath the grout cap.	Early Investigation Core Hole DH1 (El. 130 m) Early Investigation Core Hole DH2 (El. 144 m) Grout curtain cored check hole DH24 (El. 122 m)	800 mm core loss from 11.55 to 12.35 m. 200 mm core loss from 9.10 to 0.30 m. 250 mm core loss from 10.25 to 10.50 m at intersection of a shear zone and weathered seam (see Fig. 42).	No special treatment but careful grouting in this area. Hole grouted following a water pressure test.
LEFT ABUTMENT			
Grout Cap.	Excavation of grout cap foundation and batters	Seam 0.50 m thick at El. 140 m Seam 0.70 m thick at El. 120 m Both seams persisted laterally and to depth.	Excavated down to approx. 1 m across width of grout cap and filled with dental concrete.
SPILLWAY			
Behind wall slabs R1, R2, L2, L3 and partly beneath floor slabs FR1, FL2.	Excavation of spillway channel wall and spillway floor.	Seam occurs beneath a continuous quartzite stratum 0.5 m thick.	Dental treatment.
Passing behind wall slabs FR6, FL7 and beneath floor slab F7.	As for 6 above.	Seam associated with a vuggy quartz vein; discontinuous.	Dental treatment.

*(CW) Completely weathered, see Appendix 5
Note: See Appendix 4 for laboratory test results

that the phyllosilicates, reactive silica, dolomite, and pyrite content of the dolomitic siltstone prevent its use as concrete aggregate. Only the quartzite and some siliceous dolomitic siltstone proved acceptable for use in both the rockfill zones and as potential aggregate. However, because of the cost of assembling an on-site crusher, and as on-site quality control for quartzite aggregate would have been difficult owing to the wide range in feldspar content of the quartzite and its associated variably weathered nature, aggregate was brought in from a nearby commercial supplier.

Moderately weathered to slightly weathered dolomitic siltstone was used only for the main mass of the dam (Zone 3). The mineralogy of the completely weathered dolomite is discussed under the next section.

Soil Tests and Clay Mineralogy

Road Fill

Hillside colluvium from the foot of the quarry and water treatment works site was analysed to assess its suitability as fill for temporary access roads. Results (Appendix 4) show a soil comprised of approximately 50% gravel, 40% silt and clay and 10% sand. This is a very poorly graded gravel. The fines give a plasticity index of 32.7. The colluvium was considered most unsuitable for use as road fill, satisfactory compaction being extremely difficult to achieve without the addition of sand sized material.

Fill material for temporary roads was obtained on-site from any conveniently located excavation into weathered bedrock. Permanent roads were formed of crushed broken quartzite placed beneath a graded base brought onto the site from the nearby commercial quarry.

Weathered Dolomitic Siltstone

Completely to highly weathered seams of dolomitic siltstone passed beneath the grout cap alignment and across the grout curtain. These could have led to water leakage and possible piping and would therefore have been detrimental to the structures had they been left undiscovered and untreated. Table 5 summarises work done on dolomitic siltstone occurring on site. Samples were taken from the seams and forwarded to AMDEL for petrographic description. The on-site treatment of completely weathered dolomitic siltstone is discussed in Part III: Design and Construction.

Seams of completely weathered dolomitic siltstone and associated cavities (Plate 6) occur within the transitional strata which were first exposed in an early inspection trench running down the right abutment. In earlier reports on the site they were variously described as flaggy quartzite, quartz-sericite meta-siltstone, phyllitic dolomite, or foliated dolomitic siltstone and indicate the varying nature of the transitional strata. Samples of moderately to completely weathered transitional strata were forwarded for testing to the Soils and Foundations Laboratory of the E & WS Dept. Grading curves from their mechanical analysis (Appendix 4) show the weathered samples to consist of 10% clay, 55% silt, and 35% fine to medium grained sand.

Mineralogy

Because the cavities found beneath the right abutment were associated with dolomitic siltstone, and because the main mass of the dam (Zone 3) is constructed of dolomitic siltstone, information was required on the nature of its chemical decomposition. Three samples representing progressive breakdown of the dolomitic siltstone were forwarded to AMDEL for petrological analysis and mineragraphic descriptions. Results are summarised in Table 6.

Examination of clay products found within the weathered seams show that, for most samples, the clay fraction is dominant with sub-dominant to accessory kaolin-type clays and vermiculite. This was also true for ten clay samples taken from thin seams adjacent to the cavities found within the transitional strata. Dolomite occurred in only one out of the ten samples as a dominant mineral. In all other samples dolomite was completely absent; calcite occurred in eight of them (dominant in three samples but elsewhere appearing as an accessory or trace mineral with montmorillonite and mica). Samples of low plasticity clay taken from infilled joints in the quartzite beneath the grout cap on the right bank show quartz as the dominant mineral.

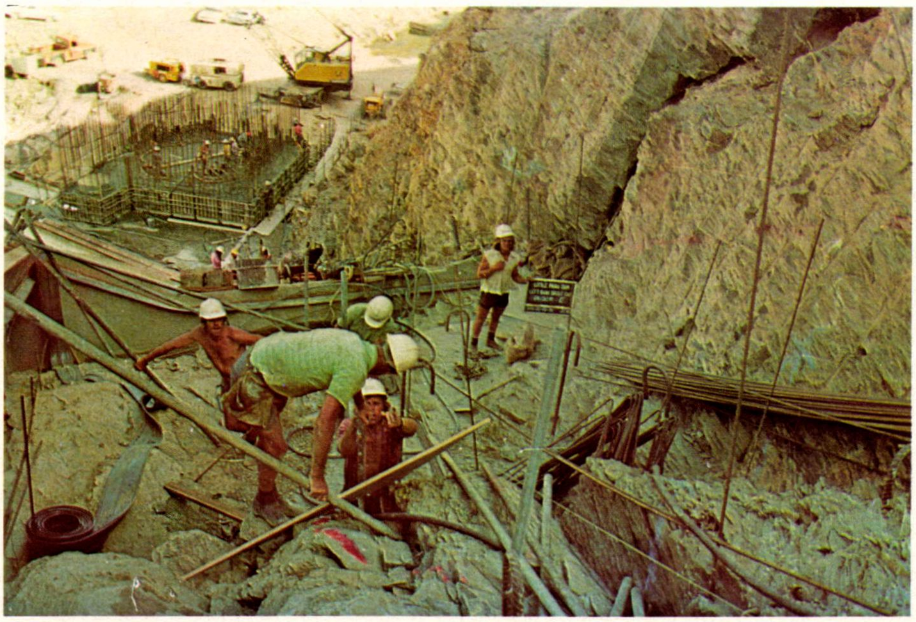
The low dolomite content of the clay samples compared to the high dolomite content of unweathered parent rock indicates a breakdown of the dolomite into calcite and in some cases limonite, the dolomite being slightly ferroan. The magnesium ions released from the dolomite are relocated to form vermiculite and montmorillonite, and possibly sericite and chlorite. It appears unlikely that the vermiculite is an interme-

Table 6 Mineralogy of weathered dolomitic siltstone

Sample No.	Petrographic Description	Condition	Quartz/ Mica/ Feldspar %	Dolomite %	Vermiculite %	Montmorillonite %
1	Moderately foliated phyllitic dolomitic siltstone.	Least weathered of the three samples	30-40	60-70		
2	Quartz-sericite metasiltstone	Highly weathered	90-95	Traces (assoc. with limonite)	5-10	
3	Quartz-sericite metasiltstone	Completely weathered (ML-CL)*	55-70 (sericite 15-25)	Nil		15-20

*Unified soil classification (Appendix 5)

Plate 7
Upstream view of wetting and compaction of zone 3 rockfill in July, 1976 (T24375).



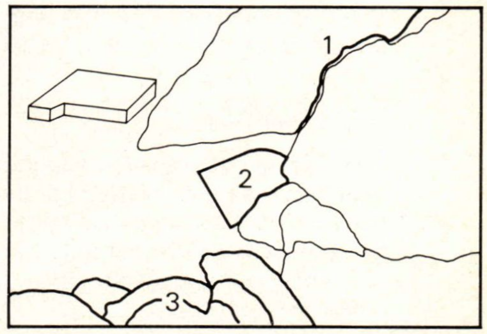


Plate 8
Left bank grout cap excavation—March, 1974 (T24376).

1. Hillside stress relief fracture.
2. Dental concrete in excavation seam of weathered dolomitic siltstone (Unit 13).
3. Shows slabby nature of dolomitic siltstone.

Plate 9
Open bedding plane on right bank grout cap excavation. Overlying quartzite was later removed by hydraulic rock splitter. Background shows temporary coffer dam (left) and diversion pipe in front of outlet tower base (T24377).

diate product in the formation of montmorillonite from sericite as the factors which determine whether vermiculite or montmorillonite forms are the concentration of magnesium ions and the intensity of leaching.

In dolomite-free horizons, where a relative paucity of magnesium ions could be anticipated during weathering, kaolinite may be expected to form rather than montmorillonite and vermiculite.

Because vermiculite has a high cation exchange capacity and may be present in weathered dolomitic siltstone lying adjacent to the grout curtain, consideration was given to the possible long term reaction between cement of the grout curtain and vermiculite. Fortunately vermiculite's high cation exchange capacity is only continuously active in areas of water circulation and, in addition, cement in the grout curtain has a high free lime content which should help stabilise dispersive clays as the calcium replaces the sodium.

Permeability of weathered dolomitic siltstone containing vermiculite is low and the effect of vermiculite upon grout curtain cement is not considered significant.

Dispersion Tests

Thin seams of completely weathered dolomitic siltstone occur on the right abutment at the base of the quartzite (Unit 10), in the transitional strata (Unit 11) and towards the top of the left abutment. Their dispersive nature was assessed because of possible leakage of reservoir water through them.

Samples taken from the main occurrence of weathered dolomitic siltstone, at the top of the right abutment beneath the parapet wall, show compositions of around 30% sand, 50% silt, and 10% clay (Appendix 4). The clay is dominantly montmorillonite and vermiculite.

Causes of dispersion (Perry, 1973) can be summarised as:

- Concentration of cations in the dilution (reservoir) water.
- Type of clay.
- The exchangeable sodium percentage (ESP), that is the ratio of single valence cations (sodium is the main concern) to the cation exchange capacity (CEC). This ratio is also referred to as the sodium adsorption ratio (SAR) but in this case only double valence cations are considered when calculating the cation exchange capacity. In summary:

$$ESP = \frac{Na}{Na + Ca + Mg + K}$$

$$SAR = \frac{Na}{0.5\ (Ca + Mg)}$$

In both cases cation values are expressed in milliequivalents/litre:

$$(\text{milliequivalent} = \frac{\text{milligrams of cation}}{\text{atomic weight}} \times \text{valence of cation})$$

This measures the total concentration of exchangeable monovalent and divalent cations in the double layer surrounding clay particles.

An extract analysis was carried out on two samples of completely weathered dolomitic siltstone (Table 7). Sample 1 was taken from the early inspection trench on the right abutment (possibly leached) and sample 2 from the spillway channel wall (construction block WL3). The latter sample is considered typical of unleached weathered dolomitic siltstone.

Possibility of Grout Curtain Failure

With a dispersive clay, small water paths may be rapidly enlarged under quite low pressure as the clay deflocculates and disperses into particles small enough to be carried by the water flow.

The lowest likely cation content of Adelaide water is 3 Meq/L according to the E & WS Dept laboratories at Bolivar. Therefore, from Figures 9 and 10, the clay fraction in Sample 2 is dispersive and Sample 1 may be dispersive (Sherard *et al.*, 1976).

It has been assumed that the weathered materials in Sample 1 and Sample 2 were originally similar. This seems likely because of the similarity of rock type from which they were formed. The difference in cation content could be due to differences in the amount of rain water percolating through the material and leaching out the cations. The leaching process removes monovalent in preference to divalent atoms which

Table 7 Results of extract analysis on samples of completely weathered dolomitic siltstone

	Sample 1		Sample 2	
	Mg/L	MeqL	Mg/L	MeqL
Na	43	1.87	710	30.87
Ca	90	4.5	202	10.1
Mg	23	1.9	138	11.4
Total		7.3		52.4
$\%Na = \frac{Na}{Na+Mg+Ca}$				
Sodium Absorption Ratio (SAR)		1.05		9.4

(E. & W.S. Dept. laboratory, Netley)

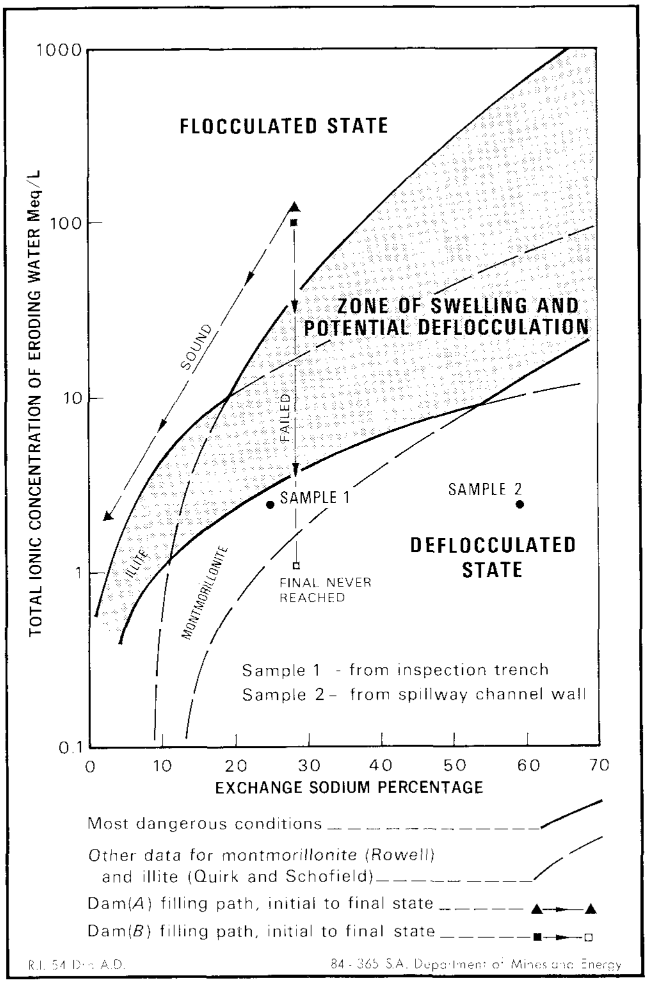


Fig. 9 Piping failure in earth dams.

would explain the lower sodium percentage in Sample 1. The rain water would also tend to disperse the clay in its natural state and this would assist the erosion process. Small holes might be enlarged by the percolating rain water even under low pressure. This is a possible cause of the cavities exposed in the early inspection trench excavated into the weathered dolomitic siltstone on the right abutment.

It is unlikely that cleaning and grouting operations have removed all erodible material from the area of the grout curtain and a dispersion failure is possible during the initial filling of the reservoir (Stapledon, 1977). However it is believed that the following factors will reduce the likelihood of grout curtain failure in the right abutment:

— The clay in samples taken near the outcrop of the seam is already leached, as in Sample 1, and is therefore less likely to give a future dispersion failure.

— Clay makes up only 10% of the weathered product. Silt and sand which make up most of the remaining material act as filters and should hold the dispersing clay (Perry, 1973). This has been observed on samples tested by the E & WS Dept from other sites.

— Cement in the grout has a high free-lime content which should help to stabilise dispersive clays as the calcium replaces the sodium.

— A dispersion test (crumb test) was done on the leached sample and no dispersion was noticed.

Nevertheless two precautionary measures have been carried out downstream of weathered seams known to occur within the right bank grout curtain.

Firstly, two interconnecting reverse sand filter trenches have been put into the right bank immediately downstream of the parapet wall. These have been excavated into the full thickness of the weathered seam and backfilled with washed concrete sand; they are intended to block any dispersion path that may form. The trench terminates downstream and away from the dam in a concrete trough to allow visual observation of any discharge.

A second precaution has been to drill two observation wells on the right abutment adjacent to the downstream toe, one downstream of the parapet wall and another at the same elevation as the shear zone running beneath grout cap Block M. They will allow measurement of groundwater potentiometric levels immediately downstream of the dam. The observation wells will be read daily until it is established that the grout curtain is performing satisfactorily, then they will be read weekly to monitor long term safety (See Part V: Surveillance Programme and Project Evaluation).

Permeability Tests

A sample of completely weathered dolomitic siltstone taken from the base of a stress relief fracture on the left bank grout cap alignment was forwarded to the laboratory. Grading curves, specific gravity, and permeability tests were carried out and results are included in Appendix 4. The sample showed traces of rock fabric but immediately decomposed after being placed in both distilled and mains water. A binocular microscope revealed the presence of many voids up to one and a half times the diameter of the silty grains. A mechanical analysis for this sample is given in Appendix 4. A permeability test gave an approximate value of 1.15 x 10^{-5} m/s and the sample had a specific gravity of 2.87.

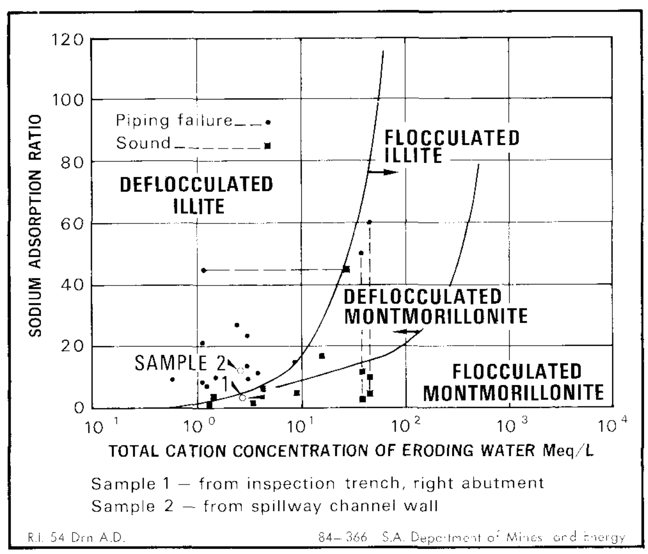


Fig. 10 Field performance of Australian earth dams.

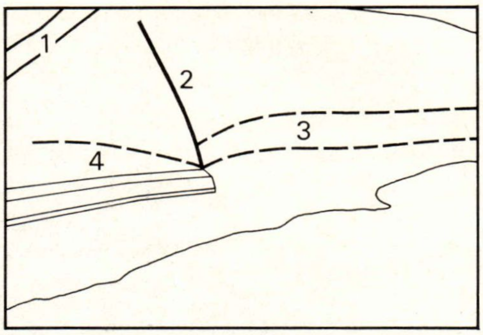


Plate 10
Left abutment and outlet duct—December, 1975 (T24378).
1. Left bank grout cap excavation.
2. Shear zone.
3. Quartzite (Unit 12) terminating against shear zone.
4. Gently folded dolomitic siltstone (Unit 13).

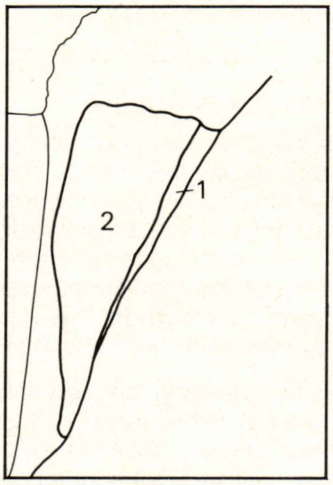


Plate 11
Spillway channel wall along ski-jump—July, 1976 (T12645)
1. Clay-filled joint daylighting into spillway wall.
2. Unstable wedge later covered with cyclone mesh and pinned with dowels.

Plate 12
Water treatment works under construction in quarry area—March, 1984 (T24683).

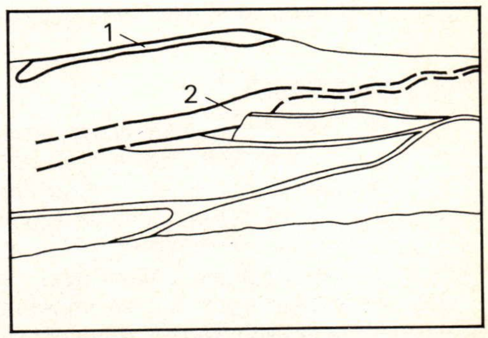


Plate 13
Quarry and site for water treatment works—April, 1976 (T24379).
1. Quartzite (Unit 12).
2. Quartzite (Unit 9) thickened by folding.

Plate 14
Water pressure test on grout cap Block 5. Canvas tape is being lowered down hole to detect Rhodamine B dye which can be seen staining concrete (T24380).

Although the permeability of foundations may be reduced by embankment pressure (Londe, 1977; Stapledon, 1976) these values of permeability are for isolated seams of completely weathered dolomitic siltstone which may occur at depth below the embankment. It is believed that the permeability of this material will not be altered by the overlying 50 m high rockfill embankment. The treatment of completely weathered seams is discussed in Part III; Design and Construction.

Testing of three tube samples taken at the top of the right bank grout cap from the sides of the parapet wall foundation trench, which was excavated into a seam of completely weathered dolomitic siltstone, gave a permeability value of 5×10^{-7} m/s.

The two permeability results give a range of values of water movement from approximately 3 to 15 m/year and the material is classified as slightly permeable (Appendix 5) indicating that leakage through the grout curtain, as a result of piping within the completely weathered dolomitic siltstone, is not likely.

Ring Density Tests

Six ring density tests were carried out on the embankment in rockfill Zones 1, 2, 3, 4 and 6 for on-site quality control. Each covered the full thickness of the 1 m compacted layer and compacted rockfill was excavated by hand pick and carefully weighed ensuring no loss of fines. Volume was calculated by lining the excavated hole with an impermeable membrane and filling it with water. A summary of the ring density tests is shown in Appendix 4.

Average results gave dry densities in the range of 2 010 to 2 280 kg/m³. An early ring test on Zone 1 gave a low dry density value of 1 840 kg/m³ resulting from accidental mixing with unacceptably low quality Zone 2 material. The top 0.5 m of this contaminated material was removed and replaced with clean fresh quartzite. Up to this stage the embankment had been compacted dry but, because the general rock quality had deteriorated, from then onwards water was sprayed onto the rockfill during compaction (Plate 7).

PART III: DESIGN AND CONSTRUCTION

Geology, Dam Design and Construction

Geological data was presented in three stages. Firstly the design engineers received results of the initial geological investigation; this allowed the choice of the dam site and dam type. Secondly, usually at the request of the designer, specific geological data required for a particular aspect of the project was presented. Thirdly, the site geologist made a daily inspection of construction work and reported to the construction engineer. Wherever the exposed geology proved different to expected geology the designers were informed who then, if necessary, inspected the geology onsite. In certain cases minor design modifications were required but, more normally, continued excavation to sounder rock or the remedial treatment of unacceptably poor quality rock sufficed to allow construction to continue.

A summary of the different construction features which required geological attention is shown in Figure 11.

The Reservoir

Although early geological mapping proved a water tight reservoir, an area of concern for possible leakage was on the right abutment adjacent to the dam where completely weathered outcrops of dolomitic siltstone occur immediately upstream of the dam (Fig. 12). A weathered seam of dolomitic siltstone passes beneath the right abutment near to the full supply level of the dam and it was thought that impounded water could find a leakage path beneath the right abutment of the dam through these seams. For this reason the possibility of sealing any potential leakage path by placing a clay blanket across outcrops of the weathered seams was considered. It was decided not to do this because the durability of such a blanket on the hilll slope would be affected by the continual filling and drawdown of the reservoir. It was also considered that a grout curtain would obviate the need for a clay blanket.

Abutments

Right Abutment

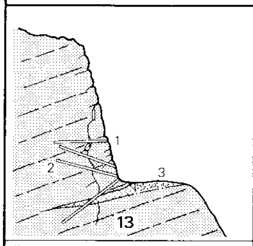
Trenching on the right abutment had exposed three small cavities associated with weathered dolomitic siltstone found at the base of the quartzite bed, Unit 12 (Plate 6).

The extent of the cavities and possible affect upon the grout cap and grout curtain was assessed by diamond core drilling. Drillholes along the grout cap and into the right abutment indicated that, although the cavities were not widespread, weathered seams of dolomitic siltstone occurred beneath the quartzite stratum and they should be intercepted when drilling grout curtain holes.

During cleanup of the right abutment, quartzite bedrock was exposed after removal of up to a metre of topsoil and loose moderately weathered quartzite boulders. Removal of calcrete, localised at the top of the right abutment and up to 800 m thick, exposed completely weathered dolomitic siltstone. This weathered strata was trenched up to 4 m into moderately strong dolomitic siltstone. The trench was filled with concrete and served as the foundation of the parapet wall.

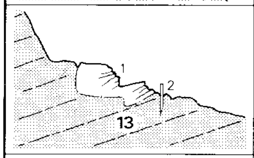
The quartzite proved to be weathered and fractured adjacent to a shear zone traversing the right abutment and passing through grout cap Block M (Fig. 5). Ease of bedrock removal resulted in inadvertant excavation of a wide trench up to 20 m long into the abutment beneath the proposed embankment (Plate 4). The trench ran along the shear zone and was approximately 5 m wide with walls up to 5 m high; these walls were permanently damp where the shear zone was exposed. In addition the excavation exposed an early exploration drillhole which flowed constantly at approximately 0.25 L/s.

LEFT ABUTMENT — REMARKS



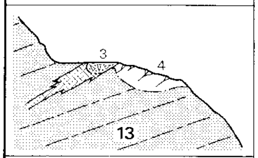
INVESTIGATION OF HILLSIDE STRESS RELIEF FRACTURE

1 Airtrack drillholes used to check location of fracture.
2 Small diameter cored holes to inspect rock quality adjacent to fracture; also used as permanent drainage holes during reservoir drawdown.
3 Dental treatment of completely weathered dolomite seam associated with base of fracture.

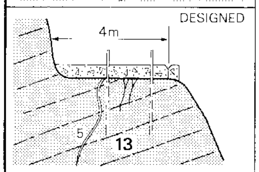


CONSTRUCTION OF GROUT CAP

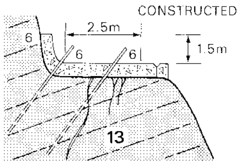
1 Removal of fissile moderately weathered dolomite by jackhammer and manual picking.
2 Extra standpipe for grout injection located in open joint.



3 Dental treatment of completely weathered dolomite seam.
4 Removal of superficial weathered bedrock.

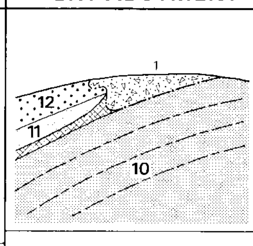


5 Location of open joint makes grout cap unacceptable both for stability and effectiveness of grout curtain.



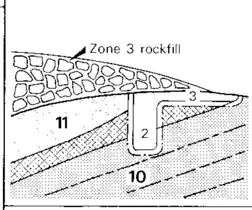
6 Grout cap redesigned by extending up batter; two rows of grout pipes were moved away from the hillslope and disturbed rock, then angled through the batter and into the hill.

RIGHT ABUTMENT — REMARKS

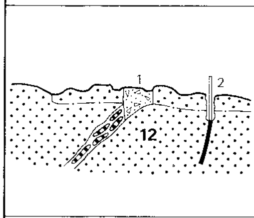


TREATMENT OF WEATHERED DOLOMITE

1 Excavation of trench beneath parapet wall to remove completely weathered dolomite up to 3 metres thick. Infilled with concrete.

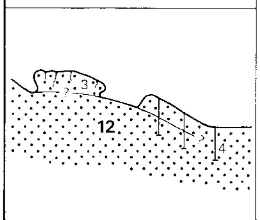


2 Reverse filter placed in trench excavated into completely weathered dolomite to intercept any water in Zone 3 rockfill downstream of the grout curtain.
3 Sand blanket over the outcrop of completely weathered dolomite not excavated, and contiguous with material in the trench.



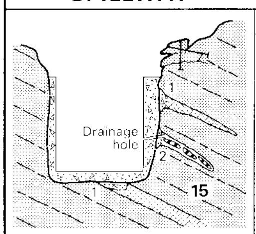
CONSTRUCTION OF GROUT CAP

1 Dental treatment of vuggy quartz vein.
2 Extra standpipes for grout injection located in opened and cleaned out area of shearing; dental treatment also used.



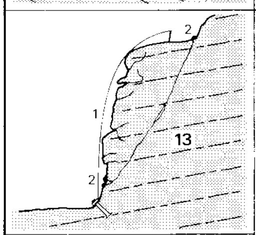
3 Removal of doubtfully stable boulders by jackhammer or pop blasting.
4 Tracing open flat lying fracture with airtrack drill and hand probe. Rock removed by hydraulic rock splitter to avoid use of blasting owing to proximity of outlet tower.

SPILLWAY — REMARKS



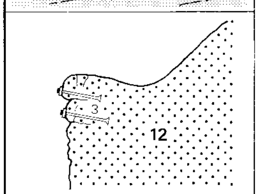
TREATMENT OF WEATHERED SEAMS

1 Dental treatment of weathered dolomite.
2 Dental treatment of vuggy quartz vein; drainage hole required.



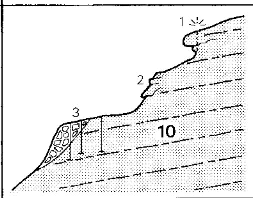
TREATMENT OF UNSTABLE STRATA

1 Cyclone mesh wrapped over spillway channel in ski jump section to resist movement of unstable blocks during construction of lower levels of the ski jump.
2 Mortar tell-tale pads placed over fracture forming potential wedge failure surface.



3 Two rows of 4.1m rock bolts were located individually at 1.5m centres to strengthen edge of spillway lip. The holes were drilled at 10° dip and the bolts grouted over the bottom metre.

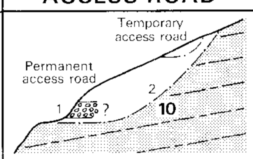
QUARRY — REMARKS



PREPARATION OF BATTERS AND BERMS

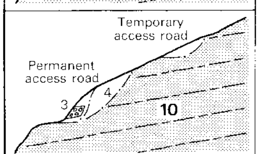
1 Blasting away overhanging large blocks.
2 Barring off unstable boulders along edge of batter prior to formation of underlying berm.
3 Checking thickness of colluvium included in large berm with airtrack drill.

ACCESS ROAD — REMARKS



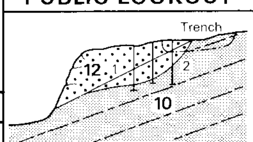
BATTER EXCAVATION

1 Excavation reveals gravel of high level river terrace.
2 Terrace may require possible deep excavation which would undercut temporary access road.



