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METROPOLITAN WATER BOARD

R I S D O N   B R O O K   D A M

EIGHTH REPORT

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H O B A R T

AMG REFERENCE POINTS ADDED

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252002

METROPOLITAN WATER BOARD

RISDON BROOK DAM

EIGHTH REPORT

SUMMARY.

Risdon Brook Dam is proposed as a storage reservoir of eight hundred (800) million gallons capacity for the Clarence area. It would enable the surplus winter flow of the Southern Regional and the Derwent pipelines to be stored for use in summer, thereby making full use of these pipelines.

In the Fourth Report of July 1964, the feasibility of a dam about one hundred feet (100') high was established and an approximate estimate of cost given, together with proposed lines for further site investigations.

Whilst these investigations will not be complete for several months, the progressive results so far have been satisfactory and nothing very unusual is now expected in the remainder. A closer estimate of cost is now possible.

The investigation into the porosity of the reservoir valley and the dam foundation rock has included seismic surveys and pressure water testing of drill holes, nothing out of the ordinary has resulted.

The further investigations into the availability of suitable dam building materials have shown an adequate quantity of satisfactory dolerite rock for the rockfill shell of a dam, however as yet no satisfactory clay deposits have been located for the impervious zone.

It is now definitely proposed to build a rockfill dam with a concrete upstream face, and because of an apparent feature of the dam foundation, a conservative approach has so

far been made to the design, in that flatter dam slopes than usual have been adopted, with several rockfill weighting or stability berms on the downstream slope. Further rock tests now under way may permit the use in design of somewhat higher strength values with a resulting reduction in the rock quantities in the dam.

Further geological studies have been made by Mr. Jennings of the Mines Department and by the Associate Geologist, Mr. Mather, and their reports are appended.

Whilst over the past few years there has been very little flow at all in Risdon Brook, it is noted that water from its catchment is mineralised, particularly at low creek flows. It is proposed to cut off altogether the low to medium creek flows, and pipe around the dam so that except in flood flows, all the water of the Brook is diverted around the dam, thus preventing the stored water from becoming harder.

The proposed method of distributing water from the dam is discussed. Briefly it would be supplied to the Clarence system by pumping from a pumpstation below the dam to the Risdon Vale reservoir, whence it would gravitate by a twenty-seven inch (27") diameter main through Lindisfarne to Warrane.

The total estimated cost of the Project is £ 1,000,000. 0. 0., and a reasonable date for its completion would be September, 1968.

I N D E X

This Report comprises the following:-

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The following drawings are included with this Report:-

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METROPOLITAN WATER BOARD

R I S D O N   B R O O K   D A M

EIGHTH REPORT

SECTION 1.00 GENERAL

1.10 - Introduction :

1.11 - This Report has been prepared at the request of the Metropolitan Water Board to consider the capital cost of the Risdon Brook Dam project including its accompanying pipelines, and to decide on the type of dam to be constructed.

1.20 - Previous History :

1.21 - In the Consulting Engineers 14th Report to the Clarence Commission of February 1960, Risdon Brook was noted as a possible site for a storage to store surplus winter capacity of the Southern Regional Scheme for use by Clarence in summer and thus make full use of the Scheme. The site is close to Clarence development, being about one mile from Risdon Vale, and 4 miles from the Tasman Bridge. The Report tentatively discussed a dam eighty feet high to store about four hundred million gallons of water to be pumped to the Clarence supply at Risdon Vale and Warrane.

1.22 - In 1961 the Rivers and Water Supply Commission installed a recording gauging weir on Risdon Brook primarily to determine the likely contribution to a storage from stream flow.

1.23 - In 1963 the Metropolitan Water Board asked the Consulting Engineers to report, initially, on the feasibility of the site for a dam of about 100 feet in height. The Fourth Report was submitted in July 1964 following some exploratory drilling and extensive geological investigations and it established that a dam of about 100 feet in height could be built on Risdon Brook, storing eight hundred million gallons of water, for an approximate cost of eight hundred thousand pounds.

1.30 - Further Investigations :

1.31 - The Fourth Report also indicated lines for further necessary investigations to enable a decision to be made as to the type and size of dam and its cost. Such investigations are in progress and will probably continue up to the stage of final design. The results of the investigations so far are discussed in Section Two of this Report.

1.40 - Purpose of the Reservoir :

1.41 - Its purpose is to store surplus winter time capacity of the Southern Regional and the Derwent supply schemes for use in Clarence in the summer, and with its proposed capacity of eight hundred million gallons it could augment the summer supply by at least an average of five million gallons per day over the one hundred and fifty days taken as the watering demand period over a very dry summer.

1.42 - The present maximum bulk supply available to Clarence totals nearly seven million gallons per day made up of three and one half million gallons per day from the Southern Regional Scheme and three million gallons from the Derwent Supply Scheme. The construction of Risdon Brook Dam would increase the ultimate supply available to Clarence over a dry summer by about seventy five per centum.

1.43 - The reservoir would not be filled by natural runoff from the catchment, as this runoff would mostly be too hard for domestic use, and it would be bypassed around the reservoir. This is discussed in Section Five of this Report. All water to fill the reservoir would come from the Metropolitan supply system over the winter period, either by pumping from the Derwent pipelines or in a suitable winter partly by gravity. The system proposed for use of water from the reservoir and its replenishment is discussed in Section 6 of this Report.

1.44 - A reservoir to store water pumped into it preferably should be more watertight than reservoirs storing large volumes of natural flows for hydro-electric irrigation or flood control, as leakage out of the basin would represent with the former a definite loss in pumping and treatment cost, and measures to prevent seepage from the reservoir can be financially justified.

1.50 - Geological Investigations :

1.51 - Reports on the geological aspects have been prepared by Mr. I. Jennings, Senior Geologist of the Department of Mines, and the Associate Geologist Mr. R.P. Mather. These reports are attached as Appendices A and B and in general it can be said that there are no difficulties about the site that cannot be countered by adequate design and construction. The regional geology of the area is shown on Drawing Number One, "Regional Geology", attached to this Report.

SECTION 2.00 SITE INVESTIGATIONS

2.10 - Purpose of Site Investigations :

2.11 - Investigations of the site have been carried out in three main directions. These are:

(i) To determine that the foundations of the Dam have sufficient structural strength to withstand the loads applied by the weight of the Dam, and hydrostatic force from the water which it retains.

(ii) To determine that the stored water basin is sufficiently watertight and if it is not, to determine what has to be done to make it so.

(iii) To determine what materials are available for the construction of the Dam, and having found them to check their adequacy.

These are now separately considered :-

(i) Strength of Dam Foundations

2.20 - Nature of Foundations :

2.21 - The geology of the dam site is shown on Drawing Number Two "Geological Map of Risdon Brook Area" and is discussed in detail in the appended geological reports. The dam is founded on slightly tilted siltstone beds of the Permian Ferntree formation. There is a minor geological fault across the valley downstream, and a major fault known as the Lindisfarne fault upstream.

2.22 - At the time of writing this report nine diamond drill holes have been drilled into the dam foundation rock and about one Thousand (1000) lineal feet of core has been obtained. Drilling is continuing under a contract with

Foundation Engineering (Aust.) Pty. Ltd. Further drilling has, and is taking place to determine the exact location and properties of the upstream and downstream faults.

2.23 - The position of the drill holes on the damsite are shown on Drawing Number Three, "Investigation Details". Also shown on this drawing are the lines of seismic traverses carried out by the Bureau of Mineral Resources of the Department of National Development.

2.24 - The drilling supported by the seismic traverses has indicated that generally the foundation rocks are sound although weathering is in evidence up to fifty feet deep in the abutments. The sediment beds dip gently to the south-west and some seams between the beds appear to be clay-filled, with a thickness of up to two inches.

2.25 - Samples of rock from cores have been tested by the Bureau of Mineral Resources to determine the physical properties of the rock. The results of the Bureau tests are tabulated in Appendix 'C' and have been used subsequently to verify that the foundation rocks are sufficiently competent to take the applied loads.

2.26 - It is not possible from the drill cores to assess the properties of the clay in the inter-bed seams for the reason that the clay must be tested in its natural or undisturbed state and it is not possible to do this in a two and one eighth inch ($2\frac{1}{8}$") diameter drill hole. Also clays are washed out of the hole during the drilling process so that even the classification and other tests which can be carried out on reconstructed samples are not possible because the clay is virtually unobtainable by drilling.

2.30 - Testing of the Clay Seams :

2.31 - To ascertain the strength of the clay in a typical seam, two blocks of rock about two feet square were exposed and prepared above a clay seam and shear tests were carried out using hydraulic jacks to provide the shear and normal forces, and dial gauges to register the deformation. The derived properties of cohesion and angle of internal friction were low.

2.32 - The location of the tests is shown on the Drawing Number Three. The seam tested was at about creek level and on the upstream side of the dam. Whether this seam is typical is uncertain but the low strengths revealed have been used for the design parameters, and lower strengths are unlikely to occur over the dam foundation area. This conservative approach is justifiable at this stage.

2.33 - At the bottom of the dam the strength of clay seams is not as important in that failure by sliding is prevented by a slight downstream component of dip in the sedimentary beds which means in effect that the beds would have to displace the valley floor before they could slide. In the abutments the situation is different in that their physical shape would tend to permit the beds to slide more readily when combined with a slope failure of the embankment, and this is considered later in this Report.

2.34 - Economies in construction might be affected if the clay seam strength is known more accurately in the abutments and for this purpose an adit or horizontal tunnel is being excavated into the eastern abutment in the location shown on Drawing Number Three. This adit will lead to the beds which are the most critical for sliding considerations and in-situ rock tests will be made from this adit and the results will be used for the final design.

2.40 - Downstream Fault :

2.41 - The downstream fault is approximately parallel to the longitudinal axis of the dam and intersects the creek about one hundred and fifty yards below the proposed centre line. Drill holes D2 and D3 have established that the dip of the fault must be nearly vertical and trenches dug through the overburden to the surface of the fault have located its strike and width at the surface. Holes D4, D5 and D6 are part of the present programme of drilling to determine the nature of the fault at depth.

The average width of the decomposed and brecciated material in the fault is four feet at the surface. The fault is shown on Drawing Number Four "Geological Sections".

(ii) Watertightness of Reservoir Basin

2.50 - Porosity of Dam Foundation :

2.51 - The relative watertightness of the foundation has been assessed qualitively from the general geology. In addition the recent diamond drill holes in the foundation area have been pressure water-tested in stages as they are drilled so as to give the porosity of the rock at varying depths. Water testing consists of measuring the water lost in the hole when it is pumped in at a pressure roughly equal to the hydrostatic head to which the section being tested will be subjected when the dam is full of water. To date four holes have been water tested, namely Numbers P7, T8, P8 and J1. The locations of these water tests is shown on Drawing Number Three. They are in the abutments. Generally water losses have been fairly high in the upper fifty feet of each hole but not higher than expected. Permeabilities of the order of 400 - 600 feet per year are average for the top fifty feet of each hole; losses in the sounder rock below have been negligible. Permeabilities are shown on Drawing Number Five.

2.53 - The permeability readings so far obtained in the abutment testholes indicate the necessity for grouting if leakage around the dam is to be avoided, and they also indicate that grouting should not be particularly difficult.

2.60 - Watertightness of Dam Basin :

2.61 - The basin of the dam as a whole is considered to be watertight in view of the area geology and the general physiography. The only area which is suspect from this aspect is the saddle in the Triassic Sandstones immediately to the north-east of the dam. Although the water path is quite long at top water level (about 700 feet) the dip of the beds is conducive to leakage, particularly if the joint system is extensive and open.

To obtain information about the saddle at depth, the Bureau of Mineral Resources carried out some seismic surveys and resistivity surveys and the detail results are not yet to hand. The verbal advice from the geophysicists indicates a considerable depth of weathering, which has been confirmed by test drill holes Permeabilities obtained from water tests so far at levels below the top water level of the dam,whilst appreciable,decrease fairly rapidly and it is now expected that sufficient water-tightness of the saddle rock can be obtained by reasonable grouting.

2.62 - The "upstream fault" is the main Lindisfarne fault which intersects the valley about two hundred and fifty feet (250') above the dam centre line. It is not considered that this fault will affect the structural stability of the dam, but it needs to be thoroughly investigated to ensure that it will not give a too ready access for leakage water to the deep dam foundations and trend to circumvent any grout curtain established beneath the dam. Also if the fault zone were very porous it could provide a leakage path through the saddle area to the Grass Tree Hill Road. Four drill holes, U1, U2, U3 and U4 are planned to examine this aspect and provision has been made in the estimates for some additional grouting to seal the likely porous upstream fault zone.

(iii) Investigations for Materials

2.70 - Exploration for Construction Materials :

2.71 - To a large extent the type of dam that is ultimately built upon the site depends upon the local materials available for its construction and a thorough exploration is of importance.

2.72 - The more likely types of dam that would be built on the site, regardless of available materials are:-

(a) Rockfill with upstream impervious concrete membrane.

(b) Rockfill with either central or upstream clay core.

(c) Rolled earth fill with zones graded from inner impervious to outer pervious.

(d) Concrete slab and buttress.

2.73 - The last type, the concrete slab and buttress, requires little in the way of local materials. Concrete probably would be obtained from existing local resources and even if a plant were established on the site, certainly the sand and probably the crushed rock would be brought in from established quarries.

2.80 - Exploration for Rock Fill Materials :

2.81 - For a dam of rock fill, the two suitable types of rock available in the area are dolerite and to a lesser extent, basalt. The dolerite occurs about three-quarters of a mile upstream from the dam and outcrops in large quantities about one-quarter of a mile further upstream. The nearer dolerite has been investigated by drilling and generally has too much overburden to be an economical quarrying proposition. However the upstream deposit is of dolerite with relatively little overburden. It has been checked seismically by the Bureau of Mineral Resources who reported that the seismic velocities were indicative of rock suitable for economic quarrying.

2.82 - The basalt, although closer to the dam than the dolerite being on the ridge immediately to the west, is in close proximity to the Hydro-Electric Commission's transmission line and large scale blasting for quarrying would not be possible without deviating the line. Also basalt in Tasmania is frequently vesicular and deeply weathered and therefore probably not suitable. It is intended to drill one exploratory hole in it but at this stage it is not expected that it will be a consideration.

2.90 - Exploration for Clay Core Materials :

2.91 - A clay core dam could be cheaper than a concrete faced rock fill if suitable clay was available close to the site. The quantity required for a clay core would be about 60,000 yards and because of the wastage and unsuitable pockets which usually occur, it is as well to prove double this quantity if possible.

Three likely areas have been investigated for suitable clay and they are shown on Drawing Number Six "Valley Investigations", which shows the test pits together with their depths. The first area was in the Tertiary sandstone near the house on what was formerly Beard's property, the second in the dolerite near the proposed dolerite quarry, and the third on the gentle dolerite slopes west of the creek and quarry sites, and at a somewhat higher elevation.

2.92 - The classification tests necessary to prove the suitability or unsuitability of the clay were carried out by the Hydro-Electric Commission's laboratories.

<u>2.93</u> - The first area was unsuitable both because there is an insufficient quantity of clay and also the material is too sandy with not enough clay content. The second area near the dolerite quarry was unusual in that the clay was interspersed with travertine, a chalky deposit which would render it unsuitable. Also its in-situ moisture content was too low and the clay itself was highly plastic. In the third area there was insufficient clay to be considered.

<u>2.94</u> - This does not represent an exhaustive survey of all possible sources of clay and exploration will continue up to final design stage. However it is believed that an adequate source of suitable clay is unlikely within working distance of the site.

SECTION 3.00 THE TYPE OF DAM

3.10 - Possible Dam Types :

3.11 - The four possible dam types for this site were listed in sub-section 2.72 and they will be discussed in detail in this section. In addition a fifth type was briefly considered, namely a heterogeneous fill.

3.20 - Heterogeneous Fill :

3.21 - This is a dam of large cross section in which the available valley materials are dumped and compacted either at random or in zones.

The main reasons for considering a heterogeneous fill are, that there are large quantities of river wash material available in the creek bed which could be collected and used without difficulty and also, the wide spread base of the dam and the flat side slopes suit the abutment conditions.

3.22 - An examination of the river wash material in the creek bed has shown that it contains relatively large quantities of organic material, including large logs and tree roots, down to a considerable depth. Although a high quality material is not required, it is considered that large quantities of organic material occurring at random in the deposits preclude its use.

3.23 - Large quantities of any other type of material are not available cheaply, and as this type of dam would contain more than six hundred thousand (600,000) cubic yards of fill against two hundred and fifty thousand (250,000) cubic yards in a rock fill, it is obviously not an economic proposition.

3.30 - Concrete Slab and Buttress :

3.31 - A preliminary design for a concrete slab and buttress dam has been prepared and is shown on Drawing Number Seven "Preliminary Buttress Sections". The buttress sections are of the tee head type and are two feet thick and spaced at eighteen foot intervals. The inclined upstream slab has a minimum thickness of two feet. Footings are taken between four and ten feet into the foundation rock, and are generally four feet in width along the longitudinal axis of the dam. Rock stresses in the foundations have been approximately checked and the footing dimensions are satisfactory. The concrete stresses are limited to 3,000 pounds per square in compression and the allowable tensile stress at the face is limited to 200 pounds per square inch. The tee head of the buttress is quite heavily reinforced and the allowable steel stress is twenty thousand (20,000) pounds per square inch.

3.32 - An overflow section seventy-two feet (72') wide is included in the structure.

3.33 - The total amount of concrete in a dam of eight hundred million (800,000,000) gallons capacity with a top water at R.L.177, would be about twenty-five thousand (25,000) cubic yards including footings, buttresses, slab and overflow section. A major contractor in the concrete construction field stated that the average rate for concrete placement, including formwork and reinforcing steel, would be not less than sixty (60) pounds per cubic yard. It is considered that with detailed pricing and competitive tendering this figure could be reduced to say forty-five pounds (£45.) per cubic yard. Even at this latter price, the cost of the dam alone would be £1,125,000. 0. 0 which is considerably more than the rock fill alternatives.

3.34 - Some economies could perhaps be made by detailed design consideration, viz a study of the most economic buttress spacing suitable to the site, and also comparisons could be made with other traditional slab and buttress types such as Ambursen and massive buttress heads, however it is doubtful whether they would yield any saving in materials sufficient to make it competitive with the rock fill. There is also the considerable risk in estimating a slab and buttress dam when it is to be carried out by contract as the number of likely tenderers is limited and without competition prices could vary unpredictably. The uncertainty in unit rates is already apparent from preliminary talks held with prospective tenderers.