3 Excavation with backhoe proves terrace very limited.
4 Shallow excavation satisfactory and does not undercut temporary access road.

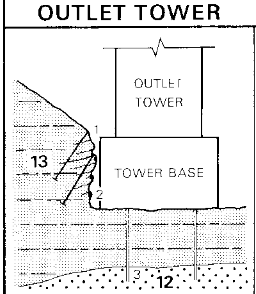
PUBLIC LOOKOUT — REMARKS



BATTER STABILITY

1 Assumed base of quartzite stratum.
2 Proven base, following airtrack drilling and trenching across quartzite–dolomite contact, indicates a more stable excavation.

OUTLET TOWER — REMARKS



SITE PREPARATION

1 Dowel 7m×10m through stress relieved dolomite slabs and interbedded 2–5mm thick clay seams.
2 Tell-tale mortar pads to check for further stress relief; none recorded.
3 Core drilling to find good quality strata for anchoring prestressed bars of tower base.

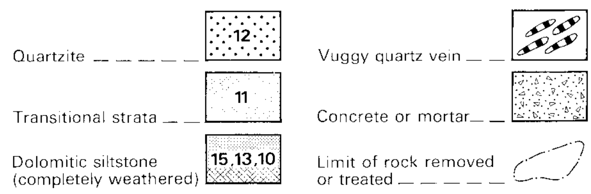


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Fig. 11 Treatment of geological features during construction.

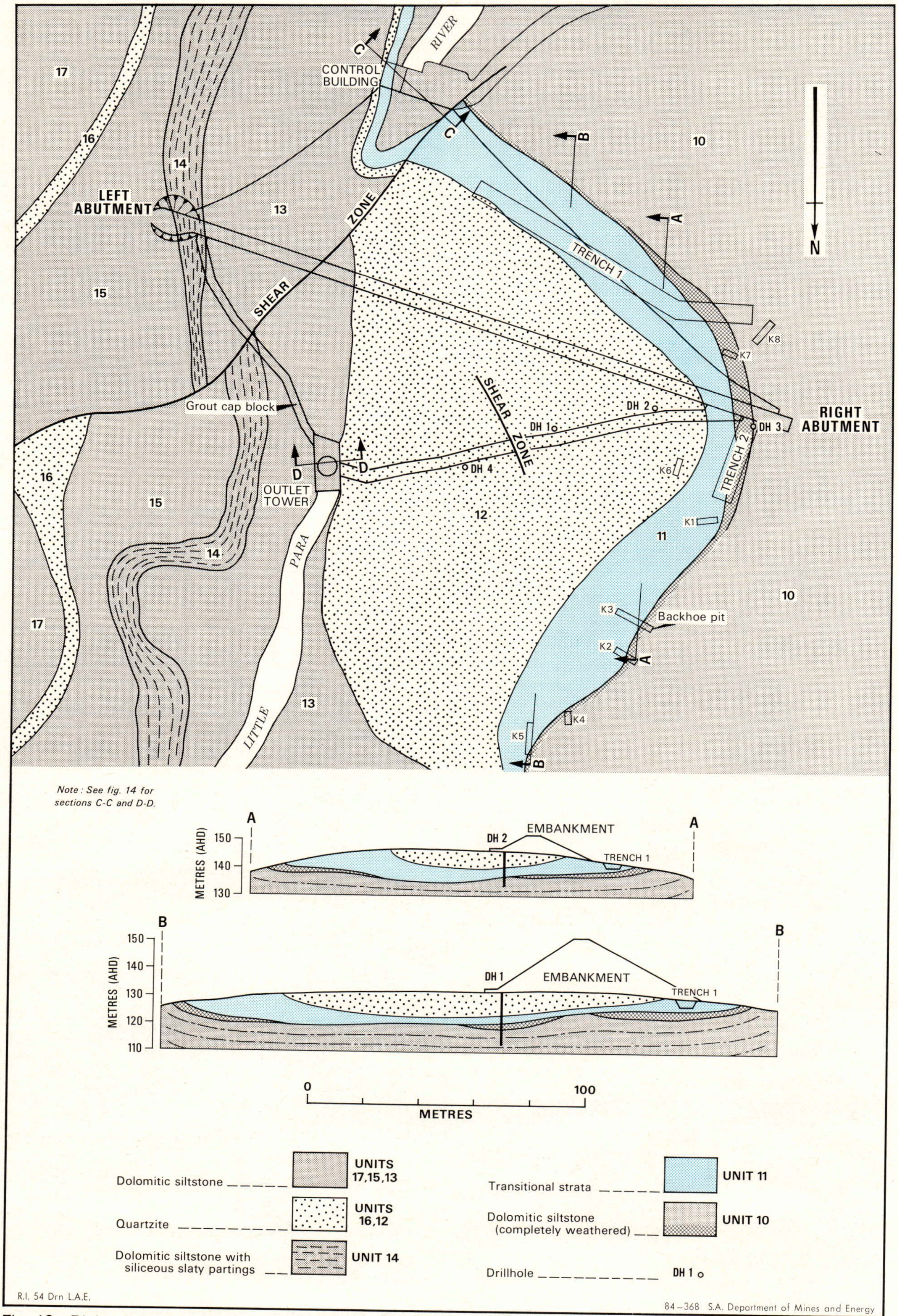


Fig. 12 Right abutment: Geological plan and sections.

To avoid future saturation of overlying rockfill the trench was filled with Zone 1 quality material connected with free draining Zone 4 material at the base of the dam.

Left Abutment

Because of the sparse vegetation, little clearing of the left abutment was necessary and was carried out during placement of the embankment. A cavity, discoid in shape and approximately 4 m long, 4 m high and 1.5 m wide, occurred at El. 114 m fifteen metres downstream of the grout cap and located along a hillside fracture. Solution and fretting of the dolomitic siltstone had widened the fracture locally, and the resultant cavity did not appear to be continuous into any other part of the abutment. All loose slabs were removed by backhoe and the cavity filled with grout injected through airtrack holes drilled into the top of the cavity.

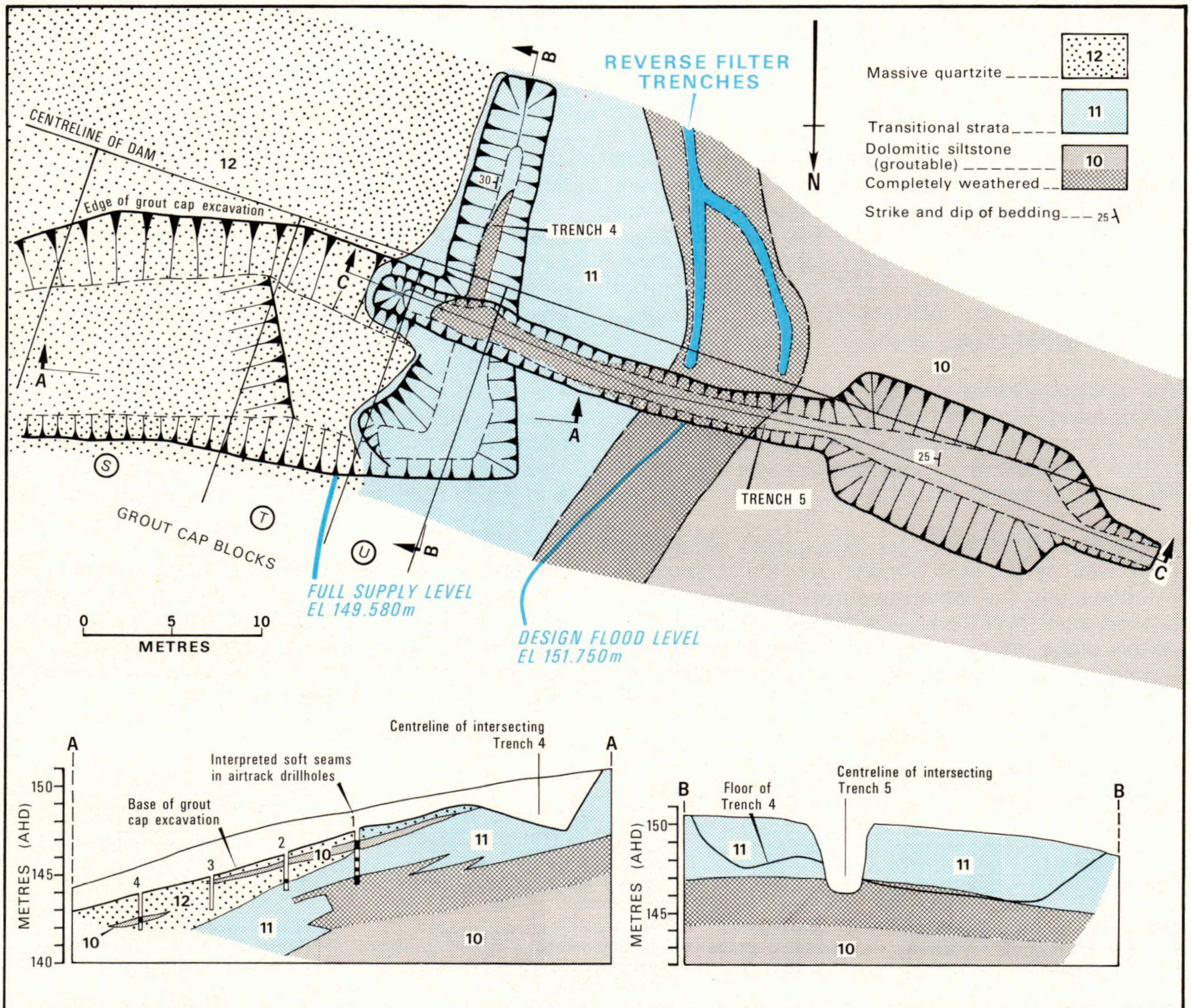


Fig. 13 Right abutment: Filter trenches.

Completely weathered dolomitic siltstone was exposed between El. 121 m and El. 124 m, extending 20 m downstream from the grout cap. After removal by backhoe down to less weathered siltstone, Zone 1 rockfill was placed in a trench excavated between the grout cap and the downstream face and running adjacent to the left bank abutment. This was done to channel any leaking reservoir water away from embankment rockfill. In retrospect the use of concrete instead of Zone 1 rockfill would have been preferable, reducing seepage velocities and hence decreasing the chances of solution and erosion of the seam of completely weathered dolomitic siltstone.

Grout Cap and Parapet Wall

The alignment of the grout cap was chosen following diamond drilling results along a proposed alignment selected as a result of the early geological assessment of the site. After drilling, overburden along the alignment was removed using high pressure water sluicing. The alignment was subdivided into grout cap blocks and each block was individually inspected and geologically logged (Figs 6 and 7).

The grout cap blocks are labelled from A to X excluding letters I and O. On the left abutment the blocks run from A at the top down to G and from X down to H on the right abutment. The control tower foundation lies between Blocks G and H.

Excavation of the grout cap and batters on the left bank was by pre-split blasting and the rubble was removed using a winch-secured bulldozer which pushed waste over the hillside into the river bed below. At this stage the river had been temporarily diverted through a 1.5 m diameter pipe and no material entered the river.

Excavation along the right bank grout cap did not require blasting. Apart from some areas requiring dental treatment, sound fresh to moderately weathered bedrock was exposed, requiring very little preparation and showing no surface deterioration between the period of excavation and grout cap concrete laying (contrast this with Roberts *et al.,* 1975).

Geological Examination of the Grout Cap

The geological conditions encountered along the grout cap alignment and subsequent treatment are shown in Figs 6, 7 and 11 and are summarised in Table 8.

Approval for pouring the grout cap concrete was given only after two criteria had been satisfied. Firstly, there had to be a minimum width of 4 m of capping. It is within this width that the two rows of grout curtain holes were drilled (Plate 8).

Secondly, all loose or highly weathered bedrock had to be removed and infilled fractures widened and cleaned out and, if necessary, filled with dental concrete (Plate 8). Before the concrete for each grout cap block was poured the reinforcing bars were laid and attached to dowel bars grouted 3 m into bedrock. The dowel holes also provided an indication of the bedrock quality at shallow depth. In particular the method allowed persistent open fractures to be traced across the grout block by using a hand-held steel rod with a thin rigid tip welded at right angles onto one end.

Because of these weathered seams early consideration was given during design of the dam to placement of a reverse filter (Fig. 13) at the top of the right abutment, beneath the downstream embankment rockfill, as a precaution against dispersion and subsequent erosion of the weathered dolomitic siltstone.

Table 8 Preparation of grout cap blocks

Feature	Location	Treatment of Block	Modification of Design
Loose slabs of dolomitic siltstone resting upon slightly to moderately weathered siltstone.	Left bank.	Surface removal only providing siltstone is not too weathered (slightly-moderately weathered is acceptable).	If associated with open joints may require extra grout hole.
Loose slabs of dolomitic siltstone resting upon highly to completely weathered siltstone.	Left bank at base of outlet tower.	Removal of slabs underlying weathered seam trenched out where necessary to approx. 1 m depth and infilled with concrete.	Sometimes necessary to extend grout cap up onto the hillside batter to obtain 4.0 m design width.
Slightly to completely weathered siltstone or quartzite associated with seams parallel to bedding or open joints.	Left and right bank.	Dental treatment as above.	Place extra stand pipe for grouting in the open joint.
Intersecting joints producing a highly to completely weathered area in quartzite usually about a metre square; often associated with quartz veins.	Right bank.	Completely removed usually down to 1.5 m and infilled with concrete.	
Presence of non-groutable seams up to 100 mm thick beneath acceptable quartzite boulders.	Right bank beneath outlet tower.	Removal of boulder and thin seam. Persistence of seam checked by small diameter jackhammer drilling.	
Presence of completely weathered dolomitic siltstone up to 3.0 m beneath quartzite.	Right bank beneath Block U.	Complete removal of dolomitic siltstone along a trench parallel to dam axis beneath the parapet wall.	Trench infilled with concrete and formed contiguously with the parapet wall. Reverse filter placed beneath rockfill across full dam width in this area.
Calcrete adhering to grout cap block surface.	Right bank.	Removal of all but the hardest calcrete.	

Based on the evidence of exposures during construction, this was eventually deemed necessary. A trench up to 6 m deep was therefore excavated on the right bank grout cap beneath the proposed parapet wall across Blocks T and U to remove completely weathered dolomitic siltstone up to 3 m thick, underlying up to 2 m of transitional strata (flaggy quartzite) beneath Block T and cropping out beneath the parapet wall.

The occurrence of vuggy quartz veins normally coincided with extensive weathering of the dolomitic siltstone and quartzite; these were removed by pick and shovel or jackhammer.

On the right bank grout cap large boulders, formed by open joints in the quartzite, rest upon a quartzite bedding plane and thin clay seams developed along this made removal of the boulders necessary (Plate 9). This was achieved by popping the boulders with small explosive charges or, where within 20 m of already placed concrete of the outlet tower, by use of an hydraulic rock splitter.

On the left bank grout cap six 25 mm diameter drainage holes lined with slotted PVC piping were drilled into the base of the open hillside stress relief fracture shown in on Plate 8. Seams of completely weathered dolomitic siltstone and weathered material associated with the vuggy quartzite veins of the shear zone were removed and the grout cap realigned in places to avoid areas of persistent open joints and loose slabs.

Grout Curtain

A detailed discussion of procedures used in grouting the curtain and of the various methods of checking its effectiveness is given in Part IV: Grouting Programme and Grout Curtain Investigation.

Early investigation of the proposed grout curtain alignment included diamond drilling and water pressure testing on the left abutment down to 20 m. This indicated that fairly low grout takes should be expected. On the right abutment, drilling showed that the quartzite (Unit 12) lies above dolomitic siltstone and one of the early diamond drillholes put down near Block P was checked by TV camera in order to examine an 800 mm core loss near to this contact. This loss was seen to be restricted to one side of the drillhole and of limited width. Although persisting laterally beyond the focusing range of the TV it appeared to narrow quickly. Therefore at the early stage of assessment only small grout takes were anticipated, providing that all cavities occurring within the grout curtain were similarly small in dimension. However, earlier trenching into the transitional strata showed the presence of cavities with a minimum dimension of 0.5 x 0.5 x 2.0 m. The trenching also revealed completely weathered dolomitic siltstone seams.

It was vital that the grout curtain achieve a water tight foundation, particularly on the right bank, and grouting was carried out up to a maximum pressure of 500 KPa.

Outlet Structures

Rock quality of the abutments at Little Para Dam was too poor to allow construction of a diversion tunnel as at Kangaroo Creek (Trudinger, 1973). Instead, an outlet duct has been constructed passing beneath the rockfill embankment connecting the outlet tower to the control house. An inclined outlet tower constructed on the valley wall was precluded because of unsuitable topography, and a vertical outlet tower has been built. The tower will exhibit a small positive buoyancy when the reservoir is full and its base has been post-tensioned to bedrock. This will assist stability under seismic forces.

Because there is no obvious surface expression of the shear zone which runs up the left abutment it was considered possible that the quartzite could have been displaced by shearing parallel to its bedding plane. In such a case the shear would run along the line of the river, passing beneath the full width of the dam and beneath the outlet tower. Diamond core drilling in the creek bed was carried out to assess rock quality for the outlet tower. This gave no evidence of shearing and therefore the original location of the shear zone, across the left abutment, was considered proven.

Drill cores from the outlet tower foundation proved the presence of quartzite and dolomitic siltstone (Fig. 14). The dolomitic siltstone (Unit 13) occurs between the base of the tower and the underlying quartzite (Unit 12) and is from 2 to 8 m thick. There was no indication within the dolomitic siltstone of weathered seams or clay seams which could be compressed during tensioning of the prestressed bars. The quartzite is fresh, strong to very strong, and exhibits tight to cemented vertical joint planes, 500-3 000 mm apart, providing adequate anchorage for the grouted prestressed bars.

During excavation of the outlet tower foundations, stress release of strata was seen in dolomitic siltstone in a near-vertical exposure at the base of the left abutment. Individual strata moved outwards up to 20 mm into the excavation immediately after blasting. Telltale mortar pads showed no further movement over the four months that the excavation remained open. Nevertheless, concrete for the tower base was not placed directly against the excavated face but was formed away from the face and the gap backfilled with rubble to prevent any damage due to further movement. Excavation along the river bed exposed acceptable bedrock for the Outlet Duct Foundation. The shear zone was also exposed and proved to be tight and unweathered and no design modification was thought necessary where the duct traverses the shear zone (Plate 10).

A seismic refraction investigation immediately downstream of the dam (Fig. 14) indicated shallow bedrock at a maximum depth of 3 m beneath the control house and dissipating chamber located at the downstream end of the outlet duct (Gerdes, 1973). Excavation confirmed an acceptable foundation not affected by

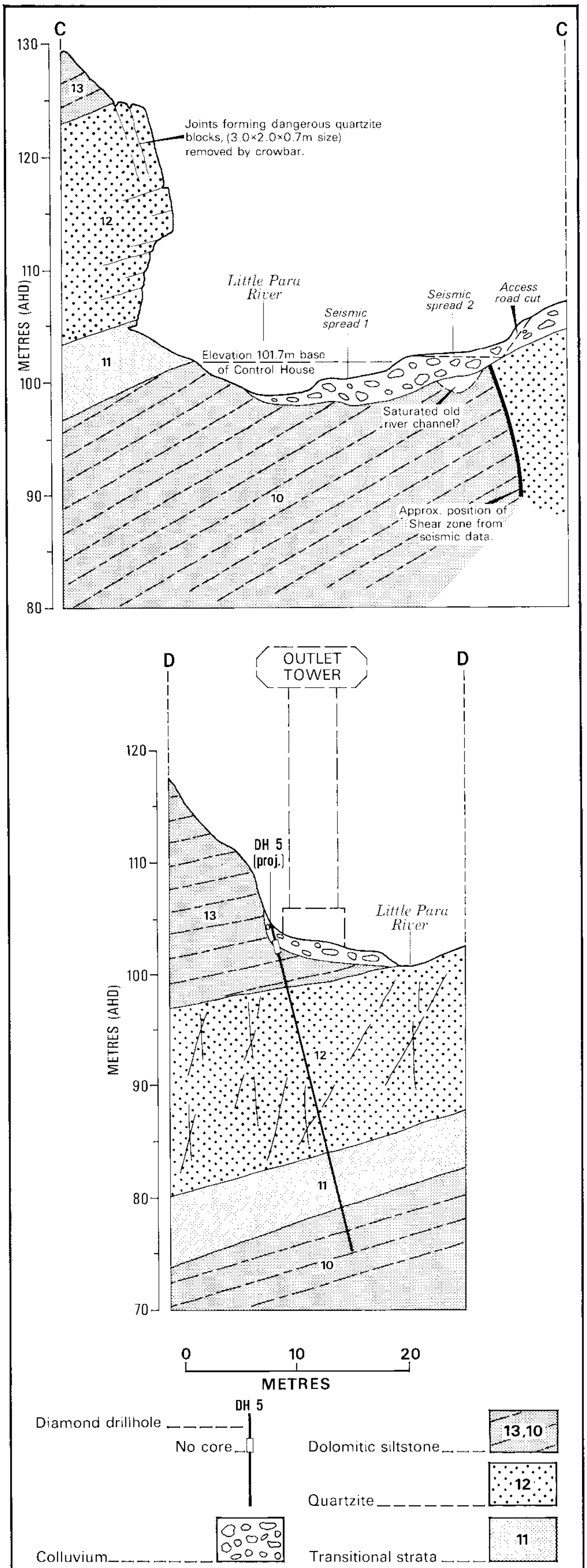


Fig. 14 Outlet tower and control house: Foundation conditions.

the shear zone, although the seismic work showed a change in bedrock nature, possibly caused by shearing, some 10 m west of the control house.

Spillway

The spillway is located across a ridge of quartzite and dolomitic siltstone.

Results of a joint and bedding survey advised a preferred orientation of between 95° and 100° for the spillway alignment commensurate with maximum stability of the spillway batters. This orientation could not be exactly followed owing to spillway design requirements for water discharge into the Little Para River and an alignment of between 80° and 85° was chosen.

Seismic survey traverses (Fig. 15) were carried out over the area of spillway discharge. These were to ascertain depth to sound bedrock and to ensure there would be no pot-holing beneath the spillway leading to undermining of the left bank cliff face beneath the spillway lip and downstream of the embankment rockfill. The seismic work proved competent rock at depths of less than 5 m below the existing valley floor.

The alignment of the spillway was chosen to allow discharge of flood waters straight into the river in this area of proven shallow bedrock. This discharge direction also minimises the amount of spray reaching the all-weather access road on the opposite right bank hillside.

Using a scale hydraulic model, the backwater effect created by the quartzite bar (Unit 9), which crosses the Little Para River at the site of rapids a short distance downstream of the spillway discharge, was studied to see if it encroached upon the downstream toe of the dam. The hydraulic model studies showed that this would not occur.

During spillway construction the spillway floor and channel walls were geologically mapped (Fig. 16). Particular attention was paid to seams of completely weathered dolomitic siltstone requiring dental or drainage treatment to prevent erosion which could undermine the concrete lining.

Unstable boulders along the top of the spillway excavation were either removed or stabilised by grouted dowel-pins.

Two diamond drillholes proved strong competent quartzite bedrock (Unit 12) beneath the base of the spillway ski-jump. The flip bucket at the base of the ski-jump was located to take advantage of this quartzite which has sufficient strength to resist resultant downward forces during spillway discharge. The quartzite cliff beneath the lip of the ski-jump was strengthened by using 4.1 m rock bolts individually located at approximately 1.5 m centres and tensioned to a maximum of 890 N (Fig. 11).

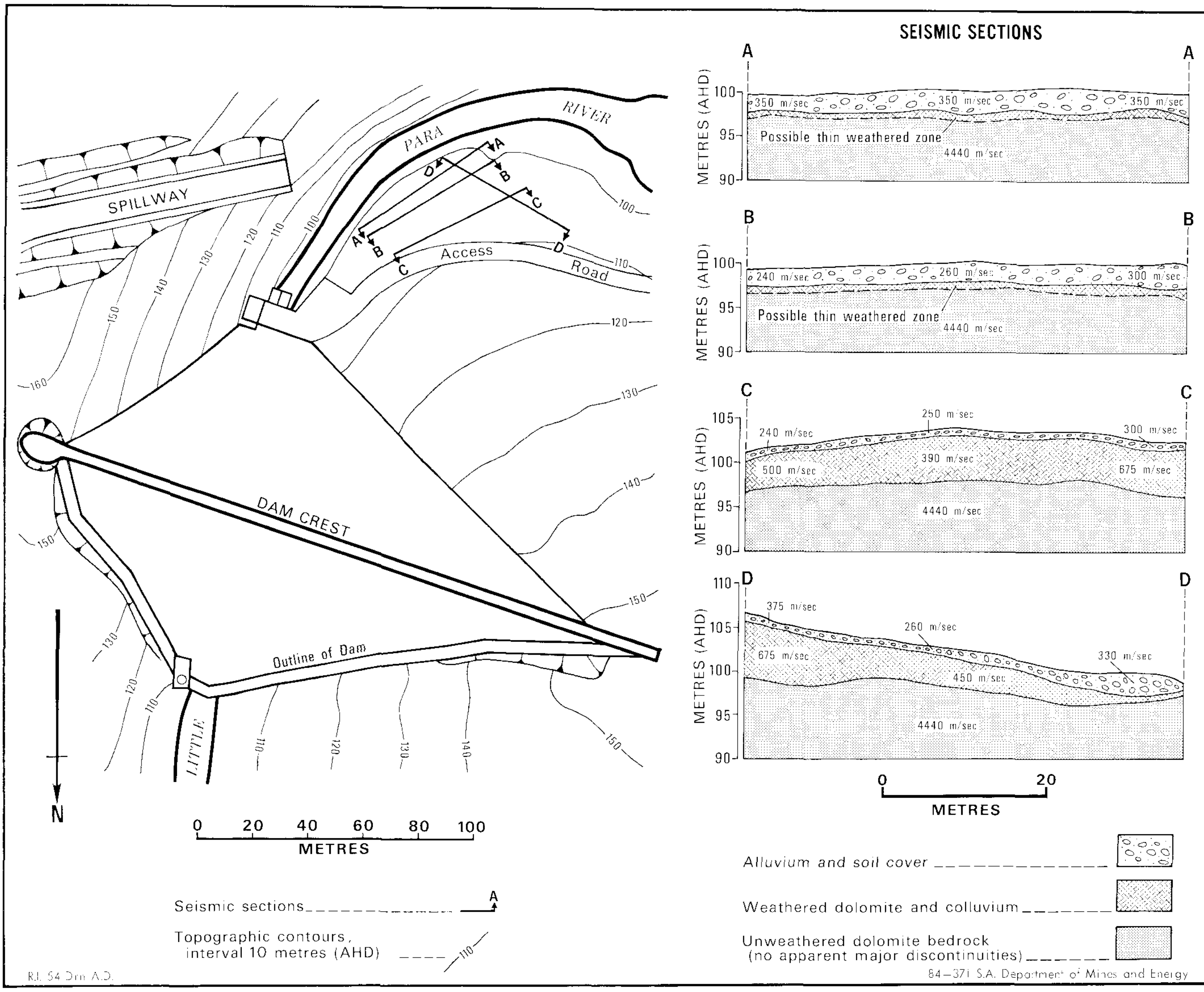


Fig. 15 Spillway discharge area: Seismic traverses.

Assessment of Batter Slopes

Examination of block sizes for the two rock types and of stereogram data (Appendix 3) resulted in an initial recommendation for a left hand batter of 75° and for a vertical right hand batter. However, as block sliding could occur on the right hand side along the bedding it was decided to excavate both batters above the channel at 75°. The lower portion of the channel, which was to be concrete lined, was to be excavated with vertical batters with the concrete to be placed directly against the rock.

Channel Walls

Rock quality and orientation of defects were used to assess stability as each section of the channel was excavated.

The face of the excavated vertical-walled portion of the channel was inspected to assess whether a free standing or non-free standing concrete wall was required. Where the face was of weathered or loose rock the channel wall was designed as a retaining wall. This was necessary for the full length of the right hand side of the ski-jump and for both right and left walls of the flipbucket. In the case of the ski-jump, excavation was close to the ground surface and rock quality was not adequate for the channel wall. For the flip bucket, walls of a specific design were required to withstand the discharge thrust and to deflect flood water into the river bed.

A free standing channel wall was also designed for a part of the left side ski-jump where excavation of the channel face had intersected a low angled, clay lined joint plane (Plate 11). Daily monitoring of small mortar pads placed across the joint plane showed no evidence of movement after exposure. During construction the face was pinned with dowels and covered with cyclone mesh fence.

Excavation of the right side channel wall at the top of the ski-jump exposed a hillside stress relief fracture. Up to 12 mm of displacement up-dip of the bedding plane was observed. Mortar pads placed across the fracture showed no evidence of movement and it was concluded that the concrete channel wall could safely be placed against the excavated face.

The main materials quarry supplied the bulk of the embankment rockfill and only a small amount of rock from the spillway excavation was utilised. Nearly all of this was of Zone 3 quality although the quartzite strata traversing the top of the spillway did provide some Zone 4 and Zone 2 rockfill.

Quarry

Investigation for a suitable quarry site ran concurrently with the dam site investigation and altogether five quarry sites were inspected. The first three were rejected because of unacceptable rock quality and the fourth on aesthetic grounds because the proposed site was at the top of a prominent hill. The site chosen was towards the base of this hill but at a lower level where it could provide both Zone 3 rock and good quality rock for Zones 2 and 4; also when excavation was completed the quarry site would provide a convenient location for the water treatment works (Fig. 5 and Plate 12).

A fracture survey was carried out over the quarry area to assist in the design of safe batters and to establish the safest excavation sequence (Appendix 3).

The main features of the survey are shown in Figure 17.