3.40 - Rockfill :

3.41 - A rock fill dam is the term sometimes used to cover various types of dams which have large dumped rock zones as the supporting shell for various water holding or impermeable membranes. The variety of types comes from the many impervious membranes used. They can be concrete, asphalt, steel, or clay, depending upon the availability and cost of these various materials.

3.42 - The types of rockfill dam investigated and considered in detail for this report are those which have an upstream face of reinforced concrete, or a core of clay. A steel membrane even if well protected with one of the methods now available would require some, even though infrequent, maintenance which would necessitate emptying the dam. Asphalt faces are used in other countries but have not been used for a major dam in Australia. It is considered that there is not enough knowledge of the technology available to us in this country to recommend its use in dam construction.

3.43 - A rock fill dam with a clay core would be the cheapest type which could be built on this site, provided satisfactory clay were available. An inclined clay core near the upstream face would contain about 60,000 cubic yards of clay and if this could be obtained in the valley, the saving by use of a clay core would be about £50,000. However as explained in paragraph 2.94, suitable clay deposits appear unlikely.

3.44 - The reinforced concrete face rockfill dam is left as the most suitable alternative. This is considered in detail in the following Section Four.

SECTION 4.00 ROCKFILL DAM WITH CONCRETE FACE

4.10 - Stability Analyses :

4.11 - Preliminary stability analyses were prepared in July 1964, and these showed that the stability of the embankment slopes in conjunction with the sliding of the foundation rock along its bedding planes was the critical feature of the design. Since then more accurate information has been obtained regarding rock densities, and re-appraisal using more favourably situated stability berms has been possible. The shear value originally adopted for the clay in the bedding planes has been retained, but as discussed in Section Two, further investigations now under way may permit less conservative parameters to be used and possibly permit the cross section used for this report and estimates to be reduced.

4.12 - In a stability analysis the weight or density of the rock used as rockfill in the dam, and as rock in the foundations, has a large bearing on the result. The densities used for the calculations are as follows:-

Dolerite rock in situ	182 lb/per cubic foot.
Dolerite rock dumped as rockfill in dam	115 " " " "
Mudstone in situ	130 " " " "
Mudstone dumped into stability berms.	90 " " " "

The mudstone densities were derived from tests on drill cores made by the Bureau of Mineral Resources and the dolerite densities were given by the Hydro-Electric Commission of Tasmania.

4.13 - The other important criterion for stability analysis is the measure of shear strength or the angle of internal friction of the materials comprising the cross section. The values used were as follows:-

Clay in mudstone bedding planes	20.3°
Dolerite dumped in dam	45°

4.14 - The figure for the clay was derived from field tests made on a bedding plane exposed in the foundation of the dam by loading and sliding a prepared block of rock above it. The friction angle for dolerite in the embankment is a commonly accepted one for clean rock well placed.

4.15 - The shapes of the slip planes used in the analyses were taken as being circular in the embankment material, and in straight lines parallel to the known bedding planes in the foundation rock. The "standard method of slices" was used for calculating the stability, and this will be checked by alternative methods for the final design.

4.20 - Stability Berms :

4.21 - Using the above described method of analysis it was found that with a downstream rockfill slope of 1.4;1, the required minimum factor of safety of 1.5 was not reached in the abutment ends of the dam, because the rock beds are there not restrained downstream.

4.22 - To counteract this effect, stability berms of siltstone, obtained mainly from the required spillway excavations, are proposed to be placed on the downstream face. These berms are sized so as to maintain the minimum factor of safety of 1.5;1, and contain 30,000 cubic yards of rock. They are shown in plan on Drawing Number Eight "Proposed Rockfill". The crest of the western berm at reduced level 130.0 is fifty feet wide, and the crest of the eastern berm at reduced level 155.0 is thirty feet wide.

4.30 - Compaction of Rockfill :

4.31 - It is important that the rockfill be built with as high a density as reasonably can be obtained for several reasons. High density increases the friction angle of the rockfill and also an increase in weight per cubic foot decreases the required size of the stability berms. The better that the rockfill is consolidated and settled into place, the less likely the bank will be to settle within itself to any marked extent when the water load comes on it, as if it does, it may cause the concrete face to crack and leak and inconvenient repairs will be necessary. Just as the large mass of rock is the principle inherent strength of this type of dam so the possible settlement and cracking of the slab is the inherent weakness.

<u>4.32</u> - To reduce subsequent settlement of the rockfill as much as possible it is proposed to sluice the fill into place as it is tipped by jetting it with relatively large volumes of water, as much as one hundred gallons per cubic yard of rock. This is the usual method of improving rockfill compaction. It tends to lubricate the rocks into place as they slide down the tipping face from the trucks. The contractor will have to build a small collection dam downstream of the main dam and sluicing water will be recirculated from it. Allowance has been made for sluicing the rockfill in the estimates.

<u>4.40 - Proposed Dam Cross Section</u> :

<u>4.41</u> - The slope of the upstream face of the Dam will be 1.5:1, which is flatter than the usually selected 1.33:1 for dolerite embankments, so as to achieve the required factor of safety using the present adopted figures for the strength of the clay seams in the mudstone foundations.

<u>4.42</u> - The crest wisth has been taken as fifteen feet and its elevation is at a reduced level of 187 feet or ten feet above the full supply or spillway level. The fifteen feet width is chosen as adequate for a maintenance vehicle. As no special provision is made in the design for earthquake loads

a fairly wide crest is desirable.

4.43 - The slope of the downstream face of the dam is 1.4:1 with berms as already described, however this may be varied during final design.

4.44 - The crest of the Dam will be cambered so as to rise by one foot from the intersection of the crest and the abutment along the longitudinal axis to the middle or deepest section. Generally the geometry of the embankment proposed may be seen on Drawing Number Eight.

4.50 - Stability to Sliding :

4.51 - As with most earth or rockfill dams there is not much likelihood of failure by wholesale sliding because of the relatively wide base and enormous mass. Because of the horizontal bedding of the foundation rocks and the possible clay filled bedding planes, the factors of safety against sliding has been checked. For a factor of safety of 2.5 which is very conservative, the angle of internal friction for the clay would have to be as low as twelve degrees (12°), thus the proposed section is considered to be quite safe against wholesale sliding.

4.60 - Freeboard :

4.61 - The freeboard is the minimum height from full supply or spillway crest level (which in our case is at a reduced level of 177) to the top of the dam. The dam proposed has a freeboard of ten feet or eleven feet at the section of maximum camber.

4.62 - The freeboard is designed such that water cannot at any time overtop the dam. The critical case is for a maximum flood occurring when the dam is full, and coinciding with maximum height waves on the surface of the dam.

4.63 - For a spillway seventy feet wide, and a maximum predicted flood of 1,200 cusecs, the greatest rise in the ponded level would be three feet above spillway crest level. The maximum wave height for a ninety miles per hour wind would be three feet, and to this must be added a factor of fifty per centum to allow for run up the sloping concrete surface. To this is added a factor of safety of two feet six inches - thus:

Flood rise	3'0"
Waves	3'0"
Wave run-up	1'6"
Safety allowance	2'6"
Total	10'0"

4.70 - Concrete Upstream Face :

4.71 - The dam will be faced with reinforced concrete slabs, which will be articulated with a jointing system to allow as far as possible for slight settlement in the rock fill. The thickness of the slab and the spacing of joints are a matter for a final design. For the purpose of preparing estimates the slab has been taken as fourteen inches (14") thick at the base and eight inches (8") thick at the crest. The slabs will be jointed about every forty feet along the longitudinal direction of the face and every eighteen feet up the slope.

4.72 - The face will be of poured concrete, about 2,800 cubic yards in all, reinforced with welded A.R.C. mesh. The water face could be poured against moving steel slip forms according to the contractors choice. The rear face will have to be poured against the dam which must be carefully prepared. There are two alternatives considered, either to provide a zone of compacted crushed rock, or else to hand-place a zone of large stones with the crevices between them chinked carefully to avoid concrete loss during pouring. The main aim with this slab support zone is to prevent any local settlement or displacement of the slab under load causing it to crack and consequently leak. After discussing the difficulties of each method with various contractors it is considered that a zone of hand packed and chinked large stone would give a satisfactory result with the least difficulty of construction.

4.80 - Sealing the Foundations :

4.81 - From water tests to date the permeabilities near the top of the hole have been found to be between two hundred and six hundred feet per year but below about fifty feet depth, permeabilities have generally been less than ten feet per year.

Approximately a permeability of five hundred feet per year around a ten feet deep concrete cut-off wall taken the full length of the dam and to a depth of fifty feet would permit a leakage of about one and one half cusecs (1.5) cusecs) beneath the dam. This approximate figure indicates that grouting the foundation to a minimum depth of fifty feet will be economically necessary, and provision has been made in the estimates for a grout curtain to be taken to a depth of seventy five feet.

4.82 - A cut-off wall will be necessary to connect the concrete membrane into the foundation rock. This wall will be of reinforced concrete constructed to an average depth of ten feet and a width of two feet six inches. This wall will also serve as the grout cap through which the grout holes will be drilled.

4.90 - Outlet Works :

4.91 - Water will enter and leave the dam by means of a thirty inch diameter mild steel pipe laid beneath the dam, in a concrete horseshoe shaped tunnel so as to permit access to the pipe up to the concrete face of the dam.

4.92 - The regulation of the water to and from the dam will be controlled from a valve tower built vertically from the toe of the dam to crest level. The valve tower will be a cylindrical shell of reinforced concrete enclosing a thirty inch diameter vertical outlet pipe with valved offtakes at four levels.

The top level off-take would have a floating draw-off arm so that the water would be taken from just below the surface where quality is usually better. The purpose of the tower is to simplify the operation of the valves and to enclose them in a dry well for any repair or maintenance that may be necessary.

4.93 - Water inlet to the dam would be by the same system of pipework and valves. Access to the tower would be a bridge from the crest of the dam.

4.94 - Spillway :

4.95 - The spillway has been designed to take a maximum flood of 1,200 cusecs which would occur if a maximum probable precipitation occurred on the catchment area at a time when the dam was full. The catchment area is 5½ square miles and the shortest duration of any storm to cause run-off from the whole catchment would be one hundred minutes. The maximum probable precipitation on the catchment is 0.80 inches per hour, from the Hobart rainfall curves from "Australian Rainfall and Run-Off" produced by the Institution of Engineers, Australia with an average run-off coefficient of 0.8. The ponding effect of the dam reduces this maximum flood to the 1,200 cusecs used for design.

4.96 - The spillway will be constructed as a chute on the eastern abutment with a concrete sill seventy feet wide and crest level of one hundred and seventy-seven feet. The channel of the spillway will be directed towards the creek downstream and well clear of the downstream toe of the dam. It will be concreted as necessary to reduce erosion. The approximate configuration of the spillway is shown on Drawing Number Eight.

SECTION 5.00 WATER QUALITY

5.10 - Hardness of Risdon Brook Water :

5.11 - Water samples taken from Risdon Brook have given,when analysed,medium to high "hardness" results. Hardness in public water supplies, its effects, and comparative typical hardness figures for a number of Australian towns and cities are set out in Appendix 'D'.

5.12 - Diversion of at least the low creek flows around the reservoir is therefore proposed to prevent stored soft Derwent water becoming harder.

5.13 - It is necessary to consider hardness of the creek water in conjunction with the stream flows, as the hardness appears to vary immensely with the flows; also this permits the increase in hardness of stored water to be assessed, and the effect of various diversions to be determined.

5.20 - Creek Flow :

5.21 - The weir and automatic flow recorder were set up in October, 1961, and flow readings are almost continuous to September, 1964.

5.22 - Only on the following occasions did the average daily flow exceed 5 cusecs or 2.7 million gallons per day:-

25th July,	1962	34	cusecs average
26th July,	1962	10.5	"
22nd August	1962	6.3	"
29th Sept.	1962	11.6	"
4th June,	1962	11.3	"
July,	1962	13.0	"
5th June,	1964	12.6	"

5.23 - Also in February 1964, when the recorder was out of action, the flow by one eye witness account probably exceeded 5 cusecs for several days.

5.24 - The rainfall at Lindisfarne for the period since the recorder was installed has been steadily below average except for 1964 -

1961	-	15.3"
1962	-	15.7"
1963	-	14.2"
1964	-	22.99"

The thirty year average at Lindisfarne is 23.4". Flow readings are not available for the storms in February, 1964.

5.25 - It must be stressed that three of the four years for which records are available have been dry ones, and it has been noted that the percentage run-off from Tasmanian catchments appears to diminish rapidly as the annual rainfall in a dry year drops to 15 inches or below. In 1962, Risdon Brook flowed 90 million gallons with a rainfall of 15.7" (at Lindisfarne), giving a run-off percentage of about 8%, and in 1963 it flowed only 16.5 million gallons with 14.2" rainfall (at Lindisfarne), this giving a run-off percentage of only 1½%. It is estimated that the run-off percentage could approach 15% and the flow 330 million gallons in a wet year of say thirty inches (30") rainfall.

5.30 - Degree of Hardness of Creek Water :

5.31 - Regular sampling of the creek water quality was carried out for the early part of 1964, and the relevant results are shown below:-

Date	Hardness ($CaCO_3$ in mgl.)	N.Cl in mgl.	Flow Cusecs
14 Feb. 64	176	184	Thought to be between 2 and 10 cusecs
18 Feb. 64	212	212	Not Recorded
3 March 64	272	280	Not Recorded
10 March 64	300	280	Not Recorded (light)
23 March 64	400	376	Nil
24 March 64	400	320	Nil

Date	Hardness ($CaCO_3$ in mgl.)	NCl in mgl.	Flow Cusecs
29 April 64	600	660	0.02
13 May 64	340	440	0.002
15 May 64	500	588	Nil
19 May 64	660	625	0.03
25 May 64	480	640	0.01
1 June 64	160	192	7.9
16 June 64	220	244	0.16
30 June 64	372	425	0.04
14 July 64	210	250	0.9 (0.19)
28 July 64	232	260	0.11
11 Augt. 64	204	226	0.58

<u>5.32</u> - Nil flows can be eliminated from the consideration, and also perhaps flows below 0.1 cusec (0.054 MG per day). There are therefore only five significant results, four of which are below 1 cusec (or ½ M.G.D.).

<u>5.33</u> - These indicate that for significant flows, 200 mgl. of hardness may be taken, and the hardness decreases in the higher flows due to the stormwater dilution. Any exact relationship, if it exists, of hardness and storm flows cannot be established from these results. However the hardness concentration would be likely to be less in normal or wet years due to the greater dilution of storm water. It is also possible that the degree of hardness at similar flow rates would decrease over the period of a spell of wet weather and high creek flows, perhaps over the period of a long storm.

<u>5.34</u> - Water samples taken from the creek at the dam site and at the top end of the proposed reservoir basin have not shown any significant difference in the degree of hardness, indicating that the hardness comes from above the reservoir basin. Water samples taken on various branches of the main creek upstream of the reservoir basin also have not shown significant difference from each other. Test holes excavated for clay

exploration have shown that travertine, a calcium carbonate, is present in the dolerite soils which occur over most of the area in the northern part of the catchment, and this travertine is almost certainly the cause of the hard water yielded by the catchment.

5.40 - Diversionary Pipeline Around Reservoir :

5.41 - A diversionary pipeline to carry the low flows around the dam is possible. Because of the succession of dry years, there have not been many days since recording was commenced in October 1961, when the average daily flow has exceeded 5 cusecs. If a 5 cusec capacity diversion had been in operation over say three years until October 1964, the excess flow entering the reservoir would have been the relatively small total of about fifty or sixty million gallons.

5.42 - At first a bypass pipeline of 5 cusec capacity was considered, however because of the travertine deposits, the water running off the catchment continues to be hard right through the winter and spring, thus there is a case for increasing the diversionary arrangements so as to bypass all but flood flows around the reservoir.

5.43 - For the diversion of the creek flow, various combinations of concrete-lined races and concrete pipelines have been investigated and estimates made. One such diversion scheme is indicated on the accompanying Drawing Number Nine, "Stream Diversion". This would bypass about fifteen cusecs around the dam and would cost about £26,000. 0. 0. Run-off from the western slopes above the reservoir could also be cut off by an earthen ditch for a further £2,500. 0. 0. A total amount of £30,000. 0. 0 has therefore been included in the estimates for creek diversion.

5.50 - Acquisition of Catchment :

5.51 - Although a fifteen cusec diversion would prevent all but the larger storms entering the dam, it will still be necessary to control the catchment area. For a water supply reservoir it is essential that no possibility of pollution from human or animal sources be permitted in the catchment. For this reason the Board has purchased, or is in the process of purchasing the land shown on Drawing Number Ten "Catchment and Storage Areas". The costs of this acquisition £37,000. 0. 0 have been included in the estimates. No decision has been made as yet as to whether the balance of the catchment should be purchased, thus its estimated cost has not been included.

SECTION 6.00 - TRUNK MAINS & PUMPING STATION.

6.10 - Population :

6.11 - The population of Clarence has been increasing rapidly and at a faster rate than the Hobart Metropolitan area as a whole. The increase predicted in the recently completed transportation study for the next twenty years is from 25,000 persons now to 77,000 persons in 1985, which represents an annually compounded growth rate of six per centum per annum (6% p.a.). As this is the most recently completed study of the Hobart population trends, it has been accepted and used for the derivation of future populations in this Report.

6.12 - The distribution of the future population in each suburb was estimated in the Consulting Engineers' Twenty-Fourth Report on Clarence Water Supply in 1960. These figures have now been correlated with the transportation study and there is a negligible difference between them with one exception which is that the recent study has estimated a larger future population in the Howrah and Rokeby areas. It is considered prudent that the larger population, as determined by the transportation study, should be used for the purpose of this report so as to ensure that the future water requirements are not under-estimated.

6.13 - The Metropolitan Water Board have designed the Sorell scheme for 11,300 persons beyond the present Clarence Water area and as these will draw-off the Clarence System, this figure has been used for this report.

6.20 - Water Consumption :

6.21 - Clarence continues to grow as a predominantly dormitory suburb of Hobart. There has been very little industrial development up to the present and so the water consumption continues as typical of a domestic community. The average summer demand is one hundred and sixty (160) gallons per person per day and in winter it is about forty (40) gallons per person per day. It is anticipated that these figures will increase over the next twenty years or so to a summer and winter average of one hundred and eighty (180) gallons and fifty (50) gallons respectively, per head of population per day.