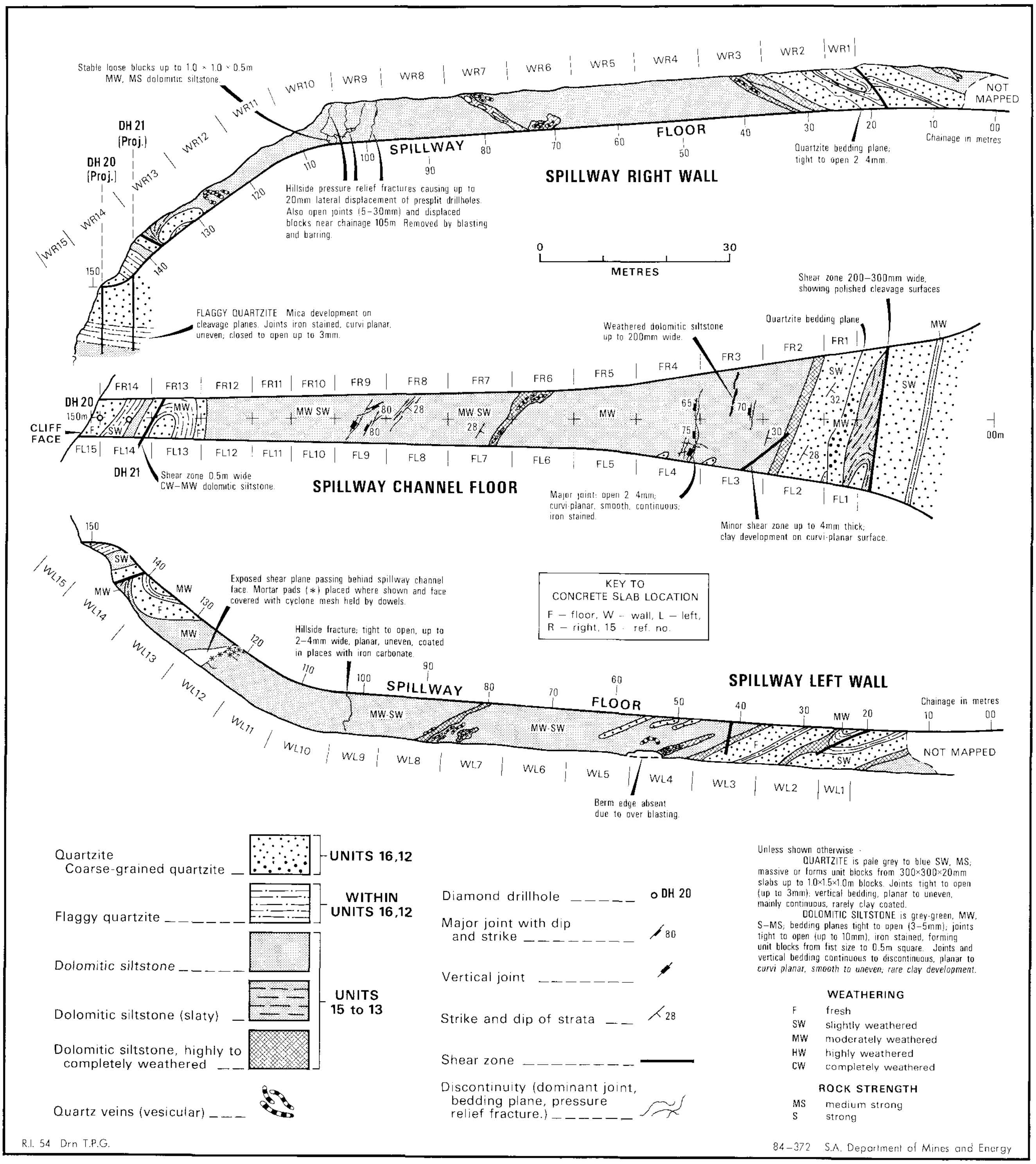


Fig. 16 Spillway: Geological log.

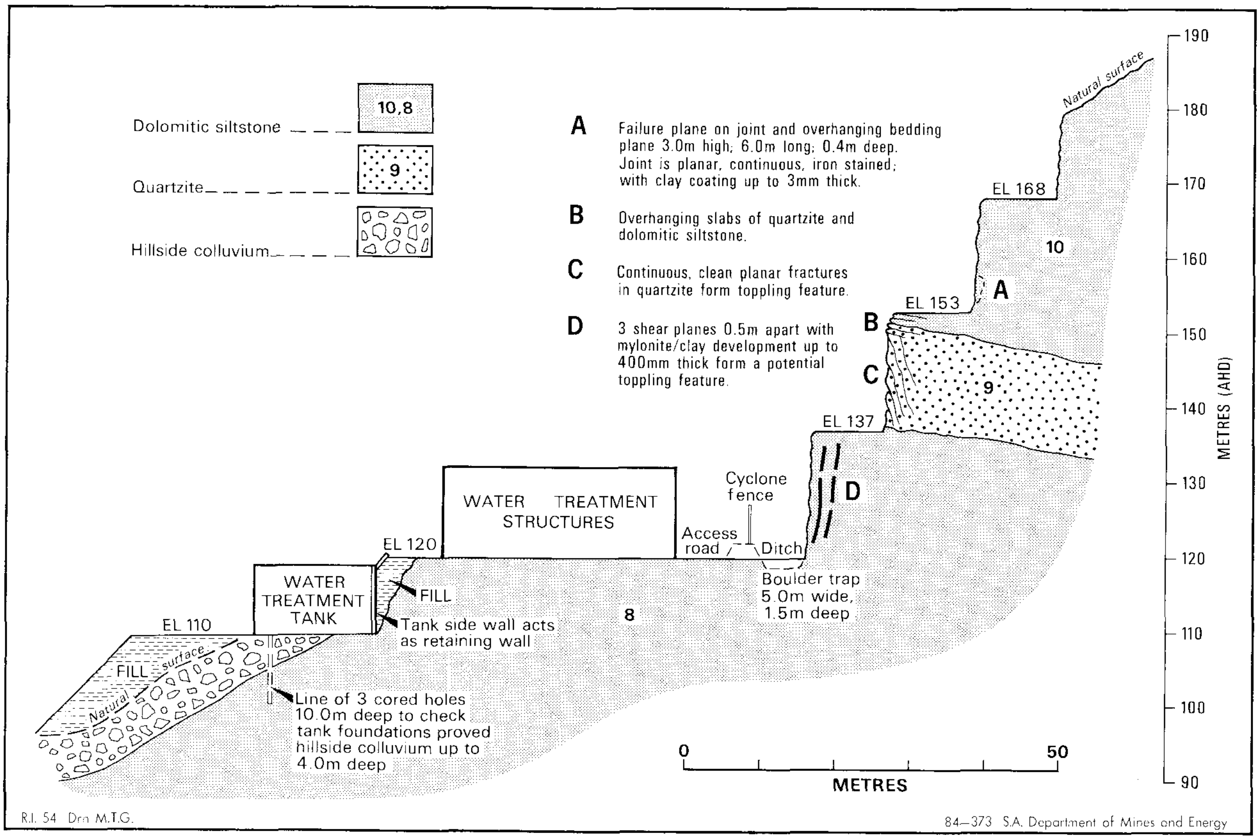


Fig. 17 Quarry: Slope stability.

Surface mapping and the results of two diamond drillholes indicated that although the main bulk of the material in this site would be dolomitic siltstone the presence of a quartzite bed (Unit 9), thickened to twice normal by folding, would supply the required amount of Zone 4 and Zone 2 material. Calculations showed that there would be sufficient quantities of dolomitic siltstone to supply nearly all Zone 3 required for the rockfill embankment.

Quarry Excavation and Stability

The folded quartzite (Unit 9) which traverses the quarry site affected batter stability during excavation of the quarry and for this reason two fracture surveys were carried out.

The first survey assessed the short term stability during the period of excavation and the second the long term stability of batters completed (Appendix 3) to ensure the safety of the water treatment structures to be built on the two main berms at El. 110 m and El. 120 m (Plate 13).

A safe sequence for quarry excavation was proposed:

— Removal of large unstable blocks in quartzite (Units 9 and 12); for aesthetic reasons this was to be kept to a minimum where Unit 12 formed a skyline.
— Excavation of Unit 9 along the central part of the quarry main face down to 3 m depending on the depth to which folded strata dipping into the quarry can be removed.
— Commence excavation of dolomitic siltstone from the western end of the quarry, working eastwards.

Main fractures in the quarry area are bedding and near-vertical joints. Both fracture surfaces may be fresh or lightly iron stained, planar to curviplanar, uneven to smooth. The main fissile plane is the cleavage which deviates from the true bedding plane by only 2-3°; it differs from the jointing by being continuous and by not showing clay development upon its surface.

Quartzite blocks sliding into the quarry along folded bedding planes caused the main concern for safety during excavation. Stability and hence site safety improved after removal of the northern limb of the shallow anticlinal fold.

Because the bedding dips easterly, blocks could be expected to slide out of the batters along the western face of the quarry. Friction tests carried out on dolomitic siltstone (Appendix 3) proved the possibility of such sliding and an angle of 40° was recommended for the western batters. Vertical batters with 5 m wide berms were recommended for the rest of the quarry.

The maximum height of vertical batters is controlled by State Mining Regulations at 15 m. Boulders rolling down an angled surface could gain sufficient velocity to overshoot the berms and damage the water treatment works. For this reason vertical batters were preferred and any unstable wedges allowed to fall vertically onto wide berms. A low safety barrier is to be erected adjacent to the water treatment works to intercept any rolling boulders. The design of the fence and ditch excavated at the foot of the berm can be calculated to give maximum safety to the water treatment works (Fookes *et al.*, 1976). At this site restricted space demands a narrow deep trench and a 3 m high safety fence. The size of the quarry was restricted by topography and vertical batters also assisted in limiting the areal extent of the quarry and in reducing the volume of material excavated.

Embankment

The 54 m high embankment has three rockfill zones (Fig. 8). Zones 1, 2 and 4 are free draining and designed to channel water penetrating the concrete face to the foundation away from Zone 3, the main mass of the embankment.

Zone 6 lies above and around the outlet duct and serves to reduce stresses on the duct resulting from the weight of the overburden (Plate 5). Zone 5 is a zone of poor to moderate quality rock placed over the foot of the parapet wall and upon which runs the dam axis access road.

The bulk of the embankment (moderately strong, moderately weathered dolomitic siltstone) was placed wet and compacted with four passes of a 10 t vibrating roller.

Settlement of the embankment during and after construction was measured using hydraulic settlement gauges. Deflection of the embankment was also measured (Appendix 2).

PART IV: GROUTING PROGRAMME AND GROUT CURTAIN INVESTIGATION

Introduction

A double curtain 21 m deep was cement grouted in three stages of 7 m using a downstage non-packer technique at maximum pressures of 110 KPa, 250 KPa and 500 KPa for the first, second and third stages respectively. This method of grouting, using a grout nipple on a stand pipe, proved time-consuming but reliable and effective. In the area of Blocks M and N rock uplift of from 1 to 4 mm occurred during grouting; here stage depths and pressures were reduced and upstage grouting with a packer used in combination with downstage grouting. Primary grout holes were first drilled at 6 m intervals and then repeatedly halving the distance for subsequent secondary, tertiary and quaternary holes. Water pressure testing of the foundations was carried out before and after grouting of the curtain. Details of the grouting procedure are discussed in Appendix 1.

During the construction period a research programme was undertaken to test the effectiveness of the grout curtain, particularly across the areas where seams of weathered dolomitic siltstone occur. Geophysical measurements and isotopic grout methods were used to see if it was possible to differentiate between ungroutable and groutable foundation rock.

Grouting Programme

Curtain grouting at Little Para Dam consisted of drilling two rows of holes (45 mm diameter) through the grout cap. The effective depth of this double curtain, due to the natural slope of the grout cap and the inclination of some of the holes, varied from 8 to 20 m but an average was close to 20 m.

Grouting started at the bottom of the valley and proceeded up both abutments with the downstream curtain drilled and grouted 15 m ahead of the upstream curtain. In this way the upstream curtain formed a check on the closure of the downstream curtain.

On the basis of known orientations of joints, bedding planes and shear zones it was initially decided to make all grout holes vertical, allowing a maximum number of fissures to be intersected.

Because of the steepness of the left abutment it was later decided to angle the grout holes into the hillside between Blocks F and A, drilling them at an angle of 30° to vertical and on a bearing at right angles to the dam axis. This geometry allowed use of normal grouting pressures without risking uplift of the adjacent hillside. However the effective depth of this angled part of the grout curtain was only 8 m and caused concern as seams of completely weathered dolomitic siltstone persist to depth beneath Blocks A and B. An upstream row of vertical grout holes was therefore drilled to an effective depth of 15 m on Blocks A and B. Low grout takes between Blocks F and C indicated that here a third row of vertical holes was not warranted and that the two rows of angled grout holes would suffice.

From a choice of suspension, liquid or aerated emulsion (foam) grouts it was decided to use an unstable suspended cement grout. The choice was based on experience from previous dam sites and on the assumption that penetration of fissures and not the filling of large cavities would form the major grouting objective (Cambefort, 1977).

Pressures adopted for grouting were calculated on maximum pressures allowable before uplift of the overburden and grout cap could occur. In fact uplift from 2 to 4 mm did occur half way up the right bank grout curtain and grouting pressures had to be lowered from a maximum of 500 KPa to 350 KPa and the number of stages decreased from three to two. Also taken into consideration were minimum grouting pressures allowable, calculated on the hydrostatic pressure exerted by the head of water at full reservoir; this was also used to calculate grout cap thickness.

The presence of completely weathered seams of dolomitic siltstone indicated that grouting in these areas could be achieved by penetration after dilation of the strata (*claquage*, see Cambefort, 1977) rather than by direct penetration. The nature of the weathered seams made impregnation grouting virtually impossible.

Evidence that *claquage* grouting took place was when several boreholes showed backflow of grout up the hole upon relaxation of grouting pressures. Subsequent drill coring in this area through the weathered seams showed none to very little grout; inspection of core from angled grout curtain checkholes also showed that grout had penetrated no further than 250 mm from the nearest grout hole.

The effectiveness of dilation and penetration grouting is related to stresses exerted upon the sides of the boreholes and the same pressures in smaller diameter grout holes may have increased grouting effectiveness. At higher pressures care would have been necessary to prevent uplift of overlying strata.

Low grout take was not interpreted as unsuccessful grouting but simply as an indication that the foundation did not require grouting and water loss would be minimal. Laboratory tests revealed the dispersive nature of some of the weathered dolomitic seams, leading to concern that leakage of water through the weathered seams could eventually cause dispersion and allow piping to take place.

Grout on the site always commenced with a 1:5 mix of cement to water, unless prior water testing of the grout hole at grouting pressures showed high losses when a start mix of 1:3 was used. The disadvantage of commencing with a dilute grout, particularly in areas of weathered seams, is the possibility of sealing the hole and greatly reducing the effectiveness of subsequent grouting.

It may be argued that more effective grouting may have been possible by using small diameter grout holes, higher pressures, and thicker initial grouts. However the results of the grouting programme show that closure was achieved on most grout holes and where refusal was not achieved holes were brought closer together (in some cases as close as 750 mm) until closure was effected.

Investigation of Grout Curtain

Geophysical self potential and point resistance measurements were used on the right bank grout curtain beneath Block M to assess the groutable nature of bedrock, in particular at depths where the grout curtain passes through the transitional strata. Trenching at the top of the right abutment exposed completely weathered strata and because of this the grout curtain was also investigated beneath Block S. This was done by using isotopic grout during the normal grouting programme and locating grouted horizons using a geiger probe.

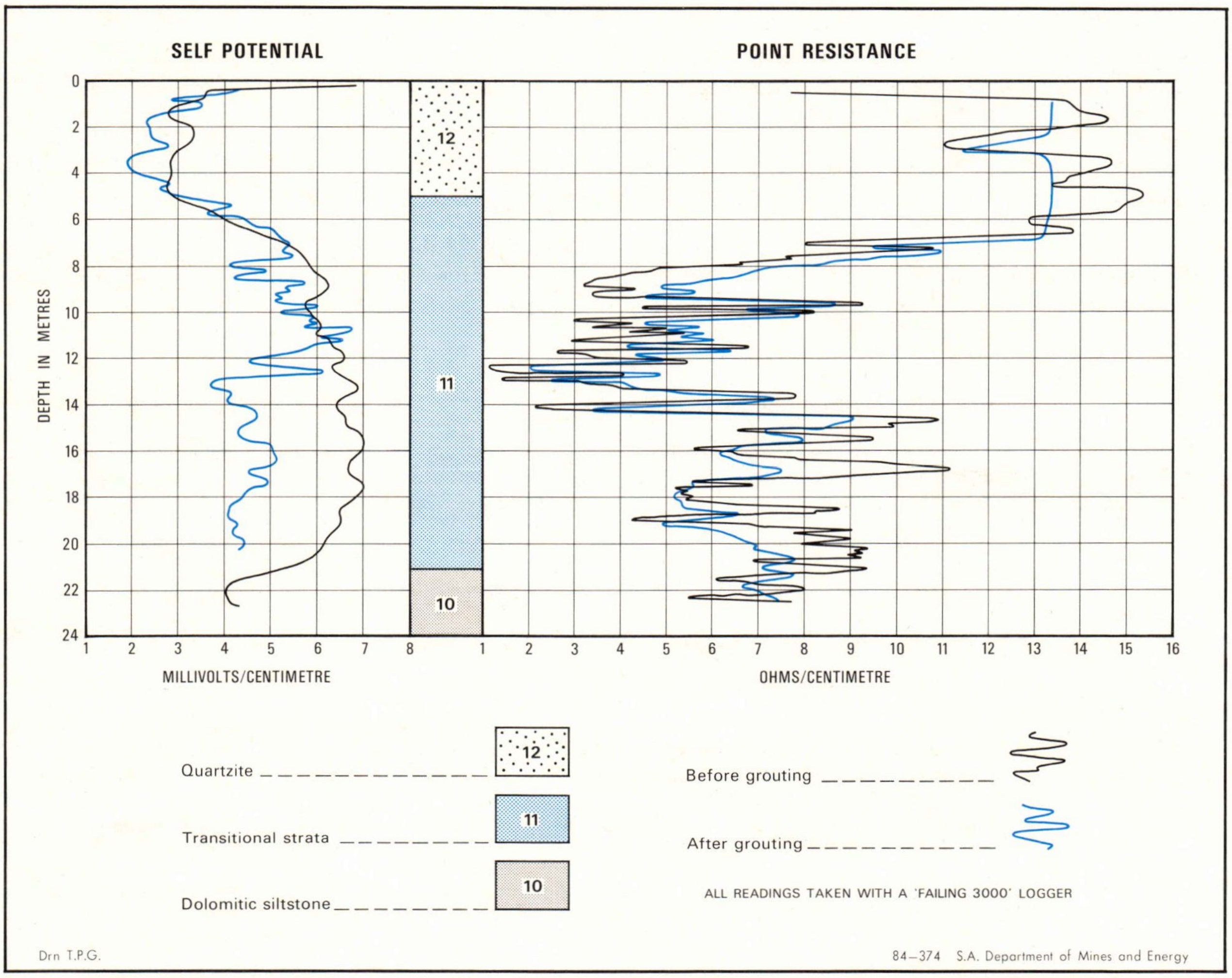


Fig. 18 Isotopic grout test: Drillhole MP40.

Geophysical Methods

Readings taken down grout hole MP 40 on grout cap Block M before and after grouting are shown in Figure 18. Differentiation between the quartzite and the transitional strata was possible using both methods, the point resistance more so than the self potential.

The effect of grouting on the point resistance graphs is to reduce the maxima and minima, particularly the maxima as grouting of the open fractures increases the conductivity of the rock producing lower resistance readings. The difference between the maxima and minima after grouting is more marked between 8 and 14 m within the transitional strata.

Grouting reduced the value of electric current measured during the self potential test, ungrouted areas showing as maxima on the graphs. Although both graphs delineate narrow bands of varying values of self potential or point resistance the significance of each band is questionable.

Construction work made it impossible to repeat the tests on adjacent grout holes but the indication is that these methods are too unreliable for making detailed assessments of grouted strata.

Isotopic Grout Method

Pilot Programme

The initial method of grout curtain testing used water dyed with Rhodamine B to establish the interconnecting nature of open fractures between grout holes and observation holes. After grouting, the dyed water test was then repeated. The presence of any dye in the observation bores was taken as indicating ineffective grouting.

To measure grout travel from grouted hole to observation hole isotopic grout was used, monitored by lowering a geiger probe down the open grout holes and observation holes immediately after grouting. Although isotopes have been used to trace mobility of dam water (Bowen, 1975) there appears to be no record of using isotopes to trace grout movement.

Bromine 82 has a half-life of 37.5 hours and radiation (0.78 Mev) is sufficiently strong to enable geiger measurements to be made up to two days after grouting. This radio-active isotope was supplied by the Australian Atomic Energy Commission, Lucas Heights, Sydney, in liquefied form and forwarded by air freight. The liquid, only a few millilitres, was held in a phial within a screwtop canister; both phial and canister were wrapped in foil and transported within a thick lead container. Only an approved agent for the Commission was allowed to handle the isotope. All men involved with the grouting were supplied with pocket dosimeters. During and after the experiment the equipment and areas affected by the grout were thoroughly washed down and radiation levels checked. All readings were extremely low and at no time constituted a health hazard.

The amount of isotope procured was calculated so as to read 5 millicuries on the morning of collection from the airport. On site it was carefully mixed with water before adding to the grout in 2.5 millicurie amounts.

Trial tests were carried out on block R and divided into two groups: those using Rhodamine B and those using isotopic grout. The tests were carried out to investigate the effectiveness of dye tracers and to calculate required dye concentrations. They were also designed to check concentrations of isotope required, sensitivity of the scintillometer and gamma probe, and the maximum period over which monitoring of the isotope was possible.

Table 9 Summary of grout curtain trial tests

Test Type	Purpose	Results
LABORATORY	To assess powdered granulated activated charcoal, and stick charcoal, as dye tracers in varying strengths of dye solution	(a) Powdered charcoal held in dialysis tubing adsorbs dye too slowly. (b) Low adsorption figures for granulated charcoal held in terylene bags, probably due to wetting problem. (c) One stick charcoal sample gave adsorption figure 10-50 times higher than other charcoals.
	Compare dye adsorption of granulated to stick charcoal.	Stick charcoal very sensitive; granulated type reactive but over a longer period of time.
FIELD	Water pressure test of grout hole RP 75; check water movement to adjacent holes by dye tracers.	Low adsorption values: poor connections between observation holes.
	Water pressure test grout hole RSE 2 as above.	Slightly higher values recorded but still poor connections; obvious staining on canvas masking tape suggested possible use of tape as cheap dye tracer (see Test 7).
	Grout borehole RTE 3 using Bromine 82; measure radiation with scintillometer probe; compare to previous geophysical tests; check site radiation safety levels.	(a) 2.5 millicuries mixed in 200 L of 5:1 grout measureable up to 3 to 5 days without background interference. (b) Pocket dosimeters showed very low exposure levels.
	Assess effectiveness of grouting using dye water pressure test and stick charcoal dye tracer.	Stick charcoal has adsorption limits; eroded clay seams led to interconnections bypassing grouted hole; test inconclusive.
LABORATORY	Assess sensitivity of canvas tape to varying concentrations of dye solution.	Discolouration of tape very sensitive: visible to 3.5 mg/L dye strength; acceptable for field use.
FIELD	Measure background gamma radiation on scintillometer probe.	More convenient than using Geiger equipment but slower and less accurate; satisfactory for the tests; background values up to 3 counts/second.

Table 10 Trial water pressure and isotopic grout tests on Block R

(a) Water Pressure Test: Losses

Borehole	Water Pressure (KPa)	Time (min)	Loss (L)	Total Loss (L)
RP 75	110	5	15	
	110	5	10	25
RSE 2	110	10	28	
	(no pressure recorded)	2	162	188

(b) Grout Test: Losses

Borehole (1)	Stage Depth (m)	Water: Cement ratio	Pressure (KPa)	Time (min)	Grout Loss (L)	Net Loss (Bags dry Cement)
RP 75	9.0	5:1	110	15	120 (2)	
		5:1	110	15	46	
		5:1	110	15	17	
		5:1	110	15	15	
		5:1	110	15	0	
				TOTAL:	78-11 (3) = 67 litres	0.5
RSE 2	9.0	5:1	110	15	28	
				15	18	
				15	2	
				TOTAL:	42-11 = 31 litres	0.2(4)

(1) Half isotope used for testing each borehole; total of 5 millicuries of Bromine 82 used.

(2) Grout loss in delivery hose.

(3) Volume of grout (5:1 mix) in first stage length of grout hole is 11 L.

(4) Calculated from $\frac{31 \times 0.18}{26.5}$

Where 0.18 reduces grout to dry weight for a 5:1 mix and 26.5 converts dry *weight* cement into *bags* of *cement.*

Dye Indicator

Because of its proven sensitivity in tracing mobile water (Atkinson *et al.*, 1973; Lockwood, 1971; Smart *et al.*, 1976) interconnections between grout holes during water pressure testing was established using the dye. Its presence was indicated on dye-sensitive canvas masking tape lowered down the grout hole (Plate 14).

Results of trial tests are summarised in Tables 9 and 10. Generally six grams of dye placed in the standpipe of the grout hole and tested over five minutes gave a measurable dye concentration where water losses are in the order of 100 L/min. Approximate Rhodamine B concentrations in the water after testing for five minutes were in the order of 200 mg/L.

Very dilute but still visual concentrations in the order of 3.5 mg/L were sufficient to permanently stain the canvas tape which was suspended down all test holes. The self-adhesive tape was cheap and easy to use and sufficiently sensitive for the purpose of establishing the presence of dye-bearing water.

It was found that charcoal was sensitive to the dye, and was also easy to handle. However the charcoal needed to be packaged before attaching to the nylon line and an analysis was required before results of the test could be known. The limiting disadvantage is that no matter what the strength of the dye solution, the stick charcoal appeared to constantly give values ranging from 20 to 80 mg/L. Analysis results therefore do not indicate the varying strengths of dye solution, only the presence of Rhodamine B.

In summary, plain canvas tape is sufficiently sensitive to show the presence of Rhodamine B. If the dye is adsorbed by the clay content of any weathered seam then the tape is clean but has a wetted appearance and can still be used to prove the presence of a water connection.

Isotopic Grout

In general 2.5 millicuries of Bromine 82 placed in 200 L of 5:1 grout is sufficient to allow gamma values in the order of 500 counts per second (cps) to be recorded immediately after grouting and 120 cps to be recorded forty-eight hours after grouting. These counts were recorded using a scintillometer probe.

Because of its length the probe became stuck in most of the grout holes. A Geiger probe was tried and proved sufficiently accurate to detect areas of grout take. The equipment used a 6 volt battery and could be kept onsite ready for use as required. Also the probe was small enough not to stick in any of the holes tested.

The geiger probe gave background values in the order of from 2 to 3 cps compared to background values of up to 125 cps recorded by the scintillometer probe. These high counts occurred at depths coincident with a change in rock type from quartzite to dolomitic siltstone, commonly the site of clay development. It was therefore necessary when using the scintillometer probe to record presence of isotopic grout within two or three days of grouting otherwise clay seam radiation could mask the grouted seams.

Results of gamma probing compared to earlier tests using self potential and single point resistivity show the former to be more accurate in identifying the before and after grouting picture; grout connections between grout holes may also be more readily identified (Fig. 19).

Trial Test

The isotope and grout mix was pumped into borehole RP 75 after background readings had been taken down the hole. The location of all grout and observation holes are shown in Figure 20.

Readings were taken in RTE 3 during and after the grouting of RP 75 to establish positions of grouting connections. Using the remaining isotope, borehole RSE 2 was then grouted and readings taken immediately afterwards and daily for the next two days. As

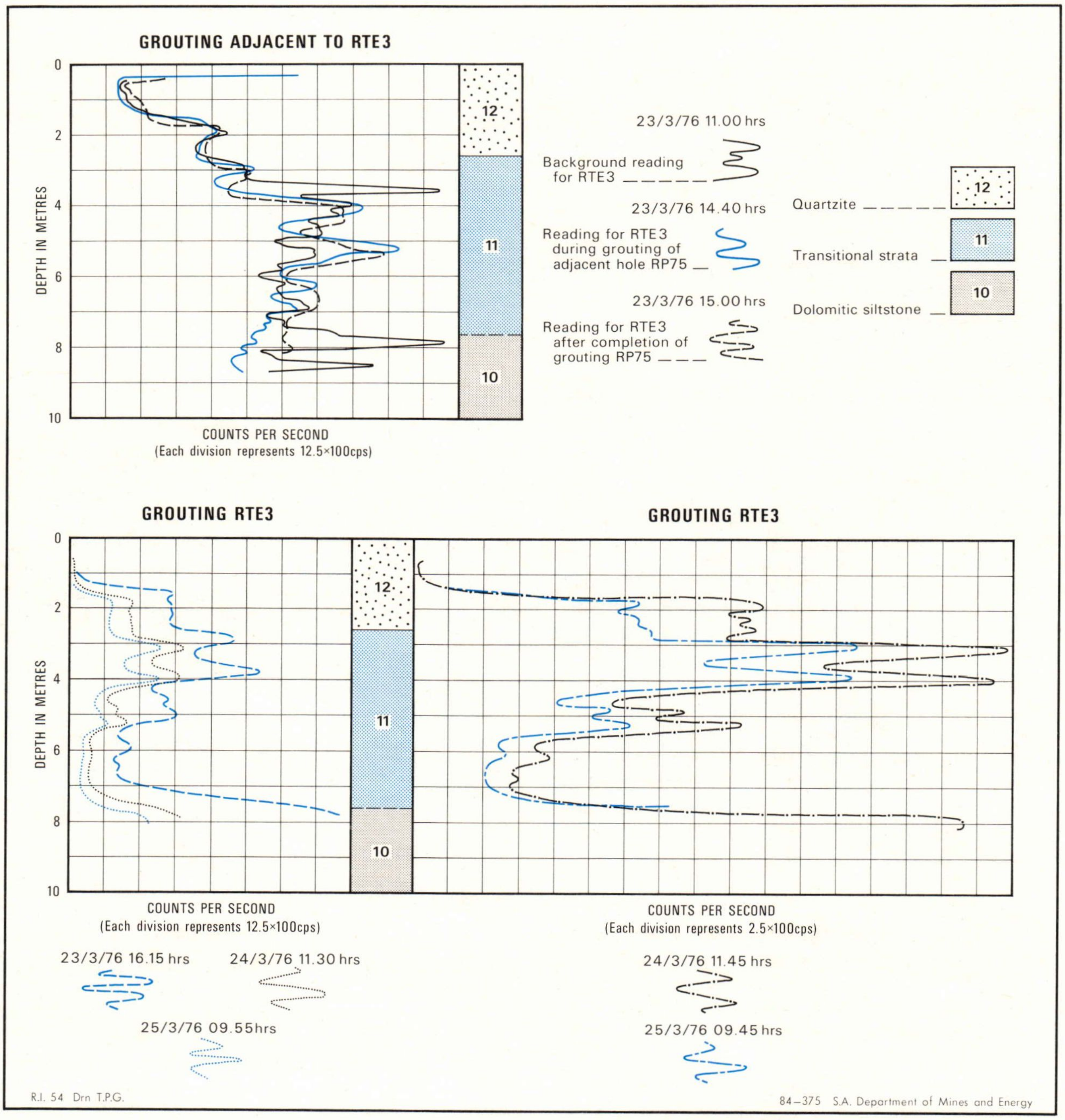


Fig. 19 Isotopic grout test: Scintillometer recordings.

activity of the isotope weakened, readings were taken on an increasingly sensitive scale.