6.22 - If the supply remains unmetered, the peak daily requirements will probably reach two hundred and seventy (270) gallons per head per day in the next twenty years. Water consumed per head of population is increasing everywhere with the generally improving standards of living and the water requirements allowed in this report allow for this trend.

6.30 - Available Water and Future Demand :

6.31 - The bulk supplies of water at present available to Clarence are -

Southern Regional Scheme -	3.8 million gallons per day
Derwent Supply -	3.0 million gallons per day

If a dry summer were taken to be of one hundred and fifty days duration, then the water available from Risdon Brook Dam would be 5.4 million gallons per day. Evidence to date suggests that a summer period sufficiently dry and hot to maintain an average water requirements of one hundred and eighty (180) gallons per head per day would be a considerably drier summer than usual and is therefore a suitably conservative figure to use for design purposes.

6.32 - In addition, Flagstaff Gully Dam can be used to augment the supply on the very hot days when the peak figure of two hundred and seventy (270) gallons per head per day would be reached. On such days, assuming that no more than six or seven occurred in succession, Flagstaff Gully could add six million gallons per day to the system.

The maximum available peak daily supply in millions of gallons per day becomes therefore :

Southern Regional & Derwent -	6.8
Proposed from Risdon Brook Dam -	5.4
Flagstaff Gully Dam -	6.0
Total -	18.2

This would be sufficient water for sixty-seven thousand (67,CCO) persons. It is interesting to note that by interpolating the population figures previously outlined, this population will be reached in 1981. A population of 67,000 persons can also be served if the average summer value of one hundred and eighty (180) gallons per head per day is used in the calculations and the Flagstaff Gully Dam contribution to make up the peak daily demand is omitted. by dis-regarding peak days.

6.40 - Filling Risdon Brook Dam :

6.41 - During the winter period, there will be an excess of supply over demand from the Southern Regional and Derwent Water Supply Schemes, and this surplus will be used to replenish Risdon Brook Dam. To assess how much water will be available depends again upon the growth in population.

6.42 - By 1981, when it is estimated that a new source of supply will be required, the population according to the transportation study figures, will be about sixty-seven thousand (67,000) and a population of this size would require approximately three and one-third million gallons per day on an average winter day. This would leave surplus from the bulk supplies about three and one-half million gallons per day. Such a surplus would fill the dam in about two hundred and thirty (230) days, which represents a normal winter period in a drier than average year.

6.43 These calculations for both filling and emptying are made using estimates of population and water consumption both of which are impossible to predict accurately. If there has been an over-estimation, then it means that it will take a little longer before a new source is required and the reverse is true if an under-estimation has been made.

6.50 - Supply from Risdon Brook Dam :

6.51 - It has been determined that the average daily draught from Risdon Brook Dam over an extended summer period is less than six million gallons per day. It is proposed to distribute this water into the system initially by pumping it to the existing Risdon Vale Reservoir whence it will gravitate to the Flagstaff Gully trunk main, as shown on Drawing No. 11, "Distribution Trunk Main". In order that the Risdon Brook Supply will not have to meet daily peaks, particularly in the early years, a regulating valve will be installed in the main and when demand is more than Risdon Brook can supply, Flagstaff Gully Dam will augment the supply. All the water cannot be accepted by the present system and so, in the reasonably near future, the Risdon Brook Gravity Main will have to be extended to Bellerive Reservoir along the suggested route shown on the drawing. As the life of the gravity supply main will extend well after the time when a new source is required and as the new source will be from the north somewhere (probably across the proposed Risdon Bridge), it is an economical proposition to make the main large enough to carry another five (5) million gallons per day. The new source would supply to a new reservoir adjacent to Risdon Vale Reservoir and probably at a slightly higher elevation to increase the carrying capacity of the main. A twenty-seven inch (27") diameter main would ultimately carry eleven million gallons per day by this means.

6.52 - The pumping station would have an installed horse power of about 700, allowing for a standby unit, and three operating units at maximum demand. The pumps would be fitted with variable speed motors to enable them to meet efficiently the large variation in head caused by the variation in the height of the dam over the pumping season.

6.53 - The economic rising main size would be 24" diameter allowing for the fact that when a new source is provided the Risdon Brook supply may have to meet peak loads with consequently increased pumping rates.

SECTION 7.00 ESTIMATED COST

7.10 - Basis for Estimates :

7.11 - The estimates are based upon ruling contract rates at the date of this Report. They include all the work necessary for the construction of a concrete faced rock fill dam on the site together with the pumping station and distribution mains.

7.20 - Analysis :

7.21 - An analysis of the estimated cost is as follows:

Preliminaries which include the purchase and survey of the land, and the investigations to the stage of final design.	£ 75,000. 0. 0
Contractors establishment costs, the clearing of the storage basin and the stripping of the site.	27,000. 0. 0
Grouting of dam foundations and the north eastern saddle.	85,000. 0. 0
Construction and excavation of cut-off wall, and construction of concrete face slab.	140,000. 0. 0
Construction of rock-fill including hand placed zone and stability berms.	390,000. 0. 0
Outlet works comprising control tower and access bridge, valves, pipework and outlet pipe beneath dam.	33,000. 0. 0
Construction of spillway.	12,000. 0. 0
Creek diversion.	33,000. 0. 0
Site improvement including caretaker's house and fencing.	10,000. 0. 0
Distribution works, comprising pumping station, rising and gravity mains.	195,000. 0. 0
Total	£1,000,000. 0. 0

Note that engineering costs such as fees, and the cost of site engineer and clerks of works are included in the various items.

SECTION 8.00 RECOMMENDATION

8.01 - It is recommended that approval be given to the construction of a rock fill dam to store 800 million gallons of water at an estimated cost of One million pounds including all ancillary and distribution works.

8.02 - If the dam is approved it is recommended that final designs be prepared and tenders invited so that construction can commence in January, 1966. The dam would be completed for filling with water by March, 1968 and the distribution works finished by September, 1968 so that the scheme as a whole can be effective for the summer of 1968/69.

DATE :

This Report is dated the Seventh Day of April in the year One Thousand Nine Hundred and Sixty-Five.

GUTTERIDGE, HASKINS & DAVEY

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METROPOLITAN WATER BOARD.

RISDON BROOK DAM.

EIGHTH REPORT.

APPENDIX A.

GEOLOGICAL REPORT ON THE RISDON BROOK DAMSITE

BY

I.B. JENNINGS, ~~SENIOR~~ CHIEF GEOLOGIST.

The regional geology of the area surrounding Risdon Brook ~~has~~ was been discussed ~~previously~~ in reports ~~submitted~~ by Spry (1960), Groves (1963), and Moore (1964). Rock outcrop is poor and similarity of lithological sequences restricts the accuracy of geological mapping considerably, however the rock distribution and structure have been established reasonably well on a regional scale.

~~As noted in the earlier reports~~, the damsite is situated on a tilted block of Permian sediments close to a major fault. The storage area is under-lain by Permian siltstone, Triassic sandstone and shale and Jurassic dolerite. The boundary between the Permian and Triassic sediments is a major fault whilst the boundary between the Triassic rocks and the dolerite is difficult to interpret, being in part intrusive and in part probably faulted.

Since few major structures in Tasmania have been founded on siltstone of the Permian Ferntree Formation, no great body of precise knowledge was available concerning the engineering properties of these rocks. Also the siting of a major dam on gently dipping sediments presents a series of unique problems demanding special investigations. As a result of the preliminary studies, a number of features emerged and an investigation of these aspects has been made to determine the feasibility of the site. The investigations were designed to provide information on the following items:-

(1) The condition of the bedrock in the vicinity of the dam, the depth of weathering and the permeability of the bedrock.

(2) The possible presence of a buried stream channel in the bed of Risdon Brook.

(3) The position and condition of the rocks adjacent to the Lindisfarne Fault at the damsite.

(4) The position and nature of other smaller scale faults in the vicinity of the dam.

(5) The presence and persistance of thin "clay" seams along the bedding planes in the mudstone and the likelihood of such seams promoting sliding.

(6) The likelihood of leakage occurring from the reservoir through the saddle to the Grasstree Hill road.

(7) Determine the physical properties of the rocks at the damsite.

(8) Establish sources of material for construction of the dam.

Investigation programme:

To study these questions the following investigations have been made.

(1) A geological reconnaissance indicated that no insurmountable problems appeared to be present and a feasibility study was therefore justified.

(2) A regional geological map of the district surrounding the damsite was prepared.

(3) A detailed map indicating all rock exposures in the vicinity of the dam was also prepared.

(4) A series of trenches and excavations were made in order to examine the rock at the damsite, and to determine the position and effect of faults in the vicinity.

(5) A preliminary diamond drilling programme, sufficient to assess the feasibility of the project, was carried out.

(6) Seismic refraction and resistivity surveys were made to determine rock properties, check geological conclusions, to guide further drilling and to outline areas for investigation of sources of materials for construction.

(7) Numerous smaller studies have been made on such items as the petrology of the rock types, detailed stratigraphy, mineralogy of the "clay" seams, the relation of jointing to the regional structure etc. However these are studies of detail and do not individually affect the feasibility of the dam.

(8) and

The purpose of this report is to collect and present the information disclosed by the investigations carried out up to date. It is considered that from a geological viewpoint sufficient information is available to assess the feasibility of the proposal although further more detailed studies will be necessary for design purposes.

Acknowledgments:

In the preparation of this report the writer has drawn heavily upon the information available from the earlier work of Spry and Groves. Most of the detailed mapping at the damsite was darried out by W.R.Moore, assisted for a time by W.L.Mathews. The logging of bore cores, together with other detailed studies ~~still in progress were carried out~~ by W.R. Moore, M.J.Longman, W.L. Mathews and the author.

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Investigation program;

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8. A detailed drilling programme to delineate the faults and test the permeability of the rocks underlying the dam and the saddle area.

Results ~~to date~~:

(1) The drilling at the damsite indicates that the rock generally is sound, competent and impermeable. However weathering, particularly along the joints and bedding planes, has proceeded to depths of up to 50 feet or so in the abutments. Below creek level most of the joints encountered ~~so far~~ are filled with a mixture of clay, pyrite and calcite.

(2) The rock itself is relatively impermeable so that any leakage which does occur will take place through the joints, broken zones and along bedding planes. Grouting will be necessary to reduce the leakage to acceptable limits.

(3) In the vicinity of the dam the general dip is to the SW resulting in a component across the dam as well as downstream. Clay filled seams along the bedding plane and some of the joints are present and they vary in thickness from mere films up to two inches (2"). They appear to be persistent over distances at least comparable with the base width of the dam.

(4) No buried stream channel at the damsite has been shown by the investigations. ~~up to the present~~.

(5) The position of the ~~Lindisfarne~~ Main Fault is fixed reasonably well and information is available as to its nature on the eastern abutment. Further studies of fault at the damsite and in the saddle area are proceeding.

(6) Minor faulting has been demonstrated downstream from the dam. However, the presence of such faults is not expected to present serious difficulties with the dam in the present position.

7. (7) Some grouting may be necessary in the saddle between Grasstree Hill and the reservoir. Preliminary results of current investigations in this area indicate that the leakage can be kept within acceptable limits by grouting. ~~Further testing in this area is in progress.~~

(8) Dolerite suitable for rock fill ap~~pears to be~~ is available ~~along~~ on the west side of Risdon Brook 1 to $1\frac{1}{2}$ miles north of the damsite. Adequate reserves ~~appear~~ to be present but further drilling is planned to prove ~~this~~.

(9) Preliminary Geophysical results indicate that about 400,000 cubic yards of clay are present on the eastern side of Risdon Brook about 1 mile north of the damsite. However the suitability of this material for construction purposes has not been established. ~~and further testing would be necessary to prove the reserves.~~

(10) Over the abutment areas many large blocks have rotated around joint planes and will require removal before construction.

Recommendations:

No

For the purpose of this report the following recommendations apply to the siting of a dam up to about 100 feet high in the position indicated on the accompanying plan. Shifting the axis of the dam up or downstream by up to 100 feet would not materially alter the conclusions reached. However movements in excess of 200 feet would require further considerations and movements in excess of 400 feet would involve additional investigations. The recommendations apply to a gravity type structure and additional investigation would be necessary in order to assess the feasibility of alternative

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Results;

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7. Grouting may be necessary in the saddle between Grasstree Hell and the damsite. Calculations indicate a yearly loss of 8 million gallons may be expected.

The evidence of the preliminary geological investigations indicate that it is feasible to construct safely a dam of the size and type indicated on the site selected provided that the designing engineers are able to satisfy the following conditions within the economic limits set by the building authorities.

(1) Provide sufficient loading or other features in the design to prevent sliding along the clay seams between the bedding planes. The preliminary tests indicate that this is quite feasible and can be satisfied by design considerations.

(2) In conjunction with (1) construct an effective grout curtain along the upstream face of the dam together with adequate relief drain behind this in order to eliminate or substantially reduce uplift pressures, and to restrict or eliminate leakage.

(3) Some provision should be made for grouting or sealing the Grasstree Hill Road saddle.

Geology

(1) Stratigraphy

The bedrock at most places is obscured by a discontinuous and variable layer of soil. Except for the valley flats and river terraces, the soils are related to the parent rocks but considerable down slope drift of soil is evident in some areas. Over the areas of Permian rocks, which are of immediate interest to the dam the soil seldom exceeds a thickness of three feet and is generally much less than this.

Pleistocene (?) Talus

Drilling, test pitting and geophysical surveys in the dolerite areas upstream of the dam have shown the presence of a surface layer up to 25 feet thick composed of dolerite boulders set in matrix of dolerite derived clay and sand and containing numerous blocks and possibly veins of travertine. It appears to be underlain by weathered and jointed dolerite and finally by fresh dolerite. The probable sequence is given below.

	Unit	Thickness
(1)	Recent soil	a few inches up to a foot or so
(2)	Pleistocene (?) talus	0- 25 ft +
(3)	Weathered dolerite	0- 25 ft +
(4)	Fresh dolerite	unknown but probably many hundred feet.

Unit (2) above is thought to be a "fossil" talus accumulation of probable Pleistocene age related to the pluvial stage of the glaciation. However direct evidence for its age is lacking.

A recent exposure of broken and angular Permian siltstone fragments set in a matrix of clay and silt derived from the weathering of Permian rocks has been located on the western abutment of the damsite. Three alternative derivations are suggested for this material.

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Notes on Geology:

Cygnet Coal Measures.

Overlying and transitional into the Ferntree formation is a brown sandstone with carbonaceous fragments, ~~and~~ some shaley ~~bands~~ horizons, and a grit band up to 18" thick. This may correspond to the Cygnet Coal Measures which occur elsewhere in the Hobart area, but direct evidence is lacking. This unit is up to 30 feet thick in a creek north of Risdon Brook and in a Failing drill hole in the saddle area.

Another exposure of talus composed of broken and angular Permian siltstone fragments in a matrix of clay and silt occurs on the western abutment of ~~this~~ the dam. The maximum depth of this deposit is 20 feet.

(1) It is a thin relict veneer of Pleistocene talus.

(2) It is infilled buried river course separated by an island from the existing stream.

(3) It is a fault breccia.

Investigations are at present proceeding on this deposit and the author considers that of the alternatives suggested (1) seems to be the most probable for the following reasons:

(1) The deposit resembles similar talus deposits examined elsewhere in the Bellerive and Lindisfarne district.

(2) It lacks the shearing and other structural features to be expected in a fault zone.

(3) If it is a fault its trend is completely different from the known faults in the district.

(4) There does not seem to be enough room to fit either a major fault or a buried stream between the known outcrops and excavations.

(5) Preliminary geophysical results indicate sound rock in this area.

(6) Diamond drilling, surface exposures and a shaft nearby all indicate sound rock in the area.

TRIASSIC SYSTEM :

The Triassic sediments outcrop immediately upstream from the proposed damsite and are not in contact with the dam in its present position. They consist of thick units of medium grained quartz and felspathic sandstone interbedded with mudstone and shale. The sandstone is massive, frequently cross bedded and sometimes friable. The mudstone and shale are thinly bedded and micaceous and weather readily to micaceous clay. They fret rapidly on exposure to air. The combination of massive, fairly competent sandstone units interbedded with weak shales leads to a predominence of sandstone in outcrop but the true proportion of shale to sandstone is probable much higher than is suggested by the outcrop.

The Triassic rocks exposed near the damsite have been correlated with the Springs Sandstone which is up to 600 feet thick in the type area but only about 250 feet can be demonstrated from the outcrops at Risdon Brook.

THE PERMIAN SYSTEM:

(a) Ferntree Formation:

This is the uppermost formation in the Permian sequence and is the unit upon which the dam will be founded. In the type area the formation is about 600 feet thick but only 320 feet have been demonstrated from out crop and drilling near the damsite. In the drill cores and outcrop the lithology is extremely uniform, the dominant rock type being a pebbly, muddy siltstone. The lack of contrasting rock types hindered investigation somewhat so that correlation of beds between drill holes could be achieved. A few thin sandy

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Notes on Triassic:

Triassic

The Triassic rocks exposed north-east of ~~west~~ the damsite form the following sequence

top	250'+	interbedded quartz feldspathic sandstone and shale
	~~60'~~	~~interbedded green sands~~
	30'	shale and mudstone
	30'	interbedded green sandstone and shale
	80'	poorly sorted feldspathic sandstone.
bottom		

Abundant outcrop of interbedded green sandstone and shale with limited drill hole information indicate the rocks in ~~indicating the sandstone in~~ the dam area are ~~are~~ close to the base of the Triassic.

Tertiary.

Hornblende tuff up to 40 feet thick occurs to the west of the damsite at 250 feet level ~~well above high water level~~. Overlying this tuff is a vesicular olivine basalt 50 feet thick.

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beds were noted as also were some thin limestone and muddy limestone bands. These are indicated on the accompanying sections. A possible correlation between two thin limestone bands is suggested, but the inference is that the remaining sandy and limey portions of the core are intersections of lenticular bodies. This is in accord with surface observations and no doubt accounts for the occasional lense shaped cavities observed in cliff sections. These may be noted in the cliffs on the eastern abutment of the dam, a few hundred feet downstream from the dam and in the quarry on East Risdon Road about $\frac{3}{4}$ mile south of the damsite. The cavities appear to be formed by the weathering out of limey concretions or sedimentary lenses. A small cavity of such type, about 9 inches long, was noted in the shaft near bore hole No.4. Since such cavities would occur only rarely below the water table and would not be connected with one another, they would not form major leakage paths.