Inspection of the graphs (Fig. 19) allowed the following deductions to be made:

— Background readings in RTE 3 show a marked difference between the base of the quartzite (Unit 12) and the top of the dolomitic siltstone (Unit 10); this difference most probably indicates the presence of clay derived from weathering of the dolomitic siltstone.

— The effect of grouting the adjacent borehole appears to reduce the intensity of the major peaks but increases the intensity of the in-between minor peaks. This increase may have been caused by grouting up of small cavities, and the decrease by flushing out of the thin reactive clay seams during the water testing process. Alternatively, grout deposited on the sides of the borehole may have masked the natural background of radiation initially recorded as being associated with the clay seams.

— Comparison between background values of RTE 3 and grouted values show a general reversal in the peak distribution. Higher peaks occur in the quartzite which has accepted more grout than the dolomitic siltstone which shows smaller peaks. This indicates the poor groutability of the weathered dolomitic siltstone. The count per second increases markedly where grout has collected at the bottom of the borehole.

Readings taken two weeks after grouting were approximately one hundredth of the value of the early readings. For sensitive measurements it is preferable for recordings to be carried out within the first three or four days after grouting.

Block S

Observation bores OB 1, OB 8 and AG 2 were tested for background gamma emission values using a geiger probe in conjunction with a 'Nuclear Equipment Portable Scaler'. Readings for the profile were taken every 250 mm.

All readings fell within the range of from 0.7 to 3.7 cps and represent normal background values for the area; readings greater than 3 cps may indicate clay seams.

Readings from the Bromine 82 isotope should be 100 to 200 times greater than background even two or three days after grouting. For this reason the background values are considered too low to affect borehole gamma profiles during logging of the isotope grout, and only the three observation bores mentioned above were logged for background profiles.

Main Test

The main test was carried out in three steps. Firstly, water pressure testing with dyed water for the first stage only in boreholes SP 116, SP 95 and the central

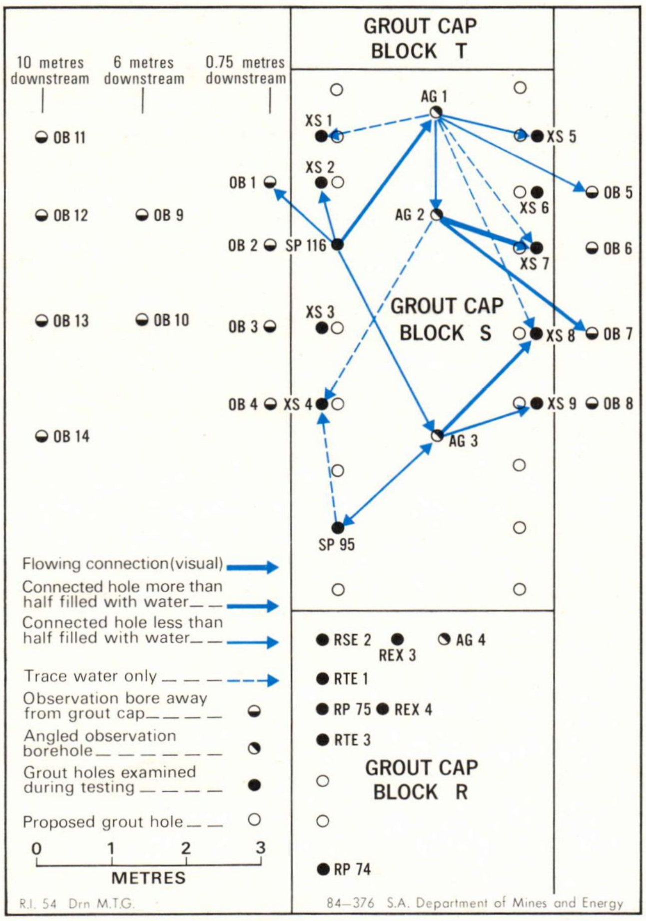


Fig. 20 Grout curtain: Water pressure connections.

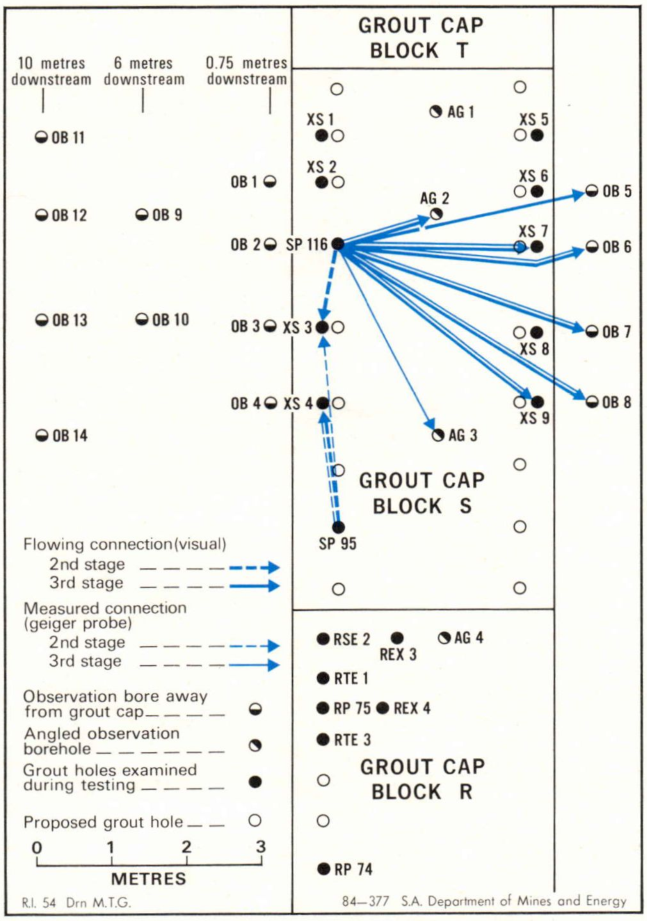


Fig. 21 Grout curtain: Isotopic grout connection.

angled boreholes AG 1, AG 2 and AG 3 (Fig. 20). Secondly, grouting the first stages of SP 116 and SP 95 using isotopic grout and noting connections between the newly grouted holes and any of the observation bores (Fig. 21). The final test checks the effectiveness of the grout curtain beneath grout cap Blocks R and S.

Summary of test results:

— During the grouting of SP 116, grout loss increased from 0.1 bags dry cement in the first stage to 16.4 bags in the third (Table 11). Subsequent geiger probe measurements showed that these losses occurred at from 2 to 3 m depth at the base of the massive quartzite (Fig. 22).

— Grouting had blocked up some of the observation bores and they therefore could not be measured with the geiger probe over their full depth. However, geiger probe measurements in XS 7 and AG 2 (Fig. 23) indicated interconnection with SP 116 between 3 and 4 m with possible connections also at 5.5 m and a connection between SP 116 and AG 3 at 8.5 m.

— This connection at 8.5 m was also shown during the first stage water pressure testing of SP 116 (Fig. 24). It is expected therefore that the flowing connections observed between SP 116 and the observation bores during all three grouting stages probably took place within the transitional strata at the same depths as the measured interconnections.

— There was little grout loss during grouting of SP 95 (Table 11) but a connection to XS 4 during the second grout stage occurred at 1.5 and 3.0 m (Fig. 21) within and at the base of the massive quartzite.

— Water pressure testing of SP 116 at 110 KPa showed reduced water losses from 150 L in 5 min (1st stage) to 42 L in 5 min (3rd stage). This is the opposite to the grout losses, which showed an increase in grout losses when the third stage of SP 116 was grouted three days later.

— Water pressure testing of SP 95 remained very similar at 195 L in 5 min (1st stage) to 100 L in 3 min (3rd stage). This is opposite to the grout losses which showed a marked decrease when the third stage of SP 95 was grouted three days later.

— The discrepancy between water loss values and grout loss values in SP 116 and SP 95 may be accounted for by the differences in the intervals of their second and third stages. The second stage of SP 95 was from 7 to 10 m and was from 7 to 14 m for SP 116. This suggests that the interval 10 to 14 m (base of the transitional strata) is far more permeable than the strata between 14 and 21 m and that the high grout loss during third stage grouting of a 3:1 grout mix in SP 116 did not move into the strata between 14 and 21 m but into first stage levels. This was indicated during earlier grouting by the uplift of grout cap Blocks M and N.

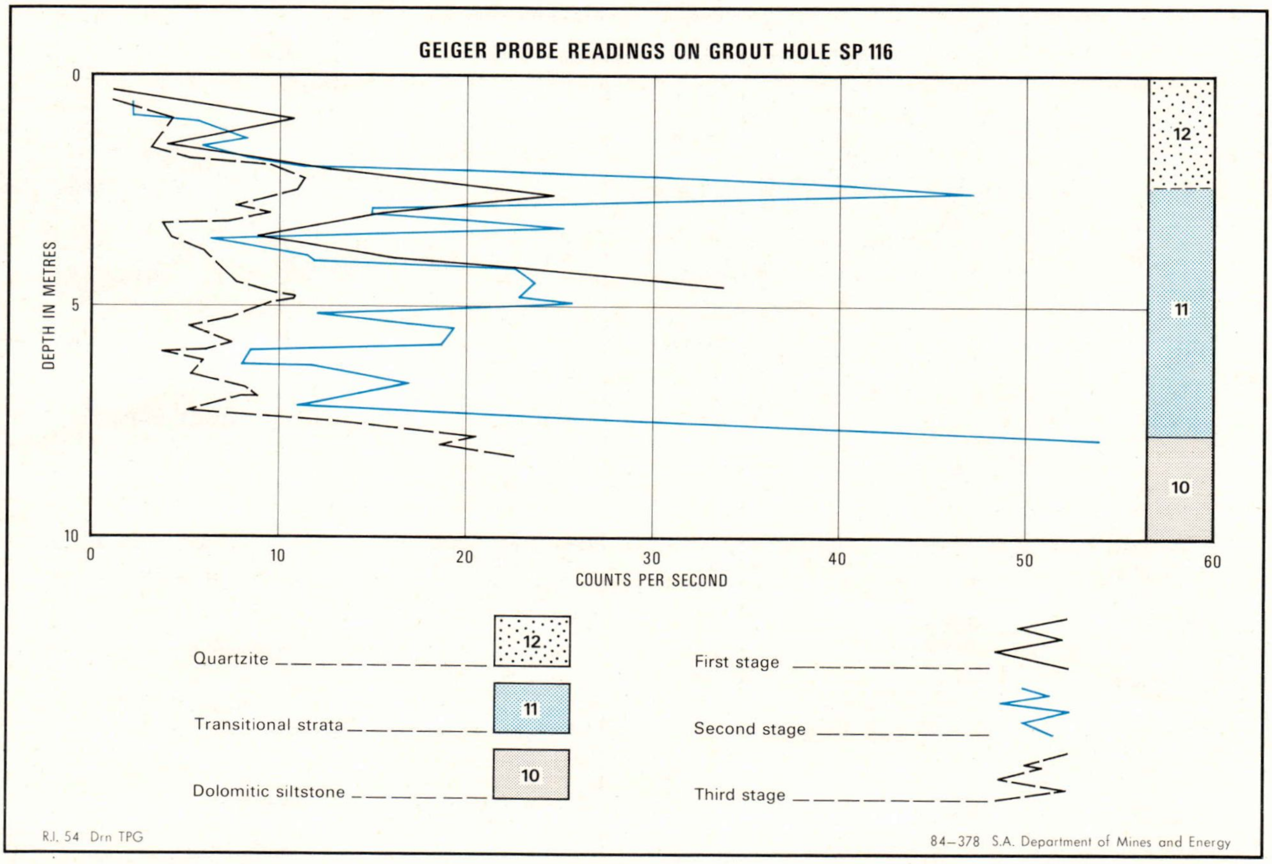


Fig. 22 Grout curtain: Drillhole SP116—Geiger probe readings.

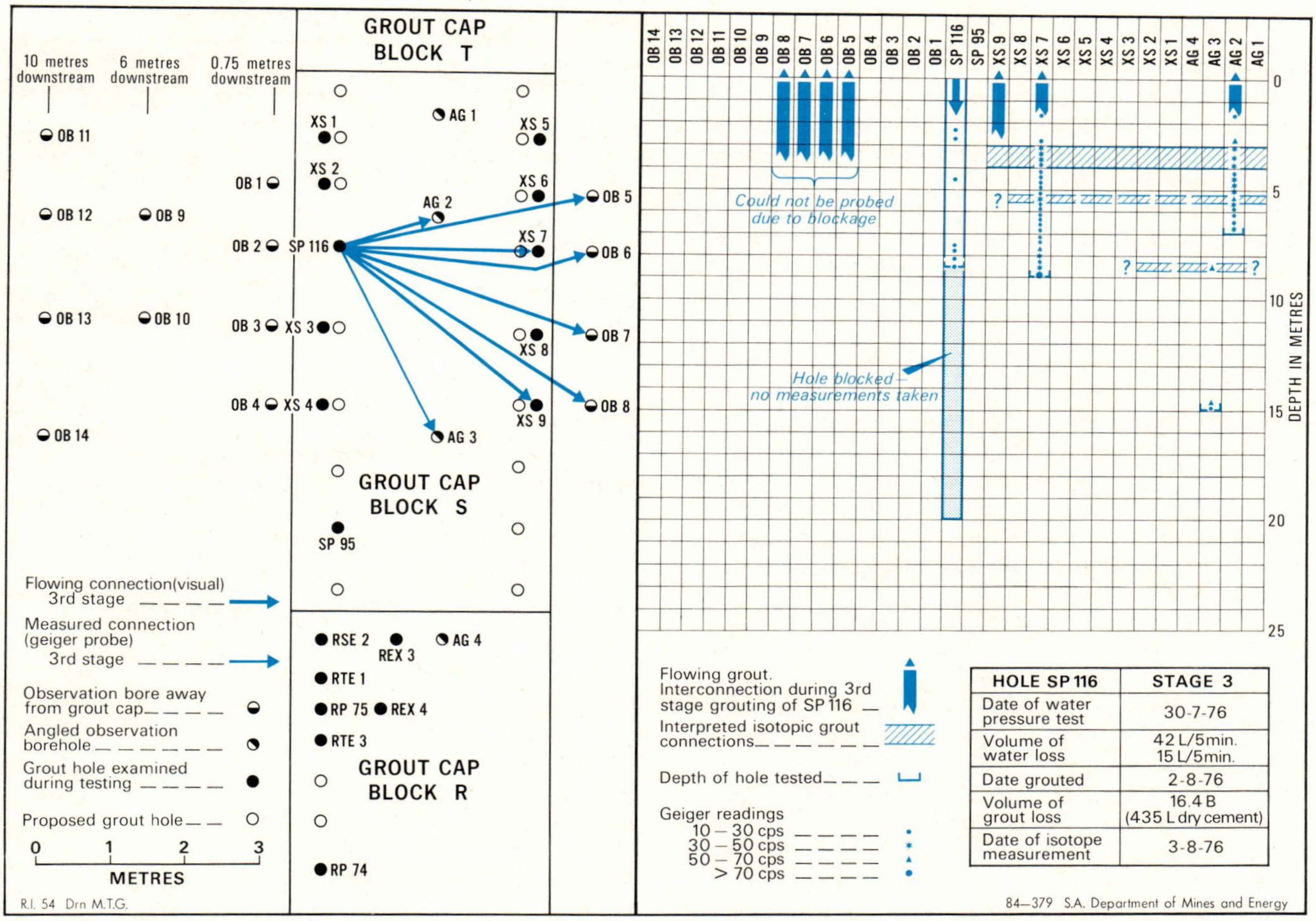


Fig. 23 Grout curtain: Drillhole SP116—Third stage isotopic grout test.

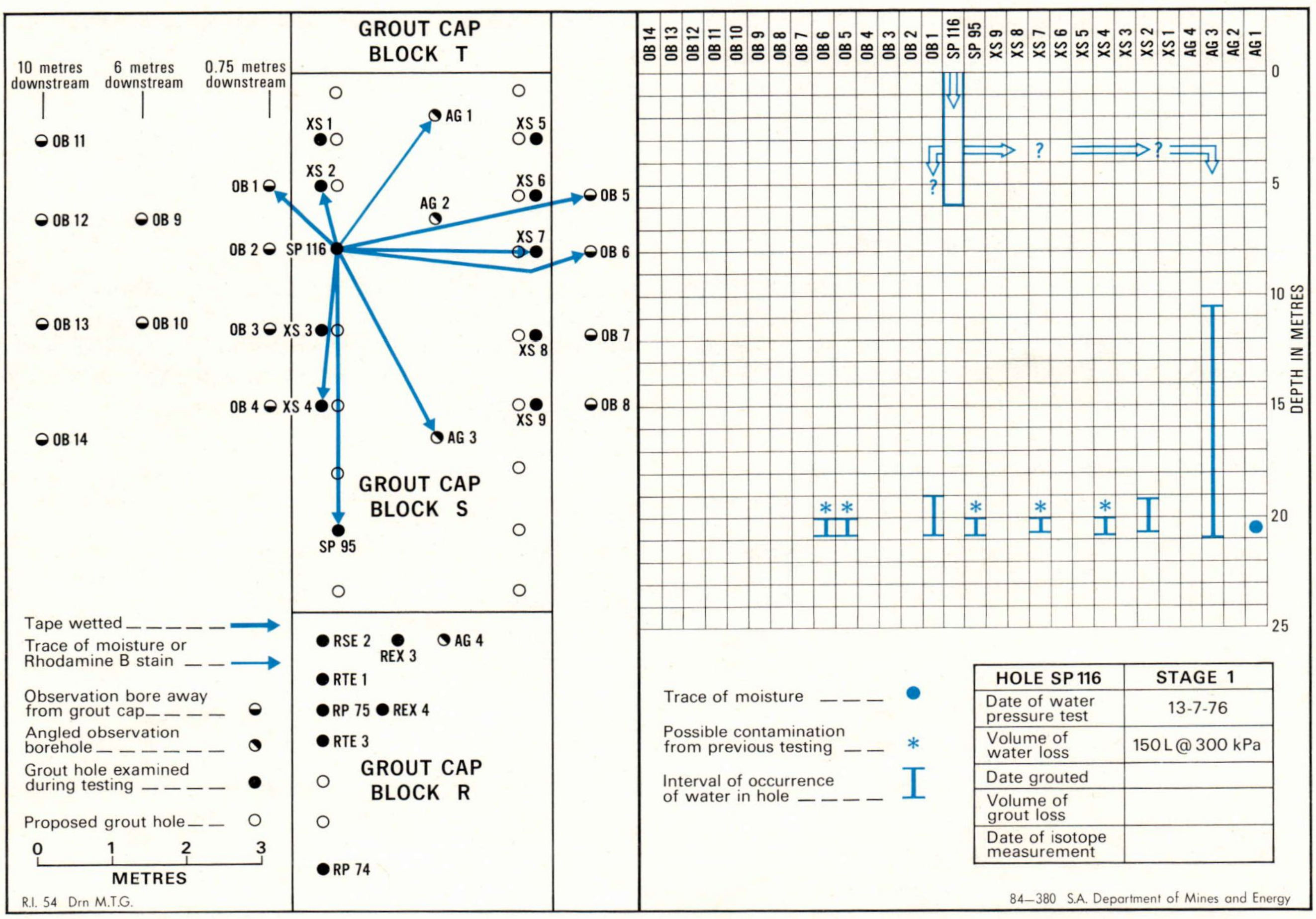


Fig. 24 Grout curtain: Drillhole SP116—Water tests.

Grout Cap Uplift

Uplift occurred during the grouting of Blocks M and N when 366 bags of cement were used in the grouting hole MT 97 (third stage) and 365 bags in grouting hole NQ 114 (third stage) as shown in Figures 25 and 26. It is believed that the high third stage pressures caused dilation of the transitional strata bedding planes, in turn causing uplift of the overburden and allowing the grout to travel away from the hole being grouted.

Grout Travel

Evidence of grout travel and dilation is that open grout holes drilled to first or second stage in advance of the grouting programme (up to Block P and lower part of Block Q) showed flowing grout during the grouting of NQ 114. Cessation of grouting and removal of the grout hose from the standpipe allowed grout to flow out of NQ 114.

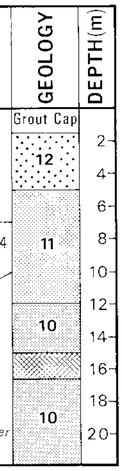
The preferred movement of grout in the transitional strata (Unit 11) has been further proved by the isotopic grout programme. This dilation of bedding planes is the direct consequence of downstage grouting from a standpipe when the full length of the grout hole is subjected to third stage grouting pressures and overburden uplift. The plots of water and grout interconnections show a strong preference for upstream movement. The downstream observation holes did not record either the presence of grout or water except in one case where water movement was detected from SP 116 to OB 1. This is difficult to explain particularly as the strike of the strata is approximately across the alignment of the grout cap. These results may indicate a preferred upstream permeability of the strata or preferred direction for dilation of the bedding planes.

Table 11 Main test on grout cap Block S.

WATER PRESSURE TEST		STAGE	GROUT TEST		GROUT CONNECTIONS
DATE	LOSS		LOSS	DATE	
20-4-76 13-7-76	195L in 5 min at 110kPa pressure 104L in 5 min at 450kPa pressure	1	0.5 bags (0.013m³) at 110kPa; water–cement ratio 5:1	14-7-76	
26-7-76	100L in 3 min at 250kPa pressure	2	5.8 bags (0.15m³) at 250kPa; water–cement ratio 5:1 thickening to 2:1	27-7-76	To XS4
30-7-76*	100L in 3min at 110kPa pressure	3	0.1 bags(0.003m³) at 250kPa; water–cement ratio 3:1	2-8-76	

Owing to water loss at 10m, third stage water pressure test (10–21m) continued after grouting second stage (7–10m).
3rd stage grouting did not use isotopic grout. All isotope used for 3rd stage grouting SP116
**Note high water loss and low grout take. Clay seam reported by driller from 15.2–16.8m.*

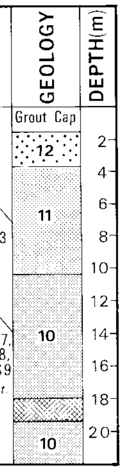
GROUT HOLE NO. SP95

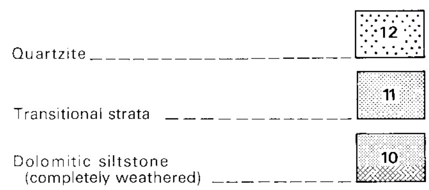


WATER PRESSURE TEST		STAGE	GROUT TEST *		GROUT CONNECTIONS
DATE	LOSS		LOSS	DATE	
13-7-76	150L in 5 min at 110kPa pressure	1	0.1 bags (0.003m³) at 110kPa; water–cement ratio 5:1	14-7-76	
26-7-76	110L in 5 min at 110kPa pressure 10L in 5 min at 250kPa pressure	2	1.0 bags (0.027m³) at 250kPa; water–cement ratio 5:1	27-7-76	To XS3
30-7-76	42L in 5 min at 110kPa pressure 15L in 5 min at 250kPa pressure	3	16.4 bags (0.43m³) at 250kPa; water–cement ratio 3:1 mix thickening to 1:1	2-8-76	To XS7, OB6,7,8, AG2,XS9

** Four applications of isotope used i.e. 2.5 millicuries per 108L dry cement.*
Driller reports water loss between 18.3m–18.9m.

GROUT HOLE NO. SP116





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Water Pressure Tests on Grout Curtain

To check the effectiveness of the completed grout curtain three cored drillholes located in the centre of the curtain were water pressure tested up to a maximum pressure of 500 KPa. Location and logs of these holes are given in Appendix 1.

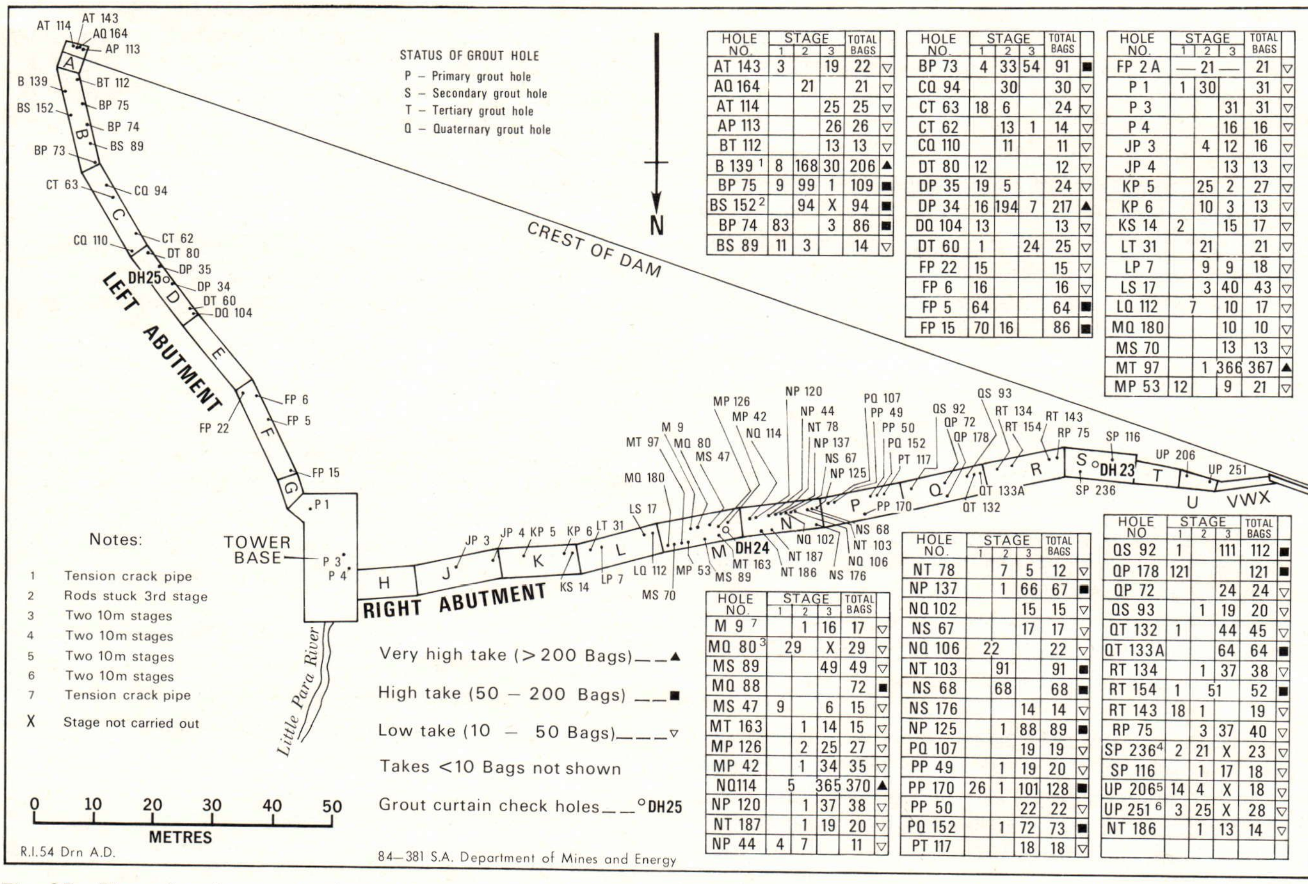
Testing was initially carried out at 2 m intervals using a double packer. Faulty operation of the lower part of the packer while moving it from one interval and resetting on a higher interval led to apparent very high water losses (up to 70 lugeons*).

Table 12 Grout curtain check hole grout takes

Hole	Date	Stage Depth (m)	Volume of hole (litres)	Grout mix (water/cement)	Grout Take: Total plus hole volume	Grout Take: Approx. net (dry bags cement)	Injection Pressure KPa	Method
DH 25 (Block D)	20.7.77	0.0 to 20.5 m	103	1:1	108	None	NIL	25 mm tremie tube
DH 23 (Block S)	12.8.77	0.0 to 21.0 m	105	2:1	105	None	150	Nipple
DH 24 (Block M)	12.8.77	10.2 to 21.0 m	43	1:1	0(1)	None	150	Inflatable Packer
	12.8.77	10.2 to 21.0 m	43	1:1	41	None	NIL	25 mm Tube
	19.8.77	8.4 to 10.2 m	9	2:1	15	0.09	250	Inflatable Packer
	19.8.77	0.0 to 8.4 m (Effectively 1.5 to 8.4 m)	34	2:1	32	None	150	Inflatable Packer

(1) Double Packer stuck at 15m

*defined in Appendix 5.