The Ferntree Formation has strongly developed partings parallel to the bedding planes. The spacing between the major partings varies from about a foot up to several feet in natural exposures. At the top of the excavation on the western abutment there is a small exposure of thinly bedded mudstone with the partings only one inch or two apart. No similar unit has been observed elsewhere at the damsite.

In detail the rock consists of irregular inter-bedded laminae of silt and clay size material from about 1 mm up to a few mm thick which results in "curly" bedding in fine detail. Occasional pebbles and chips of slate, quartzite and other exotic rock types occur throughout the formation and distortion of the bedding beneath the pebbles suggests that they may have been dropped into the soft mud and silt. The sediment has been regarded as being of glacial origin but the exact mechanism of sedimentation is complex.

Whilst the rock itself is relatively impermeable, tough and competent, the horizontal partings and strong joint system divides it into a series of rectangular blocks which, at the worst, are bonded together with thin "clay" seams. These seams are quite persistent within the limits of the exposures available and vary in thickness from films up to a few inches thick along a single parting. Many of the joints above the water table are filled with similar material and a few with gypsum. Below the water table most of the joints are filled with a mixture of"clay"and pyrite.

Two samples of "clay" from seams along the bedding planes at the damsite and two from the quarry to the south have been forwarded to the University of Tasmania for mineralogical determination by x ray diffraction. The diffraction patterns indicate that the "clays" are composed mostly of finely divided quartz with some feldspar and mica but that no true clay minerals were present. A chemical analysis of one of the "clay" samples from the damsite is in accord with these findings. However, it may be noted that the ion exchange properties of the sample have not been determined and that if the particle size was less than about 10-5 mm, clay minerals if present may not be recorded. The evidence available, although not conclusive, strongly suggests that the material is finely divided rock flour. It could have many of the properties of clay.

Dips in the bedding planes in the Ferntree Formation are indicated on the accompanying plans and sections. These indicate a general inclination to the W and SW but some relatively large variations in strike and dip have been recorded. These variations are attributed partly to the effects of nearby faults, partly to the difficulty of obtaining accurate dips on rocks of this kind and partly to

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Notes on page (VI)

of the limey concretions. The greatest permeabilities in the water testing Z also occur at these horizons, suggesting possible interconnection between cavities.

The dip of the bedding planes of the Terntree formation is remarkably uniform resulting in a downstream component of 2° and 2° component to the west along the dam axis. Variation in dip adjacent to faults occur, but the largest dip measured is 15° immediately adjacent to a fault.

Terntree Formation.

The thickness of this formation measured from regional mapping in the district is about 600 feet but only 300 feet of section have been established at the damsite by drilling and outcrop pattern.

In drill cores and outcrop the lithology is uniform, the dominant rock type being a pebbly siltstone with minor pebbly sandstone beds. Minor mudstone horizons and calcareous concretion horizons also occur.

The sandstone beds have been used to correlate over the dam area ~~also~~ particularly one horizon containing a brachiopod Terriopsis which can be correlated over 1000 feet.

The limestone in the cores are at the same horizons as the lens shaped cavities ~~and~~ at the damsite which appear to form by weathering slight rotation of blocks along the valley sides. The important features are a component of dip about 4° to 5° to the west across the dam and a gentle dip downstream. The downstream component is desirable as the bedding planes will not be truncated by the valley floor downstream of the dam.

(b) Risdcn Sandstone:

This formation outcrops beneath the Ferntree Formation a few hundred feet south of the damsite in Risdon Brook. It was also intersected in the drilling between 47 feet and 72 feet 2 inches in bore No.8. Between the two formations there is a transition zone 2 feet thick, of sandstone with similar texture to the Ferntree Formation. The Risdon Formation consists of a single massive unit of feldspathic sandstone containing occasional pebbles of quartz and quartzite. Jointing is very widely spaced and bedding is absent. In the drill core the rock appeared to be slightly friable in contrast to the surface exposures and a few strong joints dipping about 75° relatively to the core were noted. The sandstone core contains numerous irregular black patches of organic material.

(c) Malbina Siltstone

Below 72 feet in bore hole No.8 the drill passed into a zone of broken rock containing much pyrite and with indication of movement along some of the joints. Beneath this, the hole passed into dense grey to black mudstone to 81'2" and finally into a grey siltstone similar to the Ferntree Formation to 82'9".

At the boundary of the Risdon Sandstone and Malbina Siltstone the rocks contained abundant pyrite in being and replacing pebbles and probably fossils. The loss of core and the generally broken nature of this zone is thought to be due to movement and also to the solution of rock fragments, joint fillings and cement by sulphate bearing groundwater.

(2) Structure:

The rock distribution and geological structure is indicated on the accompanying geological plans and sections. The main fault just upstream from the damsite has been mapped for some miles ~~and has been named the Lindisfarne Fault by earlier workers.~~ to the east. Sub-parallel to this is at least one smaller fault which cuts across Risdon Brook 440 feet downstream from the damsite. Other smaller faults may also occur in the area though so far they have not been detected in the current investigations. In a relatively narrow block of sediment founded by two faults, this is to be expected.

The ~~Lindisfarne~~ Main Fault:

Previous workers have shown that this fault has a displacement (downthrown to the north) of at least 250 feet and possibly more than 600 feet. On the accompanying plan the position of the fault has been fixed by the distribution of outcrops, by exposures in two trenches on the east abutment and by the contact reported in a trench excavated at creek level just upstream from the dam. From this information the trace of the fault in plan is remarkably straight indicating that the fault plane is nearly vertical.

The only exposures of the fault zone are in the two trenches on the eastern abutment. In the vicinity of the fault the Permian mudstone is closely jointed, weathered and brecciated for about 20 feet whilst the Triassic sandstone is only very slightly brecciated but somewhat weathered for a few feet. In both exposures the fault plane is occupied

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Notes on page (VII)

Risdon Sandstone

This formation outcrops in Risdon Brook a few hundred feet south of the damsite and was intersected in three drill holes immediately downstream of the dam. The complete section varying between 22'0" and 22'6" in thickness is composed of well bedded quartz sandstone, ~~slightly feldspathic~~ containing erratics up to 1½ inches in size, numerous carbonaceous fragments and minor shale bands.

This sandstone rests abruptly on the underlying calcareous shale but the top of the unit is a transitional zone 4 feet thick composed of interbedded sandstone and mudstone.

Malbina Formation

Only 16 feet of this formation has been penetrated by drilling at the damsite. It is composed of dark grey to black mudstone and siltstone similar lithologically to the Ferntree formation. ~~Howe~~ However the upper unit 4'0" thick is an interbedded fossiliferous limestone and calcareous shale.

The base has not been observed in the district and the greatest measured thickness is 240+ feet.

Structure.

The rock distribution and geological structure is shown on the geological plans and sections.

The main fault just upstream from the damsite has a displacement (downthrown to the north) of between 300 and 600 feet. The position of the fault has been fixed by three trenches 4 diamond drill holes and five road cuttings. ~~From this information the fault~~

The fault plane is near vertical and occupied by a ~~nom~~ narrow dolerite dyke varying in thickness from a few inches to 32 feet. The fault zone varies considerably in width from 4 feet to over 160 feet.

by a narrow dolerite dyke. The dolerite is closely jointed and badly weathered and the type varies in width from 26 feet to 9 feet between the two trenches, but it may be discontinuous over a wider area. Thin sections of samples from the dyke rock (specimens 64-24 and 64-222) indicates that the rock has affinities with the Jurassic dolerite.

Preliminary geophysical data indicates that on the Grasstree Hill saddle, the fault zone may be 250 feet wide. Drilling at present in progress supports this evidence. As indicated on the regional geological plan two large faults intersect in the vicinity of the saddle and this no doubt accounts for the wider zone of brecciation in that area.

Minor Faulting

Diamond drilling has demonstrated the existence of a small fault which has been indicated on the plan crossing Risdon Brook 440 feet downstream from the dam. Surface mapping, together with a study of the drill cores, show that the fault must have a minimum displacement, downthrown to the north, of 65 feet. The trend of the fault cannot be firmly established from the information available and the position indicated on the map is based on drilling results and a study of distribution of outcrops. It should not be regarded as proven. Where the fault crosses the road east of Risdon Brook, there is a narrow (4-5 feet) zone of brecciation.

Jointing

Yes.

The Permian System rocks are cut by a strong near-vertical joint system. Several trends are evident but a detailed analysis of several hundred joints indicates that no reliable structural interpretations can be made from such an analysis with the existing information. Many of the joints are open above the water table and in some a small amount of brecciation is present permitting water movement along the joint planes. Some brecciation is also evident in places along the bedding planes and they must also be considered as joint fault or movement planes in the same sense as the joints.

Water Pressure Testing

No

In the current drilling programme the drill holes are being tested with water under pressure every 10 feet. From these tests the permeability of each 10 foot section of each hole can be determined. The information available at this time indicates a close correlation between small broken zones, open joints and permeability.

Where the joints are closed or filled with clay and calcite the permeabilities are low. It may be noted that the method adopted gives a means of comparing different sections of a hole with one another and with other drill holes but the absolute values for the coefficient of permeability is probably incorrect.