HOLE NO.	STAGE 1	STAGE 2	STAGE 3	TOTAL BAGS	
AT 143	3		19	22	▽
AQ 164		21		21	▽
AT 114			25	25	▽
AP 113			26	26	▽
BT 112			13	13	▽
B 139 [1]	8	168	30	206	▲
BP 75	9	99	1	109	■
BS 152 [2]		94	X	94	■
BP 74	83		3	86	■
BS 89	11	3		14	▽

HOLE NO.	STAGE 1	STAGE 2	STAGE 3	TOTAL BAGS	
BP 73	4	33	54	91	■
CQ 94		30		30	▽
CT 63	18	6		24	▽
CT 62		13	1	14	▽
CQ 110		11		11	▽
DT 80	12			12	▽
DP 35	19	5		24	▽
DP 34	16	194	7	217	▲
DQ 104	13			13	▽
DT 60	1		24	25	▽
FP 22	15			15	▽
FP 6	16			16	▽
FP 5	64			64	■
FP 15	70	16		86	■

HOLE NO.	STAGE 1	STAGE 2	STAGE 3	TOTAL BAGS	
FP 2A	—	21	—	21	▽
P 1	1	30		31	▽
P 3			31	31	▽
P 4			16	16	▽
JP 3		4	12	16	▽
JP 4			13	13	▽
KP 5		25	2	27	▽
KP 6		10	3	13	▽
KS 14	2		15	17	▽
LT 31		21		21	▽
LP 7		9	9	18	▽
LS 17		3	40	43	▽
LQ 112	7		10	17	▽
MQ 180			10	10	▽
MS 70			13	13	▽
MT 97		1	366	367	▲
MP 53	12		9	21	▽

HOLE NO.	STAGE 1	STAGE 2	STAGE 3	TOTAL BAGS	
M 9 [7]		1	16	17	▽
MQ 80 [3]	29		X	29	▽
MS 89			49	49	▽
MQ 88				72	■
MS 47	9		6	15	▽
MT 163		1	14	15	▽
MP 126		2	25	27	▽
MP 42		1	34	35	▽
NQ114	5		365	370	▲
NP 120		1	37	38	▽
NT 187		1	19	20	▽
NP 44	4	7		11	▽

HOLE NO.	STAGE 1	STAGE 2	STAGE 3	TOTAL BAGS	
NT 78		7	5	12	▽
NP 137		1	66	67	■
NQ 102			15	15	▽
NS 67			17	17	▽
NQ 106	22			22	▽
NT 103		91		91	■
NS 68		68		68	■
NS 176			14	14	▽
NP 125		1	88	89	■
PQ 107			19	19	▽
PP 49		1	19	20	▽
PP 170	26	1	101	128	■
PP 50			22	22	▽
PQ 152		1	72	73	■
PT 117			18	18	▽

HOLE NO.	STAGE 1	STAGE 2	STAGE 3	TOTAL BAGS	
QS 92	1		111	112	■
QP 178	121			121	■
QP 72			24	24	▽
QS 93		1	19	20	▽
QT 132	1		44	45	▽
QT 133A			64	64	■
RT 134		1	37	38	▽
RT 154	1	51		52	■
RT 143	18	1		19	▽
RP 75		3	37	40	▽
SP 236 [4]	2	21	X	23	▽
SP 116		1	17	18	▽
UP 206 [5]	14	4	X	18	▽
UP 251 [6]	3	25	X	28	▽
NT 186		1	13	14	▽

Fig. 25 Plan of major grout takes.

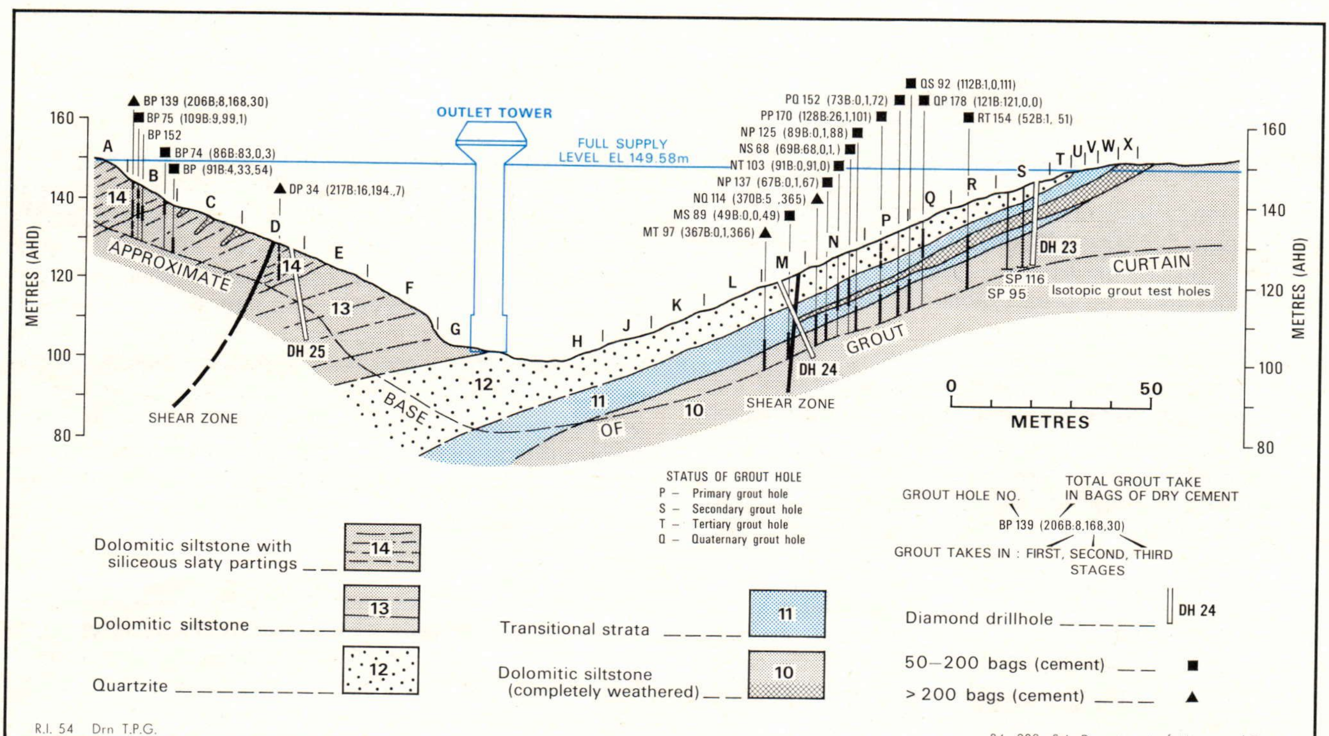


Fig. 26 Geological section showing grout takes.

Interpretation of the data assuming the test was over the complete section of the hole below the upper section gave values of from 2 to 7 lugeons. This was later verified by testing with a single packer.

Although it is possible to distinguish between a faulty packer and a highly permeable area (Pearson *et al.*, 1977) it was considered that sufficient knowledge of the likely permeability of the bedrock had been gained during the numerous water pressure tests on each grout hole during the grouting programme to allow the recognition of spuriously high results.

A single packer was used for the remaining tests by successively grouting lower parts of the hole and moving the single packer up the hole to test the next interval (Appendix 1). Even with a single packer, increased permeabilities were recorded with application of higher pressures. A hysteresis loop recorded at decreasing water pressure during the second stage of many of the tests also reflected the increase in permeability and was interpreted as dilation of the bedding planes rather than a leaking packer (Houlsby, 1977; Pearson *et al.*, 1977).

Check hole DH 24 (Appendix 1) was located and angled to intersect the most likely area of potential leakage, at approximately 10 m beneath grout cap Block M where a shear zone intersects the weathered transitional strata immediately beneath the massive quartzite (Unit 12). Losses of from 11 to 20 lugeons were measured over the interval 9.5-10.2 m. This is the same interval where the earlier self potential and point resistance logs indicated a change in rock groutability. It is considered that the presence of the shear zone within the fluctuating groundwater table, and the resultant localised weathering of the transitional strata, can account for the relatively high water losses over this interval.

Although low grout takes were recorded when test hole DH 24 was grouted (Table 12) this is considered more indicative of the ungroutable nature of the weathered strata than of a watertight grout curtain.

Low lugeon values were recorded in the other two check holes DH 23 and DH 25 (from 3 to 5 lugeons and 0.5 lugeons respectively) and both recorded low grout takes when grouted (Table 12).

Results from water pressure testing of the check holes therefore show that lugeon losses of less than seven are indicated over most of the dam site, except in the area where losses of up to 20 lugeons have been recorded over a short interval. The lower figure of 7 lugeons is regarded as satisfactory for this type of dam (Houlsby, 1977 and Appendix 5).

Discussion and Summary

Field research work run concurrently with an engineering project is necessarily compromised: the drilling equipment is requested elsewhere; the rockfill is to be placed earlier than calculated; field assistance is not always forthcoming; and many other such problems beset the field research project.

Because the dam was mainly built by State Public Service day labour it was easier to occasionally modify the construction programme than it would have been had it been built entirely by contract work. This allowed the drilling, grouting, and water pressure testing of the extra grout curtain holes required for the research programme.

Although it would have been easier to tackle the problem of establishing an investigation technique under laboratory conditions, it is believed that field-tests under real conditions are more valuable and from this point of view it is considered that efforts to develop a grout curtain testing technique were worthwhile.

Use of isotopic grout has shown that depth of grout takes and their location and thickness can be established, that the groutability of rock types may be assessed, and that correlation of grout takes from hole to hole may in some cases be observed.

The most favoured direction of grout movement during grouting can also be established. The self potential and point resistance logging methods are too inaccurate to use for locating the exact position of groutable bedrock.

Although seams of completely weathered dolomitic siltstone occur at depths in nearly all the grout holes, high grout losses (other than where cavities within the weathered seams are believed to have been present) do not appear to be associated with these weathered seams but with the transitional strata (Unit 11) found at the base of the massive quartzite (Unit 12). The weathered dolomitic siltstone is not groutable and water and grout losses are considered to have occurred along the open bedding planes and joints of the transitional strata. Grout penetration at shallow depths and at high pressures along these fractures resulted in small amounts of uplift of grout cap Blocks M and N.

Study of the isotopic grout record shows that successful grouting of flaggy quartzite (Unit 12) has been achieved and that this may still have been possible even if a wider grout hole spacing had been used. Grouting of the weathered seams of dolomitic siltstone is considered impossible owing to its impermeable nature. However during the grouting programme it was noticed that water losses during cleaning of the hole prior to grouting were large compared to the low but not negligible grout takes. The process of water testing may well have washed out the dispersive clay content of the weathered seams. This disparity indicates that water losses cannot always be used to predict grout losses.

Attempts to use Rhodamine B dye to indicate interconnections between grout holes during water pressure tests failed because the dye was adsorbed adjacent to the grout hole by the clay present in the seams of completely weathered dolomitic siltstone. The wetting effect on canvas tape proved sufficiently sensitive to establish interconnections.

Testing of three diamond drillholes within the grout curtain gave lugeon values ranging from one in slightly weathered to fresh dolomitic siltstone (Unit 10) to seven in the transitional strata (Unit 11). Within completely weathered dolomitic siltstone, lugeon losses of from 11 to 20 were recorded over a 0.7 m interval. Values of from 2 to 3 lugeons were recorded in the massive quartzite. For this type of rockfill dam lugeon values less than seven within the grout curtain are considered acceptable in terms of watertightness.

The clay in the weathered seams is dispersive and thus potentially erodible, but because the leakage paths are likely to be long (from 20 to 60 m), and the permeability of the completely weathered dolomitic siltstone moderate (1 x 10^{-4} cm/sec), erosion of the grout curtain as a whole by leaking water is unlikely.

In retrospect, a more comprehensive trial test could have been carried out on the grout curtain, to work out practical problems and to give more time for planning the main test through all of the three grout stages. On the other hand each research test disrupts the tight schedule of the construction programme, and some compromise is required.

PART V: SURVEILLANCE PROGRAMME AND PROJECT EVALUATION

Surveillance Programme

To monitor the post-construction performance of the dam a surveillance programme (Table 13) began in December, 1977. This was designed to measure daily the rate of leakage through the dam and the change of groundwater levels in the right abutment during the first filling of the reservoir. After the dam is full the monitoring programme will continue with surveillance of reservoir leakage and groundwater levels, and inspection of the dam structures, at intervals commensurate with the early results and rate of the first reservoir drawdown.

Determining the origin and quantity of leakage or seepage through a rockfill dam is more difficult than for a concrete dam, since both the collecting elements inside the dam as well as the abutments and foundations are porous media into which seepage entering through one point can be quickly distributed; as a result discharge water issuing from the dam may have come from an extensive area (Allende 1976, p. 182).

Downstream discharge at Little Para Dam prior to reservoir filling was due mainly to natural groundwater issuing from existing springs in the foundation of the dam. Post-construction discharge originates from these springs plus the effect of filling the reservoir (leakage of water through the grout curtain and the upstream concrete deck) and also from rainfall. There has been no attempt to measure these three discharge sources separately.

However, discharge from behind the outlet duct on the left abutment is measured separately and any increase in this measurement, not accompanied by a similar increase in the main discharge, can only be interpreted as leakage through the left abutment with possible leakage through the upstream concrete deck adjacent to or near the left abutment.

Table 13 Surveillance check list

Task	Frequency of observations during filling*
Evidence of leakage:	
Downstream face of bank	twice daily
River bed up to 0.5 km downstream of bank	twice daily
Left abutment up to 0.5 km downstream of bank	daily
Right abutment up to 0.5 km downstream of bank	daily
Discharge over the double V-notch weir in the downstream wall	twice daily
Flow of water into duct drainage sump:	
Spillway drainage holes	twice weekly when water is within 5 m of spillway crest
Depth of water passing through the downstream causeway	daily
Depth of water over gauging weir downstream of causeway	daily
Leakage alongside duct from left abutment	twice daily
Leakage collected from seepage from broken rock area in left abutment immediately downstream duct	twice daily
Evidence of slips in or above the reservoir area	fortnightly, after heavy rain, and after any rapid drawdown
Internal face of the walls of the tower:	
Check signs of seepage with special emphasis on concrete surrounding the bellmouths and the dayplane joints	daily
Internal face of duct	monthly
Face slabs on the bank: check especially for spalling along vertical joints or at grout cap/face slab interface	weekly
Spillway concrete; rock below ski-jump	after spillage
Area adjacent to river outlets	after operation
Spillway floor, walls, apron and adjacent cliff	after spillway operates
Instrumentation monitoring	follow existing procedures

*To be revised after filling.

Total downstream discharge had reached a maximum of 14 L/sec by November, 1978 at reservoir level El. 141 m, the highest level reached during 1978. Seepage from the left abutment hillside adjacent to the Outlet Pipe remained low at 6.35 L/sec during reservoir filling. The main discharge, at the V-notch built into the Control House retaining wall, remained fairly constant over the six months prior to filling at approximately 0.4 L/sec in November, 1978. Discharge from behind the left hand side of the outlet duct varied between 0.024 to 0.034 L/sec prior to the filling, and gradually increased to 3.8 L/sec by November, 1978.

Groundwater levels from two observation wells are being continuously monitored. These wells (MOB 1 and MOB 2 in Figure 25) are located adjacent to the downstream toe of the rockfill dam. MOB 2 (El. 146.6 m) remained dry during the early months of reservoir filling but water was recorded at El. 126 m on 8 August 1978 at reservoir El. 134 m. Prior to filling of the reservoir MOB 1 (El. 124 m), gave a static water level 11.45 m below natural ground level (El. 112.55 m). The water level rose to El. 123 m during reservoir filling and remained stable at this level.

The depth and location of these two observation wells is such as to intercept and record changes in groundwater levels resulting from the leakage of reservoir water through the grout curtain into the right abutment. Any increase in rate of rise of water level in the observation well greater than an increase in rise of reservoir will warrant very close inspection even if the well water level may be metres below reservoir level (Penman, 1977).

A third observation well DH 29 drilled adjacent to MOB 1 is not fitted with a continuous water level recorded but is read manually to check the readings in MOB 1.

Results from the surveillance programme show that the change in downstream discharge and groundwater levels reflect saturation of bedrock due to filling of the reservoir and indicates some leakage through the dam. This leakage is to be expected and is not considered great enough to endanger dam safety (see permeability tables in Appendix 5).

Inspection of the two reverse filter trenches excavated into seams of completely weathered dolomitic siltstone downstream of the parapet wall, has shown that the trench has remained dry and that there has been no leakage through the grout curtain at the top of the right abutment or beneath the 2.5 to 3.5 m deep concrete footing of the parapet wall.

Although short seepage paths through the grout curtain may occur through seams of weathered dolomitic siltstone it is very unlikely that a long seepage path from the grout curtain to downstream of the dam is possible.

Using observation well MOB 1, approximate values of 0.1 to 0.62 $m^3/d/m$ have been obtained for the transmissivity of moderately weathered dolomitic siltstone. This is a very low value and, providing that it is typical of the foundation strata of the dam, then physical erosion of weathered strata will not occur. The rockfill dam is designed to cope with a certain amount of leakage and, apart from possible stability problems, loss of water from the reservoir through the embankment and beneath the dam is not critical.

Groundwater contours (Fig. 27) approximate surface contours and do not indicate water flow away from the dam. Springs are evident in a few locations where groundwater levels intersect topographic contours, for example the springs from earth-strap hole ESH 2 and from the quartzite at El. 110 m and El.100 m respectively.

As well as measurement of discharge and groundwater levels, and the routine inspection of the dam and spillway, hillside survey pegs upstream of the dam are being monitored fortnightly or immediately following heavy rain or rapid reservoir drawdown. This survey is to assess the possibility of hillside slip into the reservoir. The area is one of shallow calciomorphic clay soils overlying a weathered dolomitic siltstone bedrock. It covers an area of approximately 25 000 m^2 and has an approximate maximum soil volume of 45 000 m^3. To date there has been no record of hillslope movement except for a slight increase in elevation (from 1 to 3 mm) attributed to ground swelling as the rising reservoir level saturates ground adjacent to the survey pegs.

Monitoring of hillside survey pegs stopped in April, 1983. Maximum movement recorded was a drop in elevation of 30 mm on one peg. This was probably the result of vandalism as nearly all of the survey pegs had been interfered with in some way.

Project Evaluation

Geological Investigation

There are many guidelines for investigating a dam site and reservoir (Burgess, 1975; Dearman, *et al.*, 1974; Hofman, 1973; Knill, 1971; Roberts *et al.*, 1975; Stapledon, 1964, Wahlstrom, 1974) but they can at best be only guidelines as each site is unique with its own geological and construction problems.

The Little Para Dam project was first discussed in 1959. Early investigations were spasmodic, depending on availability of finance and varying projections of water requirements to meet estimated population growth. Not until 1973 when building of the dam was assured could the results of previous investigations be incorporated into the final investigation and the programme for the project planned.

This discontinuous appraisal of the project had the advantage of involving several geologists and engineers and by the time construction commenced, the area was well understood and communication between geologists, design and construction engineers well established.

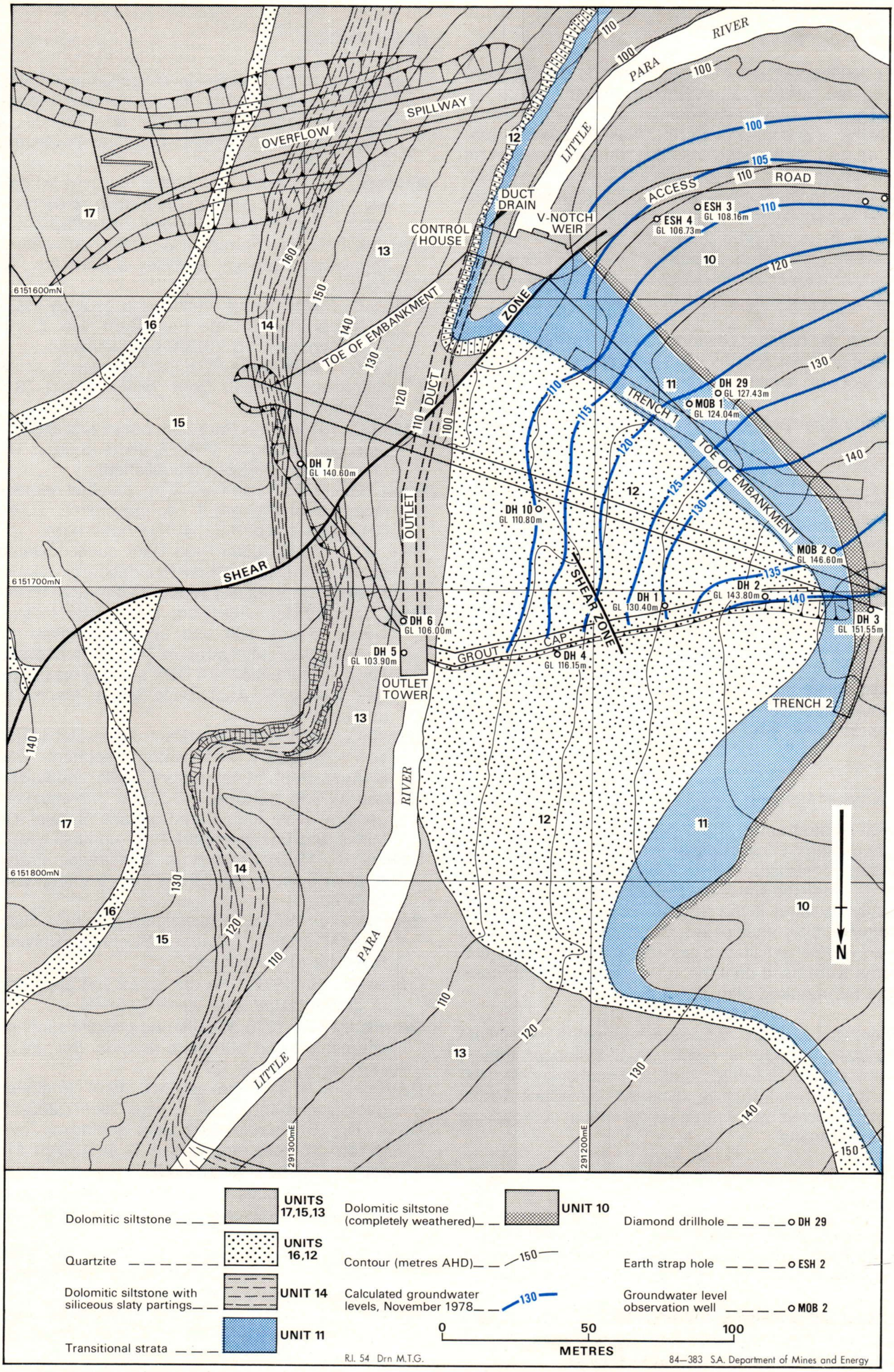


Fig. 27 Surveillance programme: Observation wells and groundwater contours.

Construction Period

On a construction site it is not uncommon for engineers to request a report on a specific problem before the geologist has had time to complete his investigations. A flexible approach to reporting was therefore essential, as was early organisation of the project in terms of communication and co-operation between the geologist and engineer (Stapledon, 1976). Engineers were asked as early as possible in the project what particular service they required from the geologist. Equally importantly, the geologist advised the engineer of any particular aspect of the project he thought warranted early investigation.

As well as reports covering specific investigations, several progress reports were compiled and submitted to a working review committee.

Detailed surveyed plans and colour photographs were used in all reports although these sometimes had to be added to the text at a later date.

Important day-to-day communication between geologist, construction engineer, design engineer, site surveyor and site workmen was verbal with follow up memoranda, letters, or reports when required.

Presentation of a report to a project working committee involved the discussion of geological facts, assumptions made, and the interpretation in respect to the site work currently being investigated. Assumptions made in interpretation of site data were pointed out to the committee, particularly if an early decision was required based on minimum geological information.

Geological Mapping

Aerial photographs at 1:1 000 scale were ideal for early geological investigations. For detailed work during the construction stage, plans at 1:500 and 1:250 scales were the most useful. The site surveyor was greatly assisted if the need for small scale detailed plans could be anticipated. Often an immediate need could not be anticipated and tape and compass diagrams were used until the surveyor had time to complete the required plan.

Where the design engineers requested immediate information from the results of a bulldozer trench, backhoe pit, or drill holes, sufficient mapping or logging was carried out to give them an immediate verbal report followed by a written report with accompanying surveyed plans.

Records

A complete photographic record comprising coloured photographs and transparencies was kept during construction of the dam and its appurtenant structures.

Photograph negatives and copies of all reports have been stored in the records section of the South Australian Department of Mines and Energy.

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APPENDIX 1

Grouting and Water Pressure Testing

by

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SUMMARY

Water pressure testing of drilled grout holes was carried out for 5 min at 110 KPa, followed by 5 min at stage grout pressure.

Downstage non-packer grouting using a maximum pressure of 500 KPa was carried out in three 7 m stages. Average grout take was 2 kg/m (0.5 bags dry cement per stage).

An initial water/cement mix of 5:1 was used unless high water losses indicated use of a stronger mix (1:1 maximum). A start mix of 3:1 was used in areas of potential grout cap uplift.

Refusal of a grout hole was taken as a maximum of 10 L of grout (7 bags or 294 kg at 1:1 mix) in 15 minutes.

Water pressure testing of three grout curtain check holes gave values of from 1 to 3 lugeons; these figures are acceptable.

Water Testing Procedure

Each stage of a hole was tested for 10 min; the first 5 min at 11 KPa and the next 5 min at the relevant grouting stage pressure. The first 5 min test was intended to give some comparison between holes irrespective of location on the grout cap, while the second gave some indication of probable grout takes in the particular stage and also helped to wash out adjacent seams.

Grouting Procedures

Grouting can be defined as the injection of materials (in this case cement slurry) into foundation rock so as to reduce permeability and increase its strength.

The grouting at Little Para Dam was of two types: blanket grouting confined to the tower base region, and curtain grouting along the length of the grout cap.

Blanket Grouting

The tower base slab was post tensioned to the rock foundation by means of high tensile Macalloy bars. To increase the strength of the foundation rock blanket grouting was used to reinforce the rock to a depth of 11.5 m.

Curtain Grouting

The grout cap is a concrete slab designed to seal the joint where the upstream deck of the dam meets the rock. It was not embedded but laid directly onto a foundation cleaned down to tight fresh bedrock or treated with dental concrete where required.

Two rows of holes were drilled through the grout cap, into the rock, for the purpose of injecting a cement slurry under high pressure to form an impermeable grout curtain right across the valley. The curtain grouting aimed to extend the impermeable grout cap a distance of approximately 20 m into the foundation. However on the left abutment the effective depth of the grout curtain was less then this, being 21 m x cos 30° (average slope of the grout cap), that is 18 m. On the left abutment the majority of the holes were angled 30° to vertical and on a bearing of 18° (at right angles to the dam axis) which resulted in a minimum effective depth of approximately 7 m. Tables 14 and 15 show the method of recording data for foundation water testing and foundation grouting respectively. This data was later summarised for each grout hole (Table 16).

In Block B of the left abutment, an additional line of holes was grouted to form a third curtain. These holes were drilled vertically resulting in an effective curtain depth of 16 m.

In addition to using downstage nipple grouting, upstage packer grouting was used in some tertiary and quaternary holes to check closure of the curtain

Table 14 Foundation water testing report

Day: Tuesday
Date: 12.10.76
Shift: Day

Hole No.	Time		Stage				Water			Net Take (L)	Pressure as Intake (KPa)	Remarks
	From	To	Period (Min)	No.	From (m)	To (m)	Start	Finish	Injected			
CT150	8.02 am	8.07	5	3	15	21	60	76	16		110	
	8.07	8.12	5				76	84	8	24	500	
BS151	8.17	8.22	5	2	9	15	84	95	11		110	
	8.22	8.27	5				95	108	13	24	250	
BS152	8.35	8.40	5	2	9	15	108	150	42		110	Slow take with no
	8.40	8.45	5				150	206	56	98	250	signs of stopping
AT114	8.57	9.02	5	3	15	21	6	56	50		110	Same as above
	9.02	9.07	5				56	123	67	117	500	
BS139	9.10	9.13	3	2	9	15	65	165	100		110	Taking quickly; no sign of stopping

Table 15 Foundation grouting report

Day: Thursday
Date: 14.10.76
Shift: Day

Hole No.	Time From	Time To	Time Period (Min)	Stage No.	Stage From (m)	Stage To (m)	Ratio*	Grout (L) In Tank: Grout to Mixer (Bags)	Grout (L) In Tank: Start	Grout (L) In Tank: Finish	Injected	Net Grout Take (L)	Waste (L)	Pressure at intake (KPa)	Remarks:
SP236	10.05	10.08	3	2	10	20	3:1	—	90	0	90			0	
	10.12	10.16	4				3:1	3	280	0	280			0	
	10.20	10.24	4				3:1	3	280	0	280			50	
	10.28	10.32	4				3:1	3	280	0	280			150	
	10.38	10.42	4				3:1	3	280	0	280	1210		120	
	10.52	10.54	2				1:1	4	172	0	172			150	
	11.06	11.21	15				1:1	6	240	120	120	292		200-250	Pump labouring
	11.21	11.36	15				2:1	—	457	360	97			250	Thinned down to 2:1
	11.36	11.51	15				2:1	—	360	356	4	101			Refusal
							Total	22				1 603			

*Ratio: water/cement by volume

when the take of a particular stage was of interest, or to locate a crack by movement of the packer up the hole. A plan summary of the major grout takes is shown in Figure 25.

Grouting Sequence

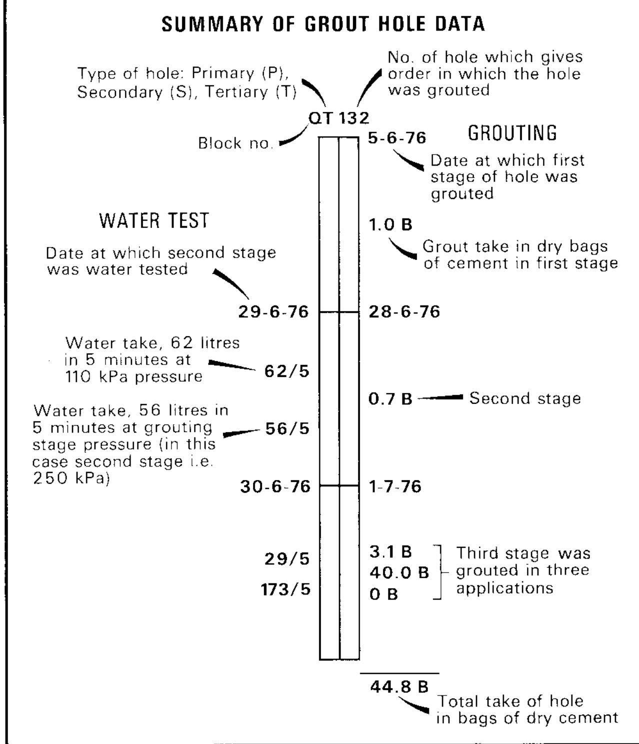
Downstage grouting through a standpipe and nipple was generally used, with the following stage depths and pressures:

First stage	0-7 m	110 KPa
Second stage	7-14 m	250 KPa
Third stage	14-21 m	500 KPa

However on the right abutment, potential uplift caused these procedures to be revised (see later).

Drilling was carried out using two Atlas Corp. lightweight, pneumatic, water flush percussion rigs pow-

Table 16 Summary of grout hole data.



TYPICAL GROUT TAKE CALCULATION

Grout hole **SP 236** — second stage (10—20m)

Total 3:1 take = 1210 litres (no. of litres of dry cement per litre of grout at 3:1)

$= \frac{1210 \times 0.29}{26.5} =$ **13.2 Bags** (26.5: no. of litres per bag of cement)

Total 1:1 take = 292 litres

$= \frac{292 \times 0.67}{26.5} =$ **7.4 Bags** (no. of litres of dry cement per litre of grout at 1:1)

Total 2:1 take = 101 litres
Volume of hole = 36 litres
Net 2:1 take = 65 litres (no. of litres of dry cement per litre of grout at 2:1)

$= \frac{65 \times 0.4}{26.5} =$ **1.0 Bags**

Total take = 21.6 Bags

Drn T.P.G. 84—405 S.A. Department of Mines and Energy ered by a 350 cfm compressor. An airtrack drill powered by a 600 cfm compressor was available as backup.

The drilling and grouting was done in successive operations, consisting of drilling the hole to a limited depth (in this case 0-7 m), washing, water testing, and then grouting. If the grout take was less than one bag of dry cement, the hole was washed out after 2-3 hours to allow redrilling to the next stage (7-14 m) followed by water testing and subsequent grouting of the new stage. The process was repeated for the third and final stage (14-20 m) (Fig. 28). The holes were washed out under pressure using a length of 25 mm diameter Nylex Class 50B PVC hose which was worked from the bottom to the top of the hole until the flushing water became clear.

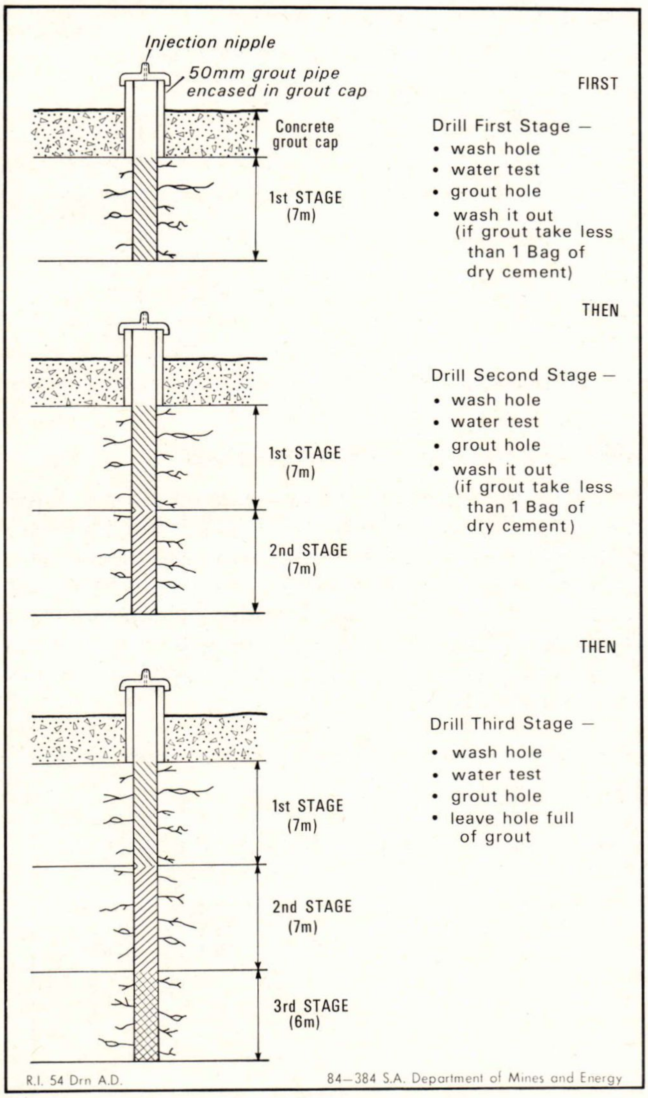


Fig. 28 Downstage grouting without packer: Procedure.

Grouting was started at the bottom of the valley and continued up the abutments. The downstream row of holes was drilled first and, before drilling of any hole on the upstream line was permitted, the downstream curtain had to be closed, a distance of 15 m from each hole (Fig. 29). The criteria for closure of the curtain was that the added grout takes should not exceed two bags of dry cement.

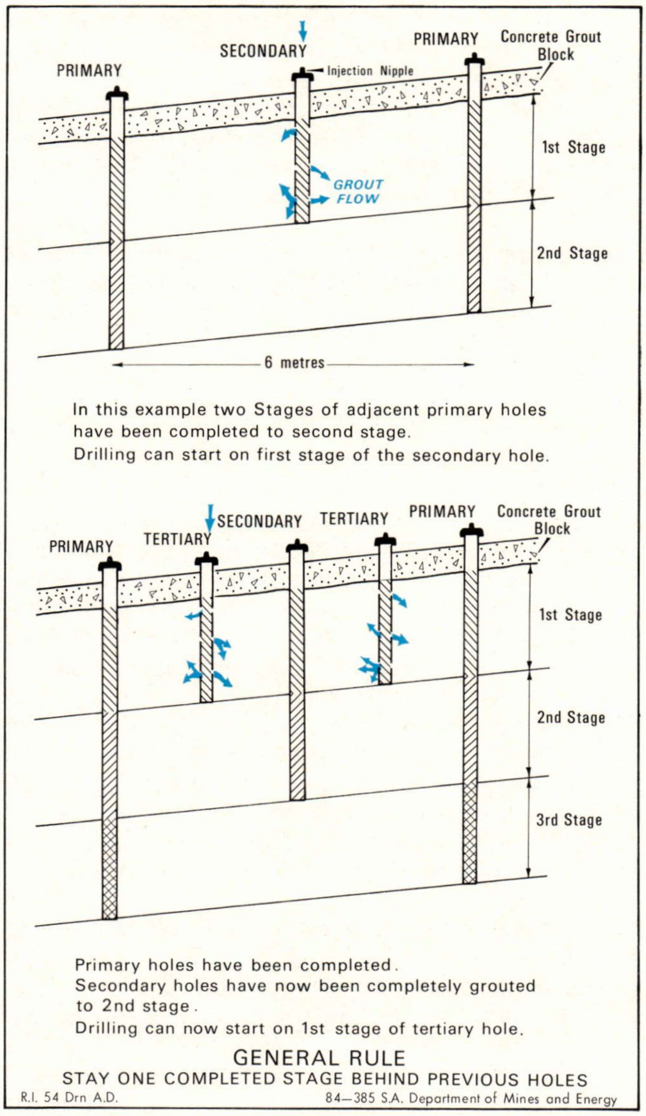


Fig. 29 Downstage grouting: Drilling sequence.

This criterion was later changed to a limit of five bags in the third stage near the top of the right abutment.

The first grout holes drilled were primary holes spaced at 6 m intervals and these were drilled and grouted one stage ahead of secondary holes. These in turn were drilled and grouted one stage ahead of further tertiary holes. This process was repeated for further intermediate holes as required for closure of the curtain. Secondary holes were mandatory but additional holes were normally only required where the take in adjacent secondary hole stages exceeded the closure limits.

Region of No Uplift

This situation corresponded to the bottom area of the right abutment (Blocks H, J, K, and L) and the entire left abutment. In general, each hole was started with an initial mix of 5:1 with a maximum of ten bags of dry cement added before thickening the mix to 3:1. This initial mix was contrary to specification, which indicated a starting mix no weaker than 3:1. If the take was high at 3:1 the mix would be quickly thickened to 1:1 and grouting continued until hole refusal. The hole was said to reach refusal when it took less than or equal to 10 L of grout (seven bags at 1:1) in a 15 min grouting period. If the water test revealed there would be a very high take in a hole, the start mix used was 3:1 or 2:1 and then quickly thickened to 1:1 which was continued until refusal.

The thinnest start mix used was 5:1 and the thickest end mix was 1:1. The end mix was determined from the fact that this was the thickest grout that the paddle mixer could satisfactorily mix.

Region of Uplift

This was confined to the upper half of the right abutment. As shown in Figure 26, the quartzite band (Unit 12) underneath the grout cap feathers out at the top of the right abutment. Between the quartzite and the underlying dolomite (Unit 10) is a band of weathered dolomitic siltstone (Unit 11) susceptible to solution cavities. In addition a shear zone, open joints, and open bedding planes were discovered through Blocks M and N and uplift was observed when these blocks were grouted using the normal pressures. Very high takes were also observed and a great number of additional holes were required to close the grout curtain in this area which included Blocks P and Q.

It was decided to lower the grouting pressures and to change stage depths and grouting procedure to minimise uplift and still ensure that the rock was effectively grouted (Fig. 30). The start mix was changed to 3:1 to ensure localised grouting of the area around the hole instead of driving a thinner grout long distances from the curtain. This method effectively reduced the grouting time for holes with large takes while still sealing the larger fractures and solution cavities in the foundation. No further indication of uplift on these blocks was observed.

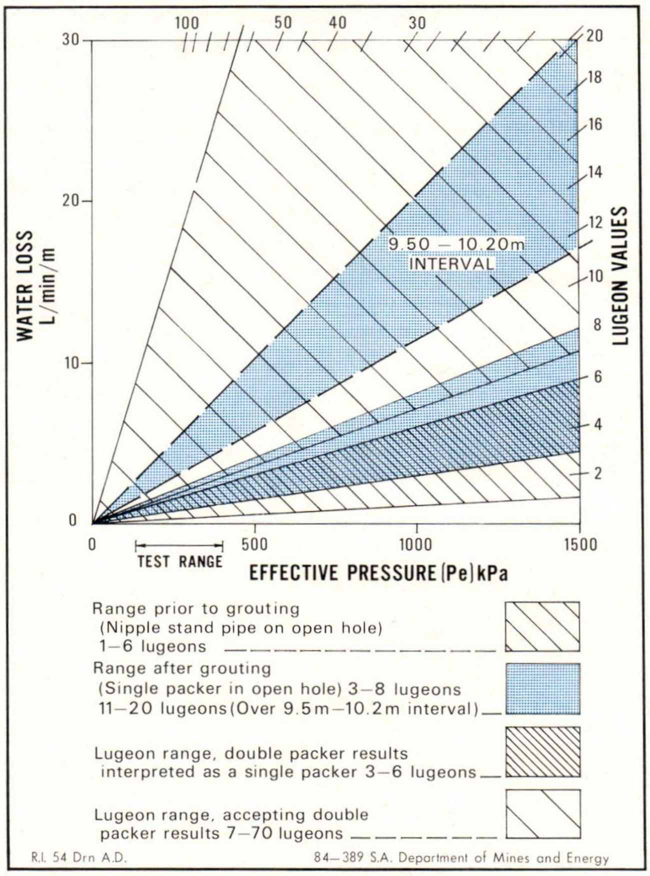


Fig. 31 Drillhole DH24: Water pressure test-lugeon values.

Water Pressure Testing of Check Holes

To check the effectiveness of the grout curtain, three drillholes, DH 23, 24 and 25 (Figs 25 and 26) were drilled within the grout curtain and pressure tested. Lugeon values obtained were compared with values obtained prior to grouting. A geological log is shown in Figure 32 and results of pressure testing given in Figures 33 and 34. An example of recorded data is shown in Figure 34.

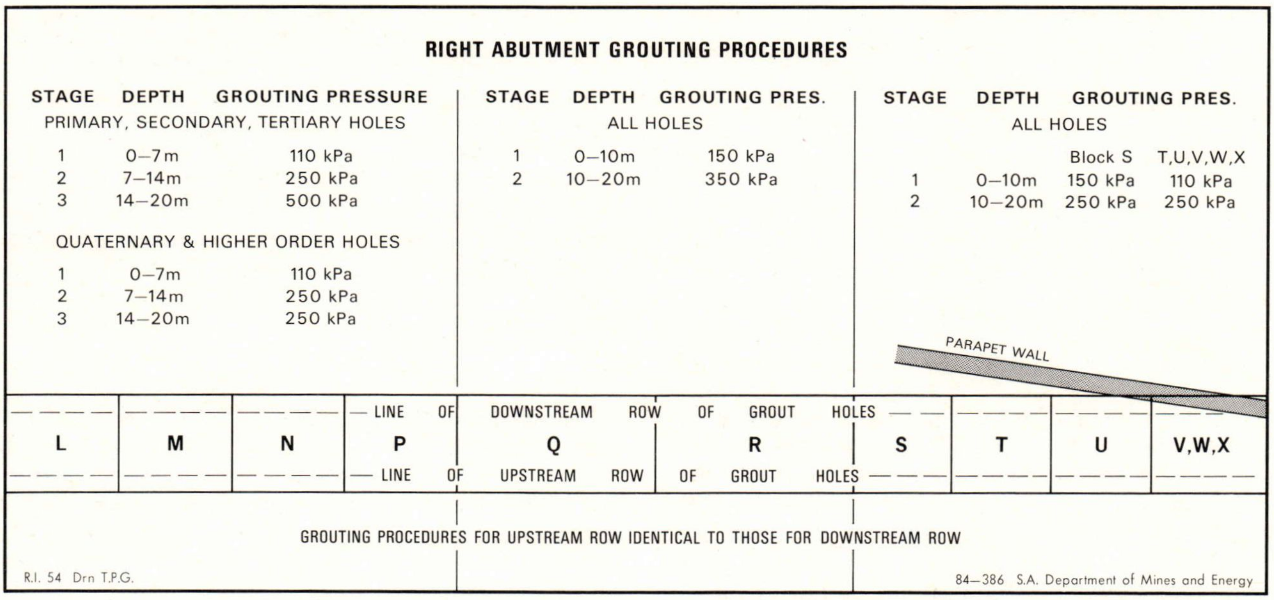
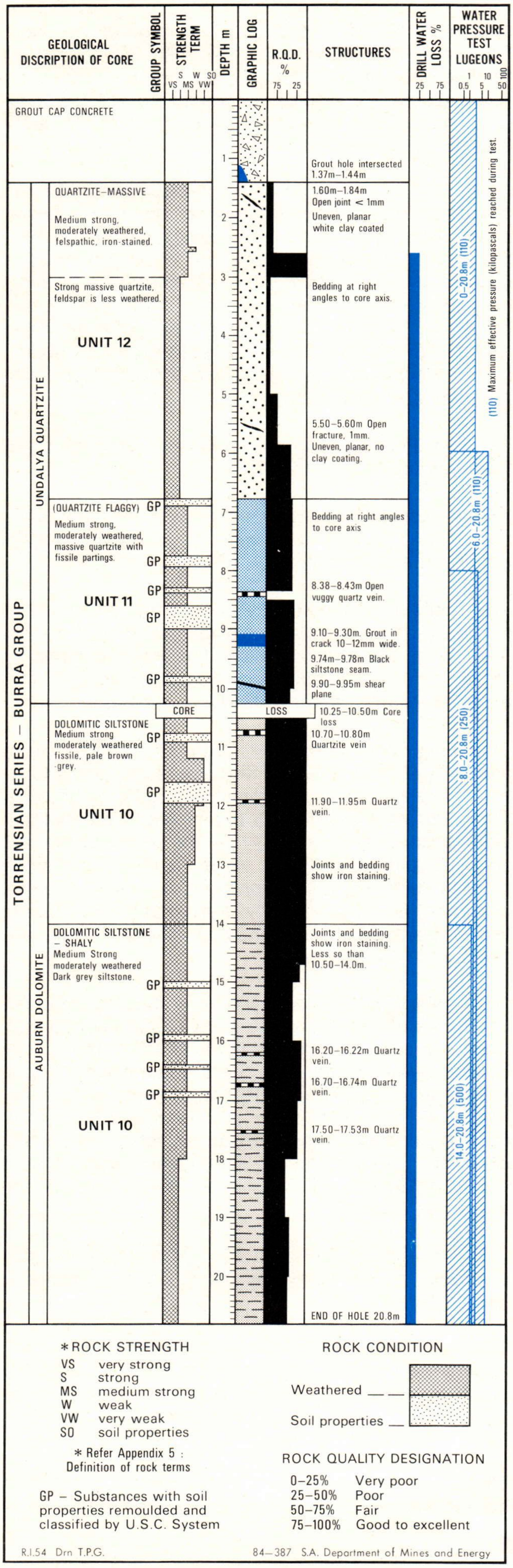
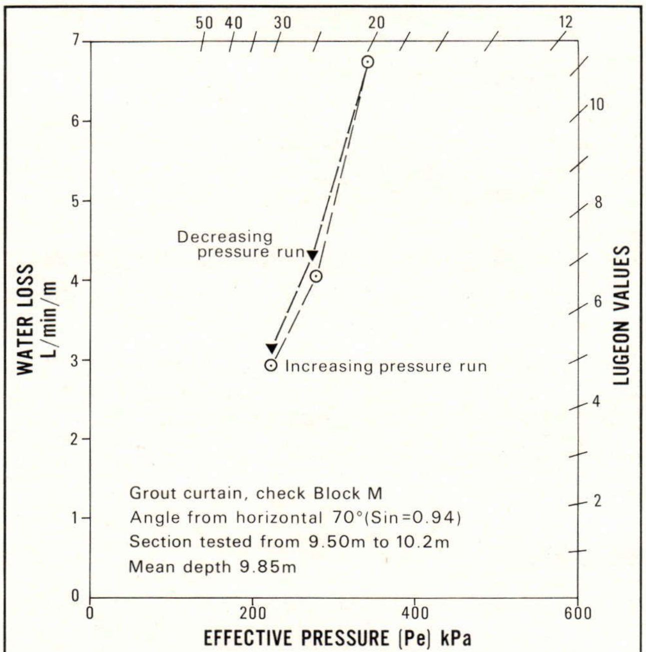


Fig. 30 Right abutment grouting procedures.





Slope height of pressure gauge above collar ______ 0.65m
Groundwater level at beginning of shift __________ 9.90m
Length of test section__0.70m

Packer type:– Single

WATER COLUMN CORRECTION
Slope height of water column above groundwater level :—
•below collar ________ 9.85m
•below pressure gauge, Hw 10.5m
Extra pressure due to head of water column, Pw 96.81kPa

GAUGE PRESSURE	WATER LOSS		SUPPLY LINE CORRECTION Ps	EFFECTIVE PRESSURE Pe
(Pc), kPa	L/min	L/min/m	kPa	kPa
125	2.0	2.9		222
180	2.8	4.0	NEGLIGIBLE	277
250	4.7	6.8		347
180	3.0	4.3		277
125	2.2	3.1		222

Pw=Hw sinθ × 9.81   Pe=Pc+Pw–Ps

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Fig. 32 Drillhole DH24: Geological log.

Fig. 33 Drillhole DH24: Water pressure test-lugeon calculation.

Fig. 34 Drillhole DH24: Examples of water pressure test results.

INTERVAL TESTED	READINGS	TEST NO. 5 DATE:8–9/8/77					PACKER:DOUBLE LUGEON
		a*	b*	c*	d*	e*	
8.0m	GAUGE PRESSURE	125	180	250	180	125	60
9.0m	EFFECTIVE** PRESSURE	265	320	390	320	285	(4–5) ***
(8.0–20.0m)	WATER LOSS	146	146	195	165	145	

INTERVAL TESTED	READINGS	TEST NO. 5 DATE:8–9/8/77					PACKER:DOUBLE LUGEON
		a*	b*	c*	d*	e*	
9.50m	GAUGE PRESSURE	125	180	250	180	125	45
10.50m	EFFECTIVE** PRESSURE	270	325	395	325	270	(3–4) ***
(9.5–20.0m)	WATER LOSS	112	130	142	116	90	

* The five, ten minute water pressure tests run at increasing and then decreasing water pressures as shown.

** Effective pressure recalculated as a single packer test over the interval shown in brackets in first column.

***Lugeon value when results calculated as a single packer test

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APPENDIX 2

Dam Instrumentation

by

B. Adams (E & WS Dept)

SUMMARY

A total settlement of 350 mm was recorded before the embankment became stable. Post construction settlement on the concrete face was 18 mm.

Stress readings of up to 97.47 MPa (tension) and 60.14 MPa (compression) have been recorded in the outlet duct. These figures are within the acceptable limits.

Movements at the grout cap-face slab joints were in the order of from 1 to 2 mm.

Introduction

Instrumentation of the Little Para Dam has been carried out to provide information on loads, settlement, and horizontal movements of the dam during and after construction. The arrangement of the instruments is shown in Figure 35. The observations include:

- Settlement, horizontal movement and relative displacement of the upstream deck concrete slabs.
- Settlement and horizontal movement of the downstream face rockfill.
- Stress distribution in the transverse reinforcing steel of the outlet duct.
- Loads induced by rockfill on the outlet duct.
- Internal settlement of the rockfill.
- Relative movement in joints between face slabs and grout cap blocks.

Instruments

Survey levelling pins were installed to monitor upstream and downstream face movements. The upstream layout consists of 43 pairs of pins placed 305 mm each side of the slab joints at various levels during and after concreting of the face. It was not possible to place all pins during concreting because the wheels of the dragscreed were in line with the pins. Cored holes were used in these instances and the pins were grouted in after concreting.

The downstream layout consisted of ten pins grouted into suitably sized rocks as close to the nominated position as practicable.

Survey triangulation and levelling are used to monitor horizontal and vertical movements of the pins. Relative movement between each pair of pins on the upstream is monitored with an INVAR bar vernier.

Carlson dynamometers were installed to monitor stress distribution in the outlet duct reinforcement. Three gauges were installed on the top face reinforcement and three on the bottom· face reinforcement. The instruments are dual coil meters which allow for temperature correction.

Loads induced by the rockfill on the outlet duct were measured with Hall total pressure cells. A total of 18 cells were placed in groups of six around the duct. Gas tubes from each instrument were channelled to reading stations in the duct. Operation of the pressure gauges is based on pressure changes of the feed gas (dry nitrogen) to maintain a constant flow rate through a dilating diaphragm.

Each instrument was set into an epoxy mortar pad in a blockout on the outside face of the duct. The height of the sensing face of the instrument above the surrounding concrete was required to be 1.6 mm (± 0.8 mm). This close tolerance was achieved by placing an epoxy mortar annulus around the blockout and screeding this flat with a large steel screed plate. After hardening, the instrument was seated into an epoxy mortar which had been set to the correct height with an interval screeding ring.

The instruments were protected with bedding sand from damage by the rockfill.

Internal settlement of the rockfill is monitored using four hydraulic settlement gauges set out in a grid pattern at approximately El. 123.5 m.

The gauges, supplied by Snowy Mountains Engineering Corporation, operate on a simple manometer basis. They were set into protective fine grained bedding material on the embankment and connected to the external reading station with Nylex duplatube.

The initial level check was carried out after field assembly and placement of the gauges was completed, but before backfilling of protective material. Checking the instruments consisted of pumping de-aired distilled water through the system. When the complete system was full and water levels were steady in the sight glasses at the reading station, the overflow weirs were accurately levelled. When the weir and reading station levels coincided, the system was accepted to be air-free and reading correctly.

Each gauge had a dual manometer system with two overflow weirs at the gauge; the weirs are set so that there is a 30 mm difference in the overflow level. This difference was designed to check that the system was air-free and reading correctly during normal operation of the gauges. Each tube and weir of a gauge is flushed separately and allowed to settle. When the difference between the two sight glass readings for a gauge is 30 mm then it is assumed that the system is reading correctly.

Joint movements between the grout cap and face concrete are monitored by using Carlson electrical joint meters.

The joint meters are similar in operation to the strain meters used in the duct except that they have a higher range of operation (12 mm). The higher range is made possible by incorporating a set of bellows at the central part of the gauge casing.

The gauge is constructed in two parts, an anchor socket and the strain meter. The anchor socket was

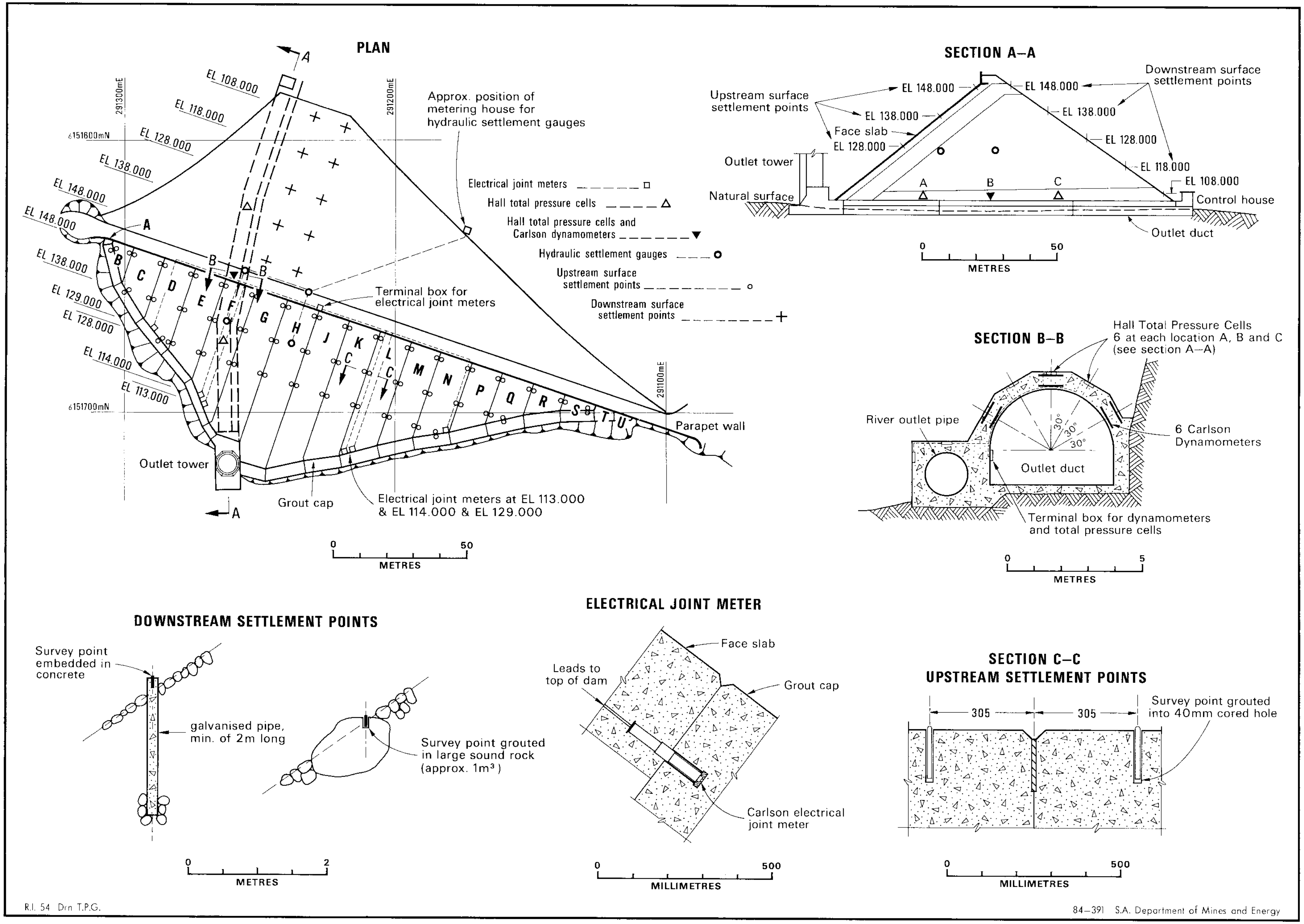


Fig. 35 Arrangement of instrumentation.

placed in position on the first side of the joint to be poured—in this case the grout cap block. When the face slab was ready to be poured the strain meter was screwed into position in the anchor socket and the lead wire run out to the reading station.

Results

Results taken up to September, 1978 show a maximum post-construction settlement of 18 mm measured at El. 148 m on the upstream face. Five readings beyond this date are to be taken at six-monthly intervals.

Embankment settlements are shown in Figure 36.

As would be expected, bulging and settlement have occurred. Movement is not as extensive in the lower part of the bank (below El. 108 m) as it is the central section. There appears to be virtually no bulging and only moderate settlement in the higher bank section (from El. 138 to 148 m).

Typical results of fill settlement indicated by the hydraulic settlement gauges are shown in Figure 37. The results indicate that the settlement rate slowed considerably after placement of the embankment was complete, as would be expected for a granular material.

Stress distribution in transverse reinforcing steel at the centre reading station in the outlet duct is shown in Figure 38 and indicates that stresses are within acceptable limits.

Pressures induced by fill loads on the outlet duct during construction are shown in Figure 39. A comparison of measured pressure and theoretical pressure indicates construction pressures in excess of the theoretical value, which was based on fill density and embankment height at the time of reading. After embankment completion, measured values have relaxed to a value quite close to the theoretical pressure.

The joint meters indicate movement at the grout cap/face slab joints in the range of from 1 to 2 mm, the maximum on Block D at El. 128 m.

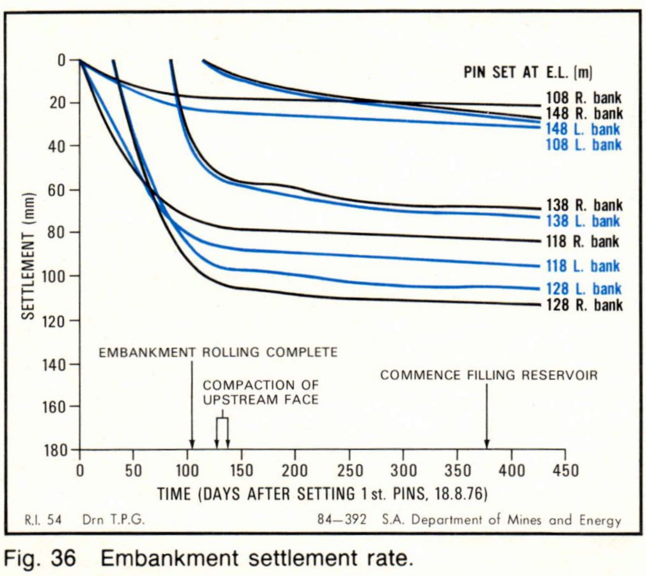


Fig. 36 Embankment settlement rate.

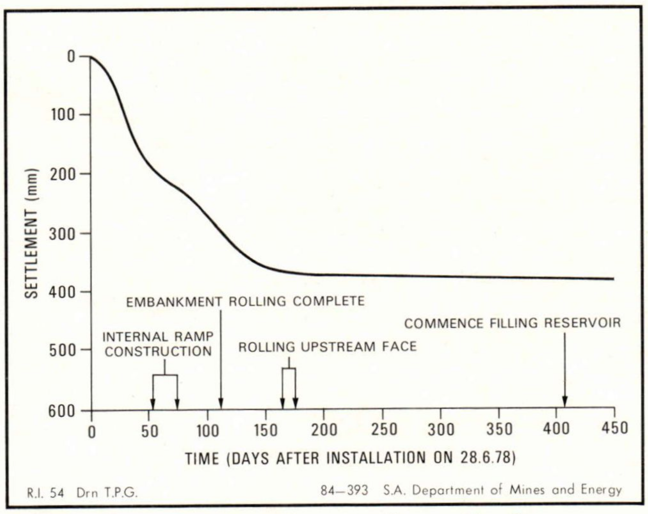


Fig. 37 Embankment settlement—Gauge No. 2.

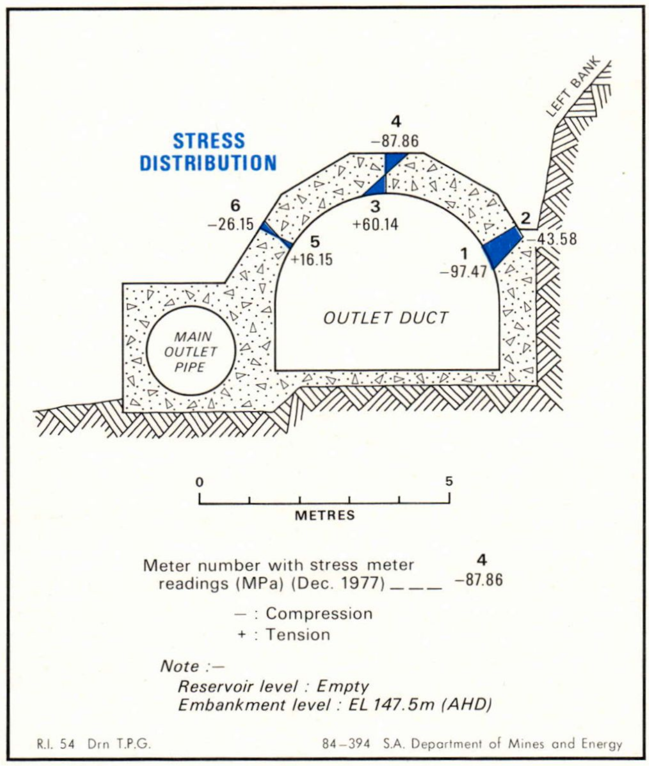


Fig. 38 Outlet duct: Pressure distribution.

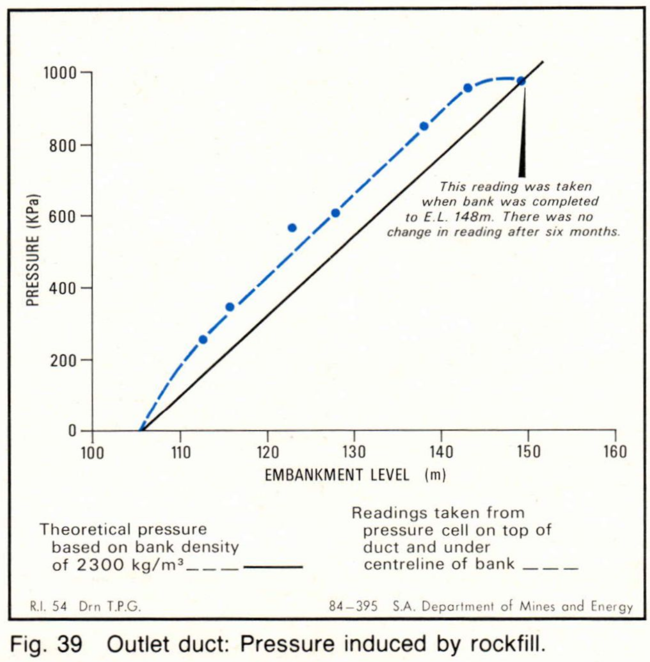


Fig. 39 Outlet duct: Pressure induced by rockfill.

APPENDIX 3

Slope Stability Investigations

by

J.C. Beal

SUMMARY

Hoek Shear Box Tests gave bedding plane friction angles of 35° for quartzite and 25° for moderately weathered dolomitic siltstone.

Stereogram plots of quarry fracture data and spillway joint surveys show stable batter conditions for all quarry faces. Falls of isolated boulders are expected to occur.

Monitoring of hillslope survey pegs shows no indication of soil-slip.

Introduction

A project such as the construction of a dam involves the excavation of many batters in soil or rock. The strata dip at about 30° eastwards across the site and therefore any strike-aligned excavation allowing down-dip movement is likely to be unstable. The residual friction angles for fracture surfaces in the two major rock types found on the site were calculated by consultants Coffey and Partners Pty Ltd using a Hoek Shear Box (Fig. 40). The minimum residual friction angle for quartzite is 35° and for moderately weathered dolomitic siltstone, 25°; these values have been used in the calculation of factors of safety for the quarry investigation discussed below.

Hill Slope Stability

The soil adjacent to the dam is mainly a red-brown calciomorphic soil, up to 1 m thick, resting upon weathered dolomitic siltstone.

Hillslope survey pegs were placed on the right bank upstream and downstream of the dam axis. Monitoring of these pegs over a two year period showed no evidence of movement.

Following construction of the dam the area on the right bank immediately upstream of the dam, involving an approximate soil volume of 45 000 m^3, was re-pegged and monitored prior to filling of the reservoir. This is the only area where it is considered that soil slip into the reservoir resulting from saturation and subsequent reservoir drawdown may occur.

Quarry

Prior to quarry excavation, field data (Figs 41 and 42) indicated the potential instability of quartzite boulders during excavation of a fold in the quartzite stratum (Unit 9) which traverses the quarry. Using this data, a safe sequence for the early quarry excavation was therefore suggested:

— Manual removal of large unstable blocks of quartzite.
— Excavation of the folded part of the quartzite stratum.
— Extraction of dolomitic siltstone starting from the western end and working eastward.

To guarantee the safety of the water treatment plant a second field data survey was carried out to assess the long term stability of the completed quarry batters.

Field data for this fracture analyses are shown as poles plotted on a stereogram in Figure 42. A summary of the field data for the eastern, central, and western batters is shown in Figure 42 and the batters are referred to respectively as Regime I, II and III (Fig. 43). Rock falls from the vertical batters of the central and eastern faces of the quarry can be expected with maximum block sizes of up to 1.0 x 1.0 x 0.5 m. Placement of a shallow ditch and cyclone fence at the foot of the batters has been advised, to prevent falling boulders from reaching the treatment works.

Small rock falls could also be expected from the western batter excavated at 40° and a low safety barrier is to be placed at the foot of the batter. Because bedding plane angle of friction is greater than bedding plane dip a planar failure of the batter into the excavation, although geometrically possible, is highly unlikely. Table 17 gives typical fracture data and their description and Figure 44 shows the stereographic calculation of the stability of rock masses formed by the intersection of fracture planes.

Mylonite occurs in three 200-400 mm wide shear zones 0.5 m apart in the centre of the quarry from El. 110 to El. 123 m within the quartzite stratum traversing the quarry (Fig. 17). Discontinuous joints, parallel to these shears, are clay-coated and where they intersect open bedding planes form the main toppling feature.

Factors of safety for the fractures were calculated by assuming zero cohesion and zero water pressure across the two surfaces. Only the angle of friction was used. This value may be assumed on the basis of results of testing elsewhere of similar rocks or, as in this case, measured in the laboratory on rocks taken from the field (Hoek and Bray, 1974). An *in-situ* test was not carried out because of the high cost. Two coefficients (A and B) have been previously calculated by Hoek for most combinations of plane intersections. These values can be derived using his graphs and substituted in a simple equation with the friction angle. The result gives the factor of safety. Factors of safety above two indicate a stable condition able to withstand all but the most severe seismic activity.

Low factors of safety shown in Table 18 indicate only the instability of small thin slabs of rock (average size 0.7 x 0.3 x 0.3 m) and do not indicate the presence of an unstable rock mass.

Spillway

Stereographic plotting of rock defects, examination of drill core, and a cross-sectional glass model were used to determine excavation conditions and batter angles in the spillway.

Dip and strike measurements of fractures were taken on outcrops of the dolomitic siltstone and quartzite, and their poles plotted on a stereogram.

Table 17 Quarry and water treatment works: fracture description

Location (Fig. 43)	Typical Fracture Directions	Description
Regime I	090/68°N* 160/75°E Bedding 010/20l°E	All joints in dolomite are planar to curviplanar, uneven, ironstained, tight to open with clay coating up to 2 mm thick Average block sizes are in the range: 0.6 x 0.5 x 0.2 m, to slabs 1.4 x 1.0 x 0.3 m, up to large blocks 2.2 x 2.1 x 0.8 m, and rarely 2.7 x 2.2 x 1.7 m; the last two sizes will require breakdown before use in the embankment.
Regime IIA (El. 153-168 m)	020/80°NW, 020/80°NW, 022/75°NW	Planar; uneven; with clay coating up to 3 mm thick; discontinuous against 090/68°N; ironstained.
	080/80°N, 080/78°N, 080/80°N	Similar orientation to 090/68°N uneven, semi-continuous, fresh and clean.
Regime IIB (El. 137-153 m)	040/75°NW 042/80°NW 040/85°NW	Toppling fracture: planar, smooth to uneven, continuous; clean, tight to open (up to 4 mm) 300 to 500 mm apart.
	125/80°SW, 126/78°SW 125/81°SW	Form wedges when intersecting with above groups; planar, discontinuous, open up to 10 mm showing clay coating with carbonate or quartz crystals.
	070/55°N, 070/55°N, 068/55°N	Planar, even, thinly clay-coated, forming very unstable wedges when intersecting with above two groups.
Regime IIC Quartzite Outcrop	010/85°W (dominant fracture) 085/85°N	Planar, uneven, ironstained, with semi-continuous clay coating up to 2 mm thick.
	(i) 040/50°N (ii) 115/85°S fairly common shear pair forming unstable wedges where they intersect.	(i) Planar, smooth, semi-continuous, ironstained with slight clay development (ii) Irregular, uneven, ironstained.
	095/75°N Shear Zones	Mylonite and clay development
	170/22°E Bedding	Curviplanar, uneven, discontinuous, tightly or rarely open to 2 mm.
Regime IIC Central (El. 120-137 m)	005/88°N 000/Vertical 000/12°E Bedding	
Regime III	085/75°N 000/80°W 170/80°E (rare) 005/30°E Bedding	
East of Centre	060/40°N	Common shallow angle joints spaced 0.3-1.2 m apart, planar, continuous, rarely coated; important in reducing overall size of unit blocks. Common size: 0.4 x 0.2 x 0.1 m to fist size. Typical large size 0.8 x 0.5 x 0.3 m; rare large size 1.8 x 0.8 x 0.8 m.
	115/70°SW	
	040/82°N	
	020/90°	
West of Centre	090/75°N	Planar to irregular; smooth to uneven, ironstained, no clay coating, tight to open (up to 4 mm) discontinuous.
	040/vertical; 040/80°SE, 038/85°NW 025/30°E	Curviplanar, even; continuous, ironstained with no clay coating; tight to open (up to 3 mm). Unit block sizes: common 0.6 x 0.3 x 0.1 m to fist size. Typical large size 0.9 x 0.7 x 0.7 m. Rare large size 3.0 x 2.0 x 1.0 m.

*Strike/dip

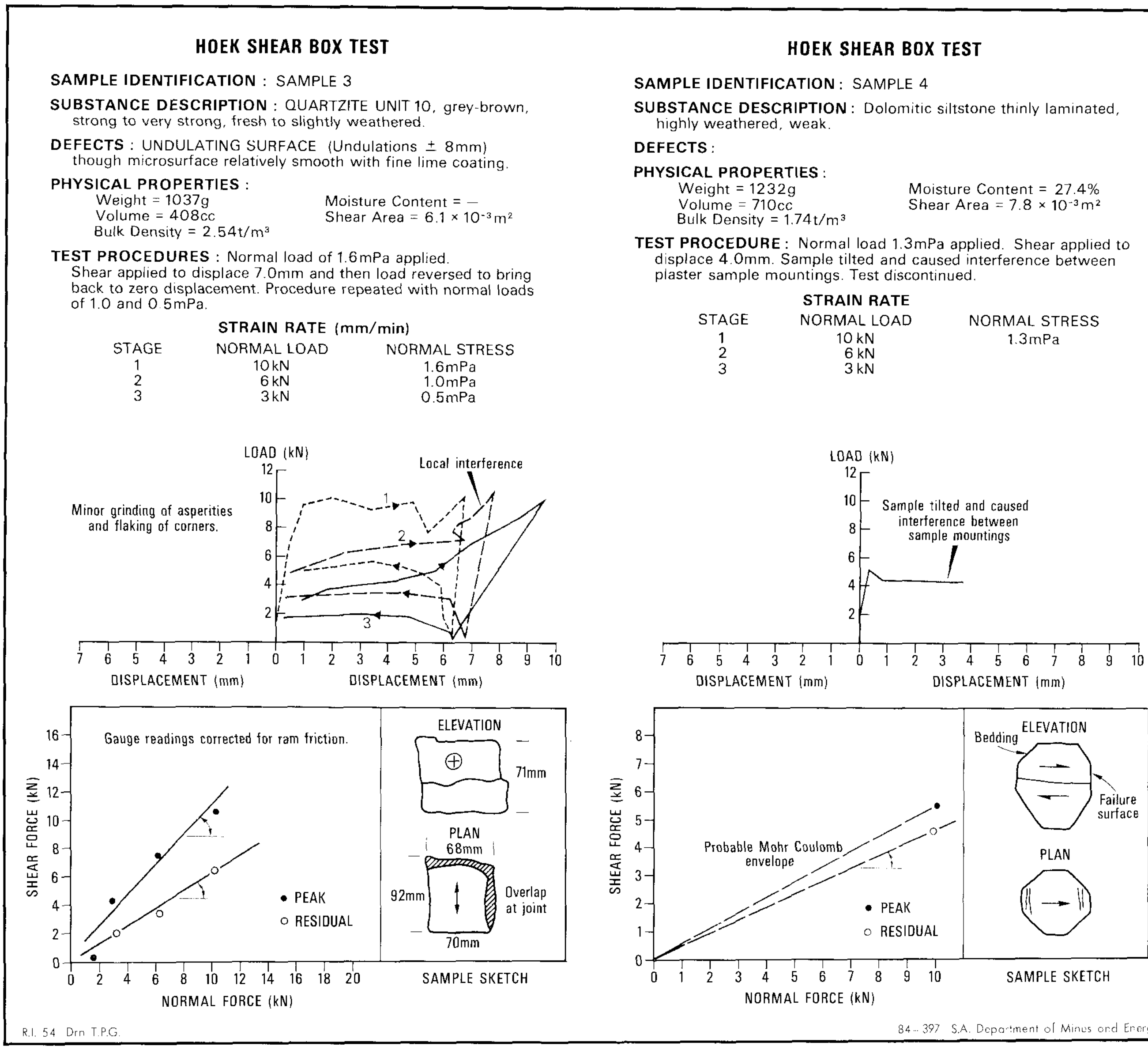


Fig. 40 Hoek Shear Box test: Results.

It was established early in the fracture survey that the strike and dip readings for the joints and bedding planes were very similar for the two rock types but that the joints differed in intensity, frequency and condition. The thickness of the parting between bedding planes and between joints is greater in the quartzite strata; the dolomitic siltstone being more fissile. Maximum block sizes for the quartzite and dolomitic siltstone are shown in Table 19.

Bedding and cleavage in the dolomitic siltstone are sub-parallel to one another. Measurements taken along the spillway alignment show strikes and dips ranging from 005°/29° eastwards to 015°/35° eastwards, with rare exceptions dipping up to 48° eastwards. The mean is taken as 012°/33°E and is shown as Group 1 below:

The mean strike and dip of bedding and major joints are shown as Groups 1 to 4 on Figure 44 and below.

Group	Average Strike	Average Dip
1. (Bedding)	012°	33°E
2. (Joint)	156°	76°W
3. (Joint)	078°	83°W
4. (Joint)	215°	77°W

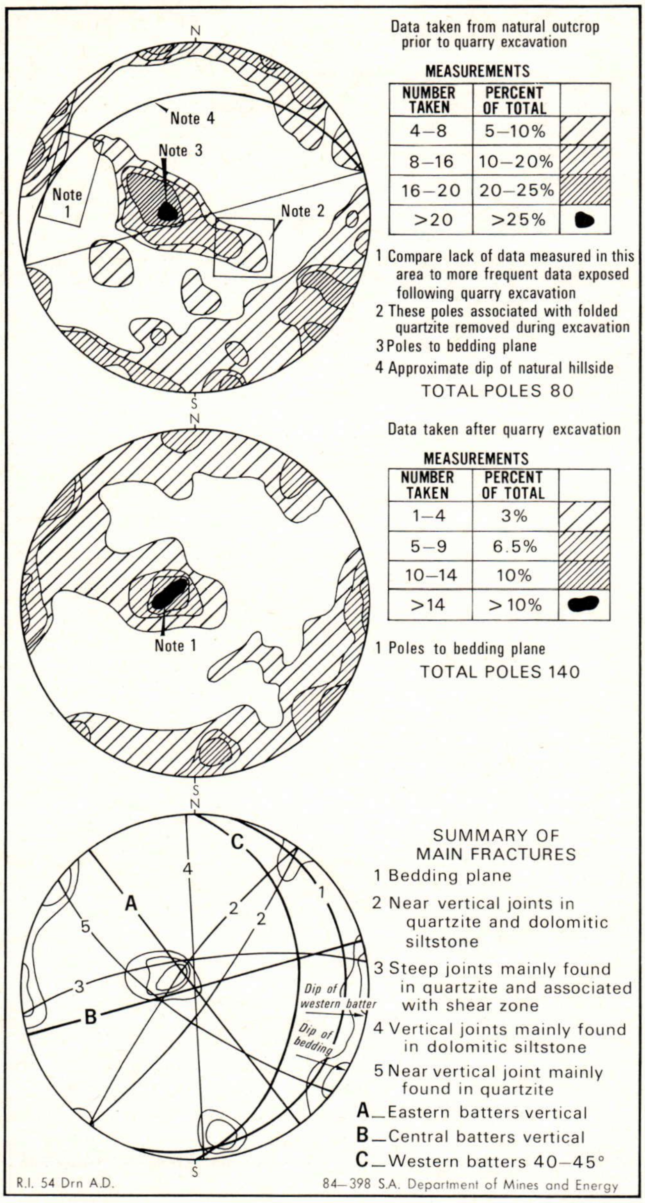


Fig. 41 Quarry: Slope stability analyses.

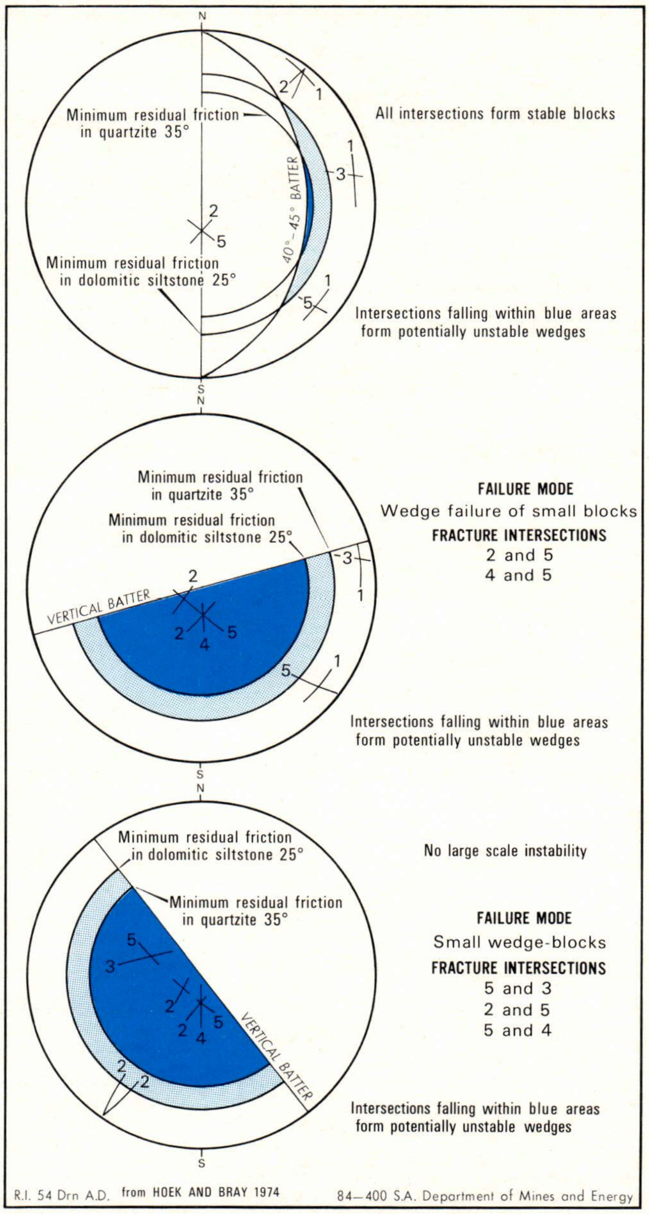


Fig. 42 Quarry: Slope stability stereographic projections.

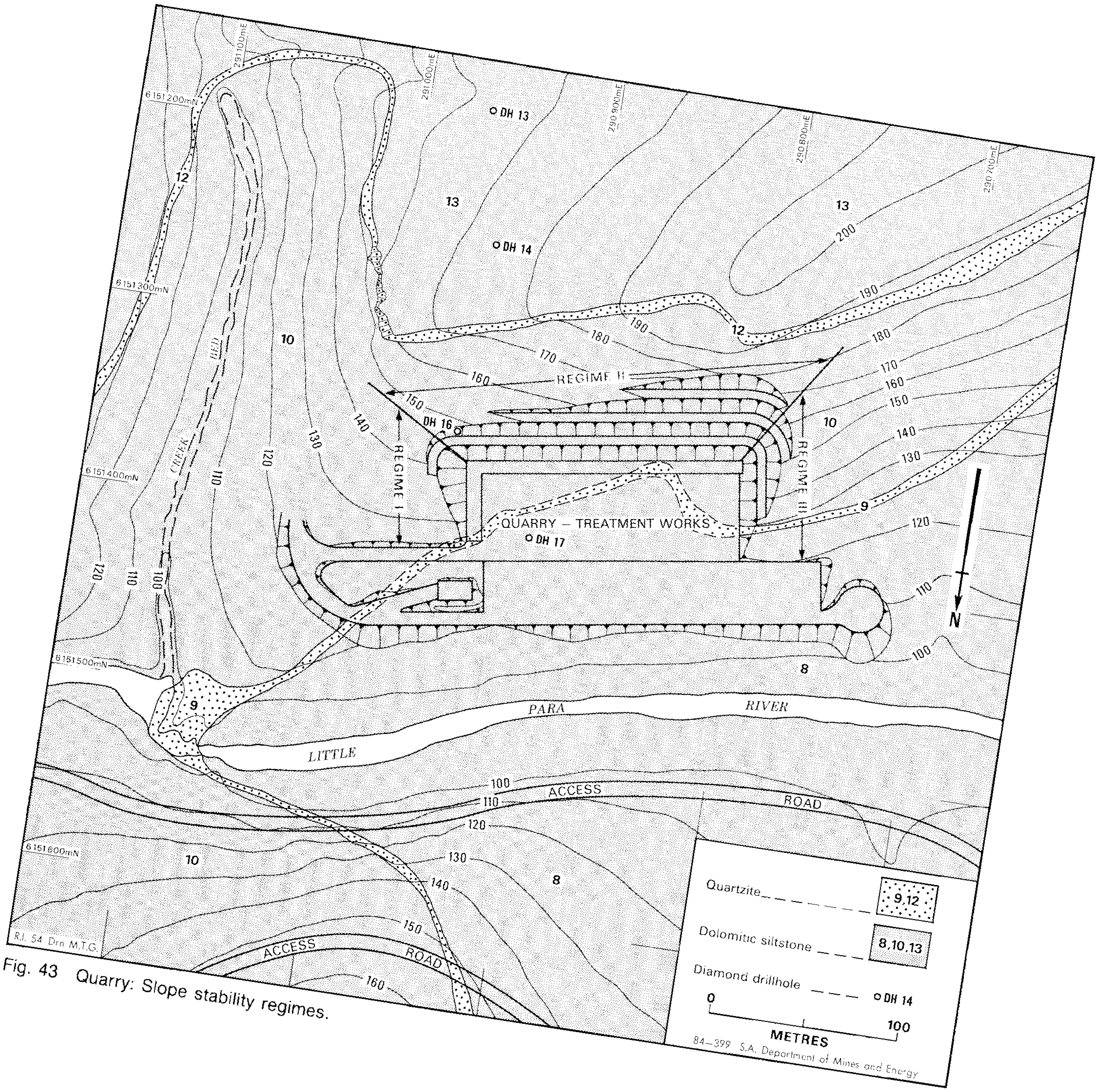


Fig. 43 Quarry: Slope stability regimes.

Table 18 Quarry site: slope stability analysis and safety factors

Intersections (1) Plane A / Plane B	Dip	Bearing	Angle Between A and B: Dip / Bearing	Ø/A / Ø/B	Tan Ø/A / Tan Ø/B	Hoek Coefficients: A Value / B Value	Comments	Safety Factor(2)
1	20°	110°	65°	25°	0.47	>5	The plane designated 1 is the bedding plane. All other planes in the table are near-vertical joints, and intersect to form thin elongate wedges. These are unstable except where they sit upon the bedding plane. In this case removal by crowbar is necessary before the batter is safe enough to work or build near. Fracture 3 has a very variable strike (065-100°). It is associated with poorly developed shear planes and provides potential toppling failures in the centre of the central batter between El. 120 m and El. 130 m.	
2	85°	130°	25°	35°	0.70	2.0		>3.75
1	20°	110°	55°	25°	0.47	3.1		
3	75°	175°	65°	35°	0.70	0.4		1.74
1°	20°	110°	70°	25°	0.47	>5		
4	90°	090°	20°	35°	0.47	2.5		>3.52
1	20°	110°	65°	25°	0.47	2.7		
5	85°	040°	70°	35°	0.70	0.4		1.55
2a	80°	130°	0°	35°	0.70	>5		
2b	80°	310°	180°	35°	0.70	>5		>7.00
2	75°	130°	10°	35°	0.70	>0.3		
5	85°	040°	90°	35°	0.70	<0.1		<0.20
3	75°	175°	10°	35°	0.70	0.6		
5	85°	040°	135°	35°	0.70	0.5		0.77
5	85°	040°	5°	35°	0.70	0.2		
4	90°	090°	50°	25°	0.47	0.2		0.23

Note: (1) See Figs 41 and 42 for identification and location of planes.
(2) Safety Factor = A Tan Ø/A + B Tan Ø/B. Where A is less steep than B.

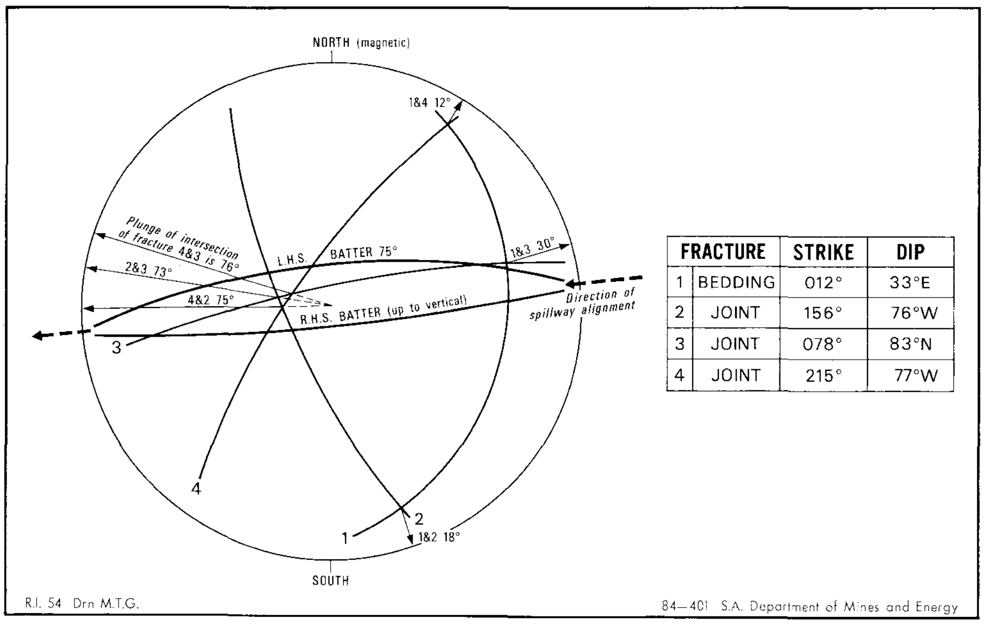


Fig. 44 Spillway: Fracture survey.

Table 19 Spillway excavation: maximum block sizes

Rock Type	Unit No. (Fig. 5)	Volume (m^3)	Approx. Mass (tonnes)
Dolomitic siltstone	17	0.2	½
Quartzite	16	2.0	5
Dolomitic siltstone	15	0.2	½
Quartzite	14	2.5	5½
Dolomitic siltstone	13	0.2	½
Quartzite	12	10.0	25

APPENDIX 4

Summary of Material Tests

During construction of the dam various tests were carried out on the rock and soil materials forming the foundations and used for embankment and road fill and aggregate.

Results are included in:

Table 20	Petrological descriptions of quartzite and dolomitic siltstone
Table 21	Summary of physical properties of quartzite and dolomitic siltstone.
Table 22	Drilling rate and point load test relationships
Table 23	Rock fill and concrete aggregate laboratory tests
Table 24	Testing of completely weathered dolomitic siltstone from right abutment
Table 25	Summary of ring density test results
Figure 45	Grading analyses

Petrological Description of Rock Samples and Their Assessment for Use as Concrete Aggregate.

by

Australian Mineral Development Laboratories, Adelaide

Sample: P384/74, TS33034

Rock Name:

Carbonate-bearing mica schist.

Hand Specimen:

Beige coloured schistose metasiltstone.

Thin Section:

An optical estimate of the constituents gives the following:

	%
Quartz	30-35
Feldspars	5
Muscovite and other Phyllosilicates	20
Carbonate	40
Opaques	1
Biotite	Trace

This sample represents a carbonate-bearing siltstone which has undergone greenschist facies regional metamorphism.

The rock is most probably unsuitable for use in concrete aggregate because it contains dolomite and phyllosilicates.

Independent whole rock XRD analysis indicated the following minerals and proportions.

Dolomite	D	(Dominant)
Quartz	SD	(Sub-dominant)
Muscovite	SD	
Biotite or similar	SD	
Feldspar (Albitic)	A	(Accessory)
Chlorite	A	
K-Feldspar	Tr	(Trace)

Sample: P385/74, TS33035

Rock Name:

Feldspathic quartzite

Hand Specimen:

Fine grained, well sorted quartzitic feldspar-bearing rock.

Thin Section:

An optical estimate of the constituents gives the following:

	%
Quartz	75
K-Feldspar	20-25
Lithic fragments	2
Plagioclase	1
Opaques	trace
Muscovite	trace
Zircon	trace

The rock would most probably be suitable for use in concrete aggregate as it does not contain abundant phyllosilicates, dolomite, or reactive forms of silica.

Sample: P386/74, TS33036

Rock Name:

Feldspathic quartzite

Hand Specimen:

Fine grained, well sorted, weakly bedded feldspar-bearing quartzitic rock containing minor muscovite.

Thin Section:

An optical estimate of the constituents gives the following:

	%	
Quartz	70-75	
K-Feldspar	15-20	
Lithic fragments	1-2	(Quartz-feldspar)
Plagioclase	1-2	
Muscovite	3-5	
Opaques	1-2	
Carbonate (?)	1-3	
Zircon, Sphene, rutile	trace	

The rock is clearly suitable for use in concrete aggregate as it does not contain reactive silica, phyllosilicates, dolomite, or pyrite; the latter causes cosmetic problems due to secondary iron staining.

Sample: P387/74, TS33037

Rock Name:

Weakly metamorphosed siltstone.

Hand Specimen:

Grey green schistose metasiltstone.

Thin Section:

An optical estimate of the constituents gives the following:

	%
Quartz	55-60
Feldspars	5
Muscovite and other Phyllosilicates	30-40
Carbonate	1-2
Opaques	1-2

Texturally and mineralogically this rock closely resembles sample P384/74; however carbonate is much less abundant and there is less evidence of recrystallization. Petrographic evidence suggests that the rock would not be suitable for use in concrete aggregate for the earlier mentioned reasons. Independent whole rock XRD analysis indicated the following minerals and proportions.

Quartz	D
Muscovite	SD
Plagioclase (albitic)	A
Chlorite	A
(?) Biotite or similar	A
K-Feldspar	Tr

Table 20 Petrological description of quartzite and dolomitic siltstone

Rept. No.	Sample No.	Rock Type	Mineral Content %			Phyllo-silicates	Feldspar		Opaques
			Quartz	Dolomite	Carbonate		Plag.	Ortho.	
MP2442/72	P853/72	QUARTZITE (1)	55-65			Trace (sericite/ chlorite)	5-10	20-30	Tr
	P854/72	DOLOMITIC SILTSTONE(2)	3-6	90-95					2-4
	P855/72	DOLOMITIC SILTSTONE(3) (impure)	2-4	90-95					2-4
MP5072/73	P307-310/73 T.S.30620-3	QUARTZITE							

Notes: (1) A well sorted sediment: an arkosic sandstone that has been metamorphosed.

(2) A fine grained mipure dolomite which has been strongly foliated and now has a schistose texture.

(3) As for (2) but lacking the schistose texture.

(4) Recrystallised arkosic sandstone firmly welded by fracturing and crystal growth; feldspar grains have fractured but have been sealed by quartz. A high confined compressive strength for the rock is expected.

Table 21 Summary of physical properties of quartzite and dolomitic siltstone

Substance	Test	Results	Comments
Quartzite	Youngs Modulus	5.3×10^4 MPa	Tested wet and dry.
Quartzite	Specific Gravity	2.593 - 2.650	Gross Apparent (Saturated Surface dried)
Dolomite	Water Absorption	1.2%	
Quartzite	Water Absorption	0.41% - 0.60%	Range of values from different samples tested in different laboratories.
Quartzite	Uniaxial compressive strength	85.5 -296 MPa	Tested wet and dry; samples from 8.0-16.0 m depth; axial failures.
Quartzite	Point Load Tests	Diametral: 36.2-44.8 MPa Axial: 15.5-39.6 MPa	Using NLMC diameter Core.
Dolomite	Point Load Tests	Diametral: 1.7-4.8 MPa Axial: 8.3-13.8 MPa	Using NLMC diameter Core.
Quartzite	Petrological Desc.	Quartz 55-95% Feldspars 5-45% Opaques, Sericite, Chlorite } Traces only	Quartzite to metamorphosed arkosic sandstone
Dolomite	Petrological Desc.	Dolomite 90-95% Quartz 3-6% Opaques 2-4%	Foliated with Schistose texture and referred to in test as phyllitic dolomite.
Quartzite	Ø Shear test	45° - 69°	Measured on plane dipping 22° into proposed Quarry.
Dolomite	Ø Shear test	25° - 30°	Measured along bedding plane in moderately weathered and highly weathered material.

Physical Tests on Completely Weathered Dolomitic Siltstone

by

Soils and Foundations Laboratory, E & WS Dept

Decomposed rock found in the excavation for the grout cap at the top of the right abutment was tested for:

— Grading and dispersion properties
— Atterberg limits
— Permeability

This information was used to determine the likely behaviour of this material when subjected to the head at full storage level of the reservoir.

Three tube samples were taken of soft decomposed dolomitic siltstone underlying a quartzite stratum and all samples were taken from the downstream face of the trench. In addition bag samples of this material were taken. The trench was later infilled with concrete and served as the footing for the parapet wall.

In situ the material appeared to vary over the depth of the deposit. Most apparent variations were in the moisture content, suggesting possible local water paths, and also the brittleness. In places the material was very fractured and could easily be broken into small, strong rectangular prisms. The three sub types ie. normal, higher M.C.%, fractured, were all sampled and an attempt was made to quantify the relative permeabilities in the laboratory.

Comment on Permeability Values

The permeability test was carried out on the actual tube samples as taken in the field and no sub-sampling was necessary. This avoided any further disturbance to the material. Even so, some disturbance when jacking the samples in the field is unavoidable and in this case, because of the presence of layers of very friable material, is likely to cause significant increases in the permeability value as obtained from the laboratory test.

In all tests the operator noticed an early rush of water through the sample which then slowed as a steady state flow was achieved. After the test was completed the samples were broken up and it was noticed that there had been a transportation of the fine particles through the sample. In sample 1 these particles were deposited on a fissure at right angles to the direction of flow. In sample 3 the fine particles migrated through the entire sample. Sample 1 probably gave the only realistic answer.

In samples 2 and 3 the operator noticed water passing through three or four small holes as well as around the edges of the sample. The hole in sample 3 was approximately from 3 to 6 mm in diameter and no sensible result could be obtained.

It would be dangerous to disregard these results altogether as the layers of very friable material break off prismoidally and constitute easy access paths for the water. However, the disturbance associated with sampling will have contributed to the high leakage between the sample and the tube. To get the affect of these causes into perspective is mostly intuitive.

Therefore, it was suggested that a permeability of 5 x 10^{-7} m/sec be used.

This result is similar to one obtained previously when other tests on similar material obtained from the upstream part of the abutment were done.

Results

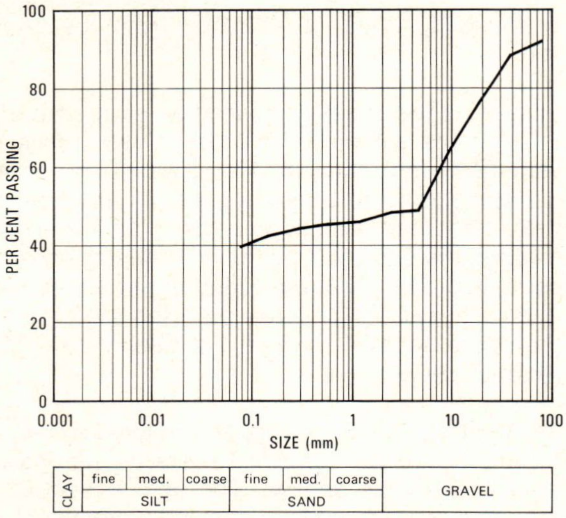
The following results were obtained from standard laboratory testing procedure on the material sampled:

Sample	M.C. %	Sand %	Silt %	Clay %	L.L. %	P.L. %	k(perm) m/sec.	HT of Sample (mm)	Head Water (mm)
1	12	20(f) 5(m)	64	11	34	25	5.5 x 10^{-7}	100	1 530
2	19	29(f) 11(m) 4(c)	47	9	42	31		100	—
3	11	29(f) 8(m) 5(c)	49	9	41	28	8.28 x 10^{-6}	65	1 630

f = fine m = medium c = coarse

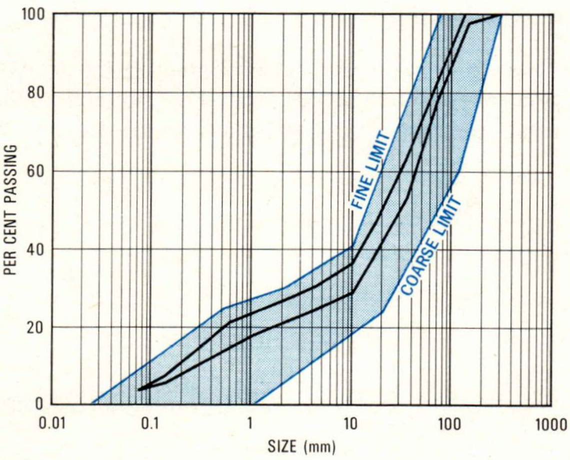
A clay dispersion test was done but no dispersion of fine material was noticed.

MECHANICAL ANALYSIS OF SOIL ROAD FILL



Mechanical analysis	% Passing	Mechanical analysis	% Passing	Soil test constants
75mm		9.5mm(0.1875")	56	LL 46.5 PL 13.7 PI 32.17 LINEAL SHRINKAGE 8.7% COMPACTION TEST Mod. A.A.S.H.O. O.M.C. 11.5% D.D. 120
63mm	96	6.7mm(No.7)	49	
53mm	90	4.75mm(No.14)	47	
37.5mm	84	2.36mm(No.25)	46	
26.5mm	78	1.18mm(No.52)	45	
19mm	71	0.60mm(No.100)	43	
16mm	65	0.425mm(No.200)	40	
13.2mm	56			

RING DENSITY TEST No.2 ON ZONE 1 ROCKFILL (EL 112m)

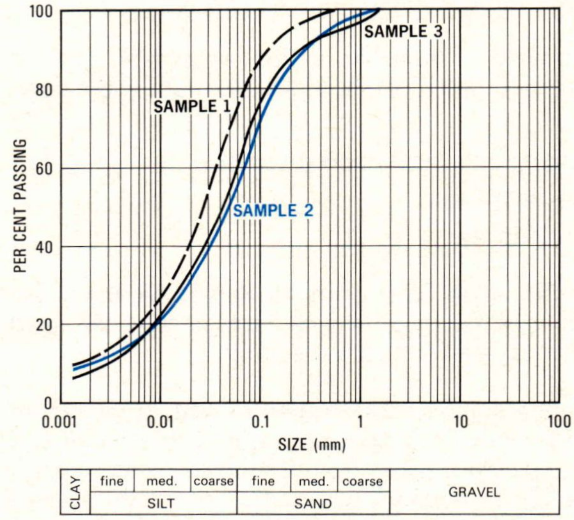


Grading curves Zone 1 grading envelope

DATA FROM ADDITIONAL LABORATORY TESTING

Wet density = 1170lb/ft³ Porosity, N = 27.3%
Dry density = 1150lb/ft³ Appt. SG = 2.55
M/C = 1.7%

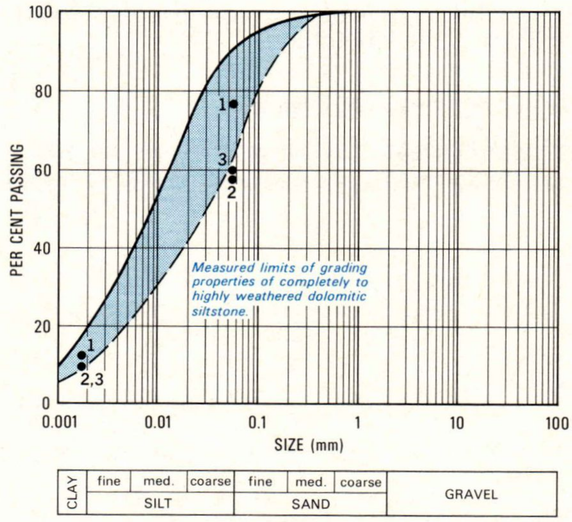
MECHANICAL ANALSIS OF ROCK SAMPLES FROM INSPECTION TRENCH (RIGHT ABUTMENT)



Sample	Description	M.C. %	Hydrometer			Sand Fraction				
			clay	silt	sand	−200	fine	med.	coarse	gravel
1	Decomposed rock	12	11	64	25		20	5	–	
2	Decomposed rock	19	9	47	44		29	11	4	
3	Decomposed rock	11	9	49	42		29	8	5	

Date tested 23.8.75
Mechanical analysis — Weathered dolomitic siltstone

MECHANICAL ANALYSIS OF WEATHERED DOLOMITIC SILTSTONE FROM GROUT CAP AREA



Samples taken from parapet wall trench at top of right bank grout cap blocks T and U ———————— 1 2 3

Sample taken from weathered dolomitic siltstone seam associated with stress relief fracture towards base of block E, left bank grout cap ————————————

Date tested 11.3.75

R.I. 54 Drn T.P.G.

84–402 S.A. Department of Mines and Energy

Fig. 45 Grading analyses.

Table 22 Quarry: Drilling rate and point load test relationships

Drill Hol No.	Drill Interval (m)	Drill Rate (m/hr)	Rock Type/ Comments	Point Load Test		Strength Designation	Unconfined Compressure Strength MPa
				MN/m^2 Axial	MN/m^2(1) Diametral		
1	0- 0.6	34	Surface rubble				
	0.6- 3.0	40	(W) dolomite				
	3.0- 5.3	15					
	5.3- 6.0	6	Rods jamming				
	6.1- 7.9	-					
	7.9- 9.1	31	(MW) dolomitic siltstone				31
	9.1-10.7	35	(MW) dolomitic siltstone				
	10.7-12.2	31	(MW) dolomitic siltstone				
	12.2-13.4	24	(MW) dolomitic siltstone	7		Very high	30 (57 where siliceous)
	13.4-14.4	27	(MW) dolomitic siltstone		2.2	High	
2	0- 1.2	25	(MW) dolomitic siltstone				
	1.2- 3.0	34	(MW) dolomitic siltstone				
	3.0- 4.3	40	Soft weathered dolomitic siltstone				
	4.3- 4.9	29	(W) dolomitic siltstone				
	4.9- 5.8	44	(W) dolomitic siltstone		2.0	High	
	5.8- 6.5	78	(HW)dolomitic siltstone seam				
	6.5- 6.7	10	Quartzite		6.4(2)(3)	Very high	90-290
	6.7- 7.6	21	Flaggy quartzite		0.5(2)	Medium	
	7.6- 9.1	16	Rods Sticking				
	9.1-12.2	37		-			
	12.2-15.2	26	(SW) dolomitic siltstone	5.0		Very high	30 (105 where siliceous)
3	0- 1.2	58	Surface rubble				
	1.2- 2.7	46	(HW) dolomitic siltstone				
	2.7- 3.0	27	Flaggy	1.8(4)			
	3.0- 5.8	21	Quartzite	2.4	High		
	5.8- 9.1	21					
	9.1-10.1	43	(W) Dolomitic siltstone	1.3	High		
	10.1-12.1	20	Rods				
	12.1-13.7	11	Jamming				

Drilling done by air track drill using a 76 mm bit.
Point load tests carried out on core from nearby earlier test holes in quarry area.

Notes: (1) True diametral fracture very difficult to achieve
(2) Fracture along cleavage/bedding plane
(3) Quartzite gave extremely high readings and damaged equipment; new dial required
(4) Part fracture along cleavage/bedding plane

Table 23 Rockfill and concrete aggregate laboratory tests

Table 23 Rockfill and concrete aggregate laboratory tests

Test	Quartzite		Dolomitic Siltstone		Comments
Size	40mm	20mm	40mm	20mm	
Los Angeles Abrasion Test (% weight loss)	21%	19%	21%	18%	Considered very resistant to abrasion.
Aggregate Crushing Value (% weight loss)	Insufficient material	18%	19%	19%	Considered hard material.
Flakiness Index	22%	24%	36%	36%	Australian Standard A.S. 1141 - recommended value less than 35.
Elongation Index	43	64	34	47	A.S. 1141 - recommended value less than 35.
Sodium Sulphate solution (% weight loss)	0.2%		6.5%		12% loss considered unacceptable.
Phyllitic Dolomitic Siltstone Accelerated Weathering Test	Prior to Cycles		Following	10 cycles	Dept. of Main Roads (NSW) test using sodium chloride, on Zone 3 material. Sieve analyses carried out using sized 3/4, 3/8, 3/16, 7, 14, 25, 52, 100, 200. Some breakdown of the coarser material is shown, otherwise very little change following the weathering and compaction tests (modified Proctor).
	L.L.	26-27	L.L.	21-23	
	P.L.	14-15	P.L.	8-9	
	P.I.	12-13	P.I.	13-14	
	Linear Shrinkage	2-5	Linear Shrinkage	No Result	

SUMMARY—The carbonate of the dolomite is not affected to any great extent by the weak acid of the sulphate and chloride tests. Presence of silicates renders dolomitic siltstone more resistant to abrasion but may however increase the flakiness.

Table 24 Testing of completely weathered dolomitic siltstone from right abutment

Test Type	Sample 1	Sample 2	Sample 3
Moisture Content %	12	19	11
Liquid Limit	34	42	41
Plasticity Index	9	11	13
% passing .425 mm	99	93	93.5
Mech Analysis % passing .075 mm	81	63	65.5
% finer than 0.005 mm	17	14	13.5
% clay	11	9	9
*Activity	0.82	1.29	1.44
% finer than .002 mm			

*Some typical Activity values are:

Bentonite		7.2
Calcium Montmorillonite		1.5
Illite		0.9
Kaolinite		0.3-0.5
	Mica	0.23
Clay Size	Calcite	0.18
	Quartz	0

Values for the Activity Index fall within three main groups:

Inactive Clay	0.75
Normal Clay	0.75-1.25
Active Clay	1.25

Table 25 Summary of ring density test results

Zone	Test Date	Elevation (m) AHD	Test Material	Material Specified	Dry Density kg/m³ (lb/ft³)	Bulking Factor %	Compaction (No of passes of 10T VIB Roller)	% Fines Passing 26.5 mm		Remarks
								Test Sample	Specification	
1	27-2-76	112.0 to 110.0	Quartzite: Medium weathered to weathered.	Hard durable rock	1840 (115)	27.6	8 (min. of 4 specified)	47 Refer to Fig. 45 grading envelope	28-67	Grading slightly outside grading envelope on lower sizes. Material slightly on soft side (L.A. Value is 40%).
2	24-3-76	112.0 to 111.0	Quartzite: Fresh with traces of dolomitic siltstone in lower sizes and fines	Hard durable rock	2150 (134.4)	18.5	8 (min. of 4 specified)	40	20	Fines less than 26.5 mm excessive. Sample not truly representative for Zone 2 to El. 112 m. High degree of contamination from adjacent Zone 3.*
4	26-3-76	110.0 to 109.0	Quartzite: Fresh, hard, some medium weathered; few shaley dolomitic fines	Hard durable rock	1973 (123.1)	25	8 (min. of 4 specified)	23.4	20	Fines less than 26.5 mm slightly above specified limit.
6	27-3-76	110.0 to 109.2	Shale & dolomitic siltstone: weathered to slightly weathered	Not specified: poorer quality rock acceptable	1997 (124.6)	21.3	2 (2 specified)	26.7	40	Material and % Fines within specification.
1	1-7-76	125.0 to 124.5	Sound quartzite	Hard durable rock	2116.5 (132.1)	4.9	8 (min. of 4 specified)	38	28-67	
3	9-6-76	121.0 to 120.0	Sound shale: Dolomitic siltstone with some weathered dolomite	Hard durable rock	2010 (125.4)	17.9	8 (min. of 4 specified)	38	30	

*Top 0.5 m of Zone 2 layer was removed and replaced with clean fresh quartzite after the ring density test samples were taken.

APPENDIX 5

Reference Data:

Table 26 Definition of rock terms

1. Rock Condition

Term	Abbrn.	Definition
Fresh	(F)	No weathering effects visible to naked eye.
Weathered	(W)	Shows visible effects of chemical decomposition caused by air and groundwater. Can be subdivided:
Slightly weathered	(SW)	— change in appearance but no loss in strength
Moderately weathered	(MW)	— change in appearance but with significant loss in strength
Completely weathered	(CW)	— has soil properties and often shows complete change in appearance
Altered	(A)	Shows chemical and physical alteration to rock fabric caused by temperature, pressure or injection of other material.

2. Rock Strength

Term	Abbrn.	Kg/cm² (p.s.i.)	Field Test
Very Weak	VW	70 (1 000)	Breaks and crumbles easily in the hands.
Weak	W	70-200 (1 000-3 000)	Breaks easily with hammer tap.
Medium Strong	MS	200-700 (3 000-10 000)	Rings and breaks to firm hammer blow.
Strong	S	700-1 800 (10 000-25 000)	Very difficult to break with hammer and requires sledge.
Very Strong	VS	>1 800 (>25 000)	

Table 27 Summary of permeability values

Soil description	Permeability value	
	Term	k in m/s
Clean gravels	Highly permeable	10^{-2}-1
Clean sands, sandy gravels and gravelly sands	Moderately permeable	10^{-5}-10^{-2}
Fine sands, silts, some weathered clay	Slightly permeable	10^{-9}-10^{-5}
Clays	Effectively impermeable	$<10^{-9}$

Rock mass description	Permeability value	
	Term	k in m/s
Very closely to extremely closely spaced joints	Highly permeable	10^{-2}-1
Closely to moderately widely spaced joints	Moderately permeable	10^{-5}-10^{-2}
Widely to very widely spaced joints	Slightly permeable	10^{-9}-10^{-5}
Unjointed, solid	Effectively impermeable	$<10^{-9}$

Permeability value classification after Geological Society Engineering Group Working Party (Q.J. Eng. geol., 5 (4), 1972).

The Lugeon Test

1 lugeon unit is defined as a water take of 1 litre per metre of test hole length per minute at 10 bars (1000 KPa or 150 psi) and very approximately equates with permeability values of 10 ft/year or 1.2 x 10^{-5} cm/second.

1 lugeon is the degree of permeability encountered in those nearly tight foundations which require almost no grouting at all.

3 lugeons represent a foundation where some grouting may be required, if the dam is to be a concrete one, or where the water is so precious, or piping is so possible, that all seepage must be stopped.

5 lugeons warrants extensive grouting for a concrete dam, or light grouting for some earth rockfill ones.

10 lugeons warrants grouting for most types of dams.

20 lugeons is typical of heavily jointed sites with relatively small joint openings.

100 lugeons is encountered in heavily jointed sites with relatively open joints. It is also met in sparsely cracked foundations where joints are very wide open.

(Taken from Houlsby, 1976).

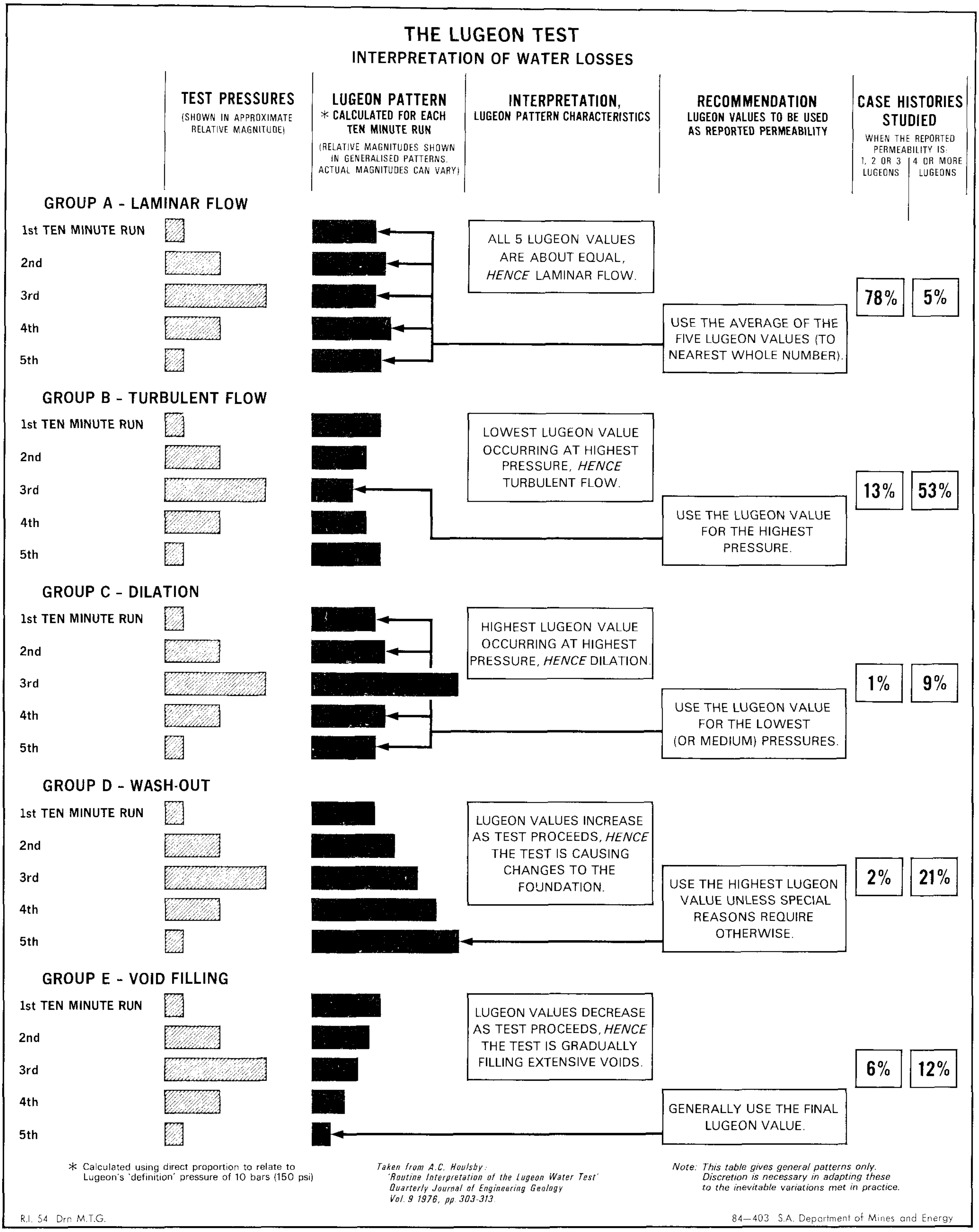


Fig. 46 The lugeon test: Interpretation of water losses.

Table 28 The Unified Soil Classification.

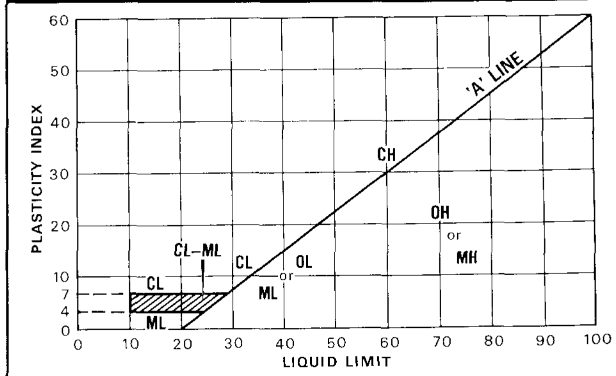
			FIELD INVESTIGATION PROCEDURES Excluding particles larger than 7.5 cm and basing fractions on estimated weights	GROUP SYMBOL	GROUP NAME and typical materials	LABORATORY CLASSIFICATION CRITERIA	
COARSE—GRAINED SOILS More than 50% of material is larger than No. 200 B.S. sieve size	**GRAVELS** More than 50% of the coarse fraction is larger than 2 mm. (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	GW	**GRAVEL, well graded;** gravel and mixtures, little or no fines	$Cu = \frac{D60}{D10}$ Greater than 4 $Cc = \frac{(D30)^2}{D10 \times D60}$ Between 1 and 3	
			Predominantly one size or a range of sizes, with some intermediate sizes missing	GP	**GRAVEL, poorly graded;** gravel sand mixtures, little or no fines	Not meeting all gradation requirements for **GW**	
		DIRTY GRAVELS Appreciable amount of fines	*Non-plastic fines—for indentification see **ML** below*	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
			Plastic fines—for identification see **CL** below	GC	**GRAVEL, excess clayey fines;** poorly graded gravel-sand-clay mixtures	Atterberg limits above "A" line or PI greater than 7	
	SANDS *More than 50%* of the coarse fraction is smaller than 2 mm. (passing B.S.7 sieve)	**CLEAN SANDS** *Little or* no fines	Wide range in grain sizes, and substantial amounts of all intermediate particle sizes	SW	**SAND, well graded;** well graded sands, gravelly sands, little or no fines	$Cu = \frac{D60}{D10}$ Greater than 6 $Cc = \frac{(D30)^2}{D10 \times D60}$ Between 1 and 3	
			Predominantly one size or a range of sizes, with some intermediate sizes missing	SP	***SAND, poorly graded;*** poorly graded sands,gravelly sands, little or no fines	Not meeting all gradation requirements for **SW**	
		DIRTY SANDS Appreciable amount of fines	Non-plastic fines—for indentification see **ML** below	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
			Plastic fines—for identification see **CL** below	SC	**SAND, excess clayey fines;** poorly graded sand-clay mixtures	Atterberg limits above "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

PER CENT 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		

		FIELD INVESTIGATION PROCEDURES on fraction smaller than 0.4 mm. (passing B.S. 36 sieve)						GROUP SYMBOL	GROUP NAME and typical materials
		SOIL CAST (soil wet)	SOIL THREAD	SHINE	DILATANCY	ODOUR	DRY STRENGTH		
FINE—GRAINED SOILS More than 50% of material is smaller than No. 200 B.S. sieve size	**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	ML	**SILT SOIL, low plasticity;** inorganic silts and very fine silty or clayey sands, rock flour
		Cast may be 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	CL	**CLAY SOIL, low plasticity;** inorganic clays of low to medium plasticity, gravelly clay, sand, clays, silty clays, lean clays
		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	OL	**ORGANIC SOIL, low plasticity;** organic silts and silt clays of low plasticity
	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	MH	**SILT SOIL, high plasticity;** inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
		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	CH	**CLAY SOIL, high plasticity;** inorganic clays of high plasticity, fat clays
		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	OH	***ORGANIC SOILS, high plasticity;*** organic clays of medium to high plasticity
		Readily identified by colour, odour, spongy feel and frequently by fibrous texture						Pt	**PEATY SOIL;** Peat and other highly organic soils



PLASTICITY CHART
For laboratory classification of fine–grained soils

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

Based on "The Unified Soil Classification System" United States Department of the Interior, Bureau of Reclamation "Earth Manual" First edition, Denver, Colorado 1960

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