Permeabilities claculated so far range from almost 600 feet per year, down to less than 1 foot per year. Since most of the leakage occurs along open joints, it should be possible to reduce the leakage to acceptable limits by means of grouting.

Geophysical Surveys

The Bureau of Mineral Resources carried out a series of refraction seismic and resistivity surveys over the damsite, saddle area and proposed borrow pit sites.

No

Preliminary results of this work are not yet available but the author has discussed the results informally with the geophysicist.

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Notes on pages (VIII)

Notes on Minor Faulting

"Minor Faulting" and surface exposure.

Diamond drilling in the dam area indicates minor faulting parallel to ~~major~~ the main fault occurs ~~to~~ 200 feet and 450 feet downstream of the dam with throws in the order of 20-30 feet.

~~Further downstream at~~

At 350 feet downstream another fault with displacement in excess of 50 feet

Several faults in the saddle area and upstream of the dam

also occur but displacements are generally small (less than 100 feet).

Notes on Water Pressure Testing.

Water Pressure Testing

All diamond drill holes for the detailed investigation were water pressure tested in 10 foot sections using a mechanical packer. Results of the tests are summarized in Appendix A. They show that the Triassic rocks, even in the faulted sections have the lowest permeabilities & that water movement occurs mainly along joints & fractures in the more brittle Permian rocks.

~~(The early~~ No ~~indications are that)~~ the geophysical work supports the geological and drilling results. Over the damsite the bedrock velocities varied between 11,000 and 16,000 feet per second indicating generally sound sedimentary rock with some areas of incipient weathering and open joints. Upstream of the dam in the vicinity of the Lindisfarne fault, lower velocities (6,500 ft.per sec.) were recorded. A similar zone of low velocity was recorded in the vicinity of this fault at the south end of the traverse across the saddle. Both seismic refraction and resistivity surveys indicated a considerable depth of weathering in the saddle area which has subsequently been substantiated by drilling. The greater depth of weathering in this area however does not necessarily imply a higher permeability as the clays resulting from such weathering tend to seal the rocks. Further water pressure testing in this area is at present being undertaken but the results to date indicate that it should be possible to seal off the more permeable zones and to restrict any leakage to an acceptable level. No

(I.B.JENNINGS)
SENIOR GEOLOGIST

30th March, 1965.

METROPOLITAN WATER BOARD.

RISDON BROOK DAM.

EIGHTH REPORT.

APPENDIX B.

NOTES ON RISDON BROOK DAMSITE

BY

R.P. MATHER - ASSOCIATE GEOLOGIST.

1. INTRODUCTION :

Geological reports and maps dealing with the site have been prepared by the Mines Department based on information obtained by the Department and also from other sources. The notes presented below are intended to be considered in conjunction with these geological reports.

Geological considerations concerning the proposed scheme are dealt with under the three headings - Stability of Site, Water Tightness and Available Materials of Construction.

2. STABILITY OF THE SITE :

Factors influencing this are the type and strength of the rock and the type and condition of the jointing it contains. At Risdon Brook the information to date shows the actual rock to be suitable but indicates that problems exist concerning flat joints or possible flat faults.

The existence of flat zones containing soft material and breccia have been established. The frequency and extent of these is not fully known at this stage, but it is expected that with the completion of the current drilling programme and the proposed adit, enough information will be available to define them in more detail. Because of the possible threat of sliding of the foundations produced by such planes of weakness, information about their positions is necessary to allow an adequate design. In addition to this a knowledge of the properties of the material filling these weaker zones is also necessary for adequate design.

It is expected that some form of further testing in situ will be carried out to determine this prior to final design.

Present information suggests that these zones are not very frequent but as their existence has been proved, a conservative view as to their affect must be taken until such time as there is enough information to indicate the contrary.

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3. WATER TIGHTNESS :

Experience has shown that in rocks of the type which occur within the storage at Risdon Brook, seepage of water through joints accounts for the major part of their permeability.

Existing reports have drawn attention to the problems of leakage, both at the dam and saddle adjacent to the Grass Tree Hill Road. In both areas, water pressure tests have been carried out in drill holes to determine permeability. Although this work is incomplete, tests to date show in general satisfactory permeabilities which indicate that leakage could be adequately reduced by grouting. However, in addition to considering leakage from a water loss view point, it must also be considered from the dam stability viewpoint. Water seepage through joints under the dam could result in piping within the foundation rock, or for certain types of dam within the material of the dam itself. The former is considered the less likely, especially with the low leakages which could be tolerated at this site from the water loss viewpoint, but could be countered by use of a graded filter in a fill type dam. The latter is considered possible with a clay core type dam unless some measures a.e taken to protect the clay core from the effects of seepage through joints in the adjacent bedrock. For example, a cut-off trench into the bedrock with some form of seal between the clay and the bedrock in the base of the trench.

With respect to grouting, which will certainly be necessary to some extent, there is now information available to show that diamond drilled grout holes are more suitable for a satisfactory grout take than percussion drilled holes. It may be necessary to consider this because of the importance in reducing leakage to a low amount at this site.

4. AVAILABLE MATERIALS OF CONSTRUCTION :

Clay: Experience has shown that clays derived from the weathering of igneous rocks have certain advantages over sedimentary clays. It is for this reason that clay derived from the dolerite or the basalt would be recommended rather than clay from other sources for a clay core dam.

Investigations to date suggest clay of these types is limited in extent within reasonable proximity of the site.

Rock: The dolerite nearest to the site is weathered in part and covered by overburden, but a little beyond and within reasonable proximity to the site it improves. Dolerite has advantages over the mudstone and sandstone from both strength and density considerations.

(sgd). R.P. MATHER.

Associate Geologist.

March, 1965.

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APPENDIX 'C' - DYNAMIC ELASTIC CONSTANTS measured by BUREAU OF MINERAL RESOURCES.

LAB. NO. (SAMPLE NO.)	BORE NO. AND (DEPTH in ft.& in.)	DENSITY (ρ) grm/cm^3	POROSITY (ϕ) %	BULK VELOCITY (V_p) ft/sec.	BAR VELOCITY (V_b) ft/sec.	POISSON'S RATIO (σ)	YOUNG'S MODULUS (E) 10^6 lb/in^2	BULK MODULUS (K) 10^6 lb/in^2	RIGIDITY MODULUS (G) 10^6 lb/in^2	LOG DECRE-MENT (δ)	QUALITY FACTOR (Q)	SAMPLE CONDITION	BEDDING
1472	4	2.26	15+	12,100	10,300	0.31	3.2	2.8	1.2	0.39	8	DRY	No bedding planes visible. A few fracture planes at about 90° to core axis.
(1)	(1')	2.37	11	12,450	9,300	0.37	2.8	3.7	1.0	0.22	14	SATURATED	
1473	4	2.34	12+	12,300°	9,100	0.37	2.6	3.4	0.9	0.18	17	DRY	As Above
(2)	(71' 3")	2.44	10.5	13,200°	11,000*	0.32*	4.0*	3.8*	1.5*	-	-	SATURATED	
1474	3	2.20	17+	12,070	11,330	0.22	3.8	2.2	1.6	0.12	27	DRY	No bedding or fracture planes visible
(3)	(14' 5")	2.35	15	12,100	10,450	0.30	3.5	2.9	1.3	0.15	21	SATURATED	
1475	3	2.29	13.5+	13,300	12,750	0.19	5.0	2.7	2.1	0.10	33	DRY	As sample 1.
(4)	(80' 3")	2.41	12.5	13,400	10,820	0.34	3.8	4.0	1.4	0.18	17	SATURATED	
1476	2	2.38	10.5+	13,550	9,290	0.40	2.8	4.6	1.0	0.15	21	DRY	No bedding planes visible. A few fracture planes between 15° and 85° to core axis.
(5)	(11'10")	2.47	9	14,100	8,860	0.42	2.6	5.4	0.9	0.18	17	SATURATED	
1477	2	2.44	8+	12,750	10,670	0.32	3.7	3.4	1.4	0.10	33	DRY	As sample 1.
(6)	(54' 2")	2.52	8	13,200	9,700	0.38	3.2	4.5	1.2	0.26	12	SATURATED	

REMARKS :

+ porosity calculated from dry weight assuming a matrix density of 2.65 grm/cm^3; other porosity values from wet and dry weights.

o V_p (dry & sat.) = 13,900 ft/s after sample broken; sample too small for complete set of measurements.

* values taken from incomplete set of measurements; less reliable than for other samples.

METROPOLITAN WATER BOARD

RISDON BROOK DAM

APPENDIX "D"

LIMITS OF HARDNESS IN A TOWN WATER SUPPLY

Hardness can be considered as being due to calcium carbonate dissolved in the water. Its degree is widely variable and is expressed in milligrammes per litre of calcium carbonate (mgl).

When water is heated, the carbonate tends to come out of solution and form encrustations on hot water coils and utensils generally. As hardness increases, it becomes increasingly difficult to form a lather when washing, soap consumption increases and a typical brownish scum forms which floats on the surface of the water.

Hard water is thus a nuisance and where people have been used to soft water, a hard water, or even a moderately hard water would probably be unacceptable. Where people have been used to moderately hard water, as in Adelaide, it is tolerated, and people tend to store and use rain water for clothes washing and bathroom purposes, or use domestic water softeners, which are not very satisfactory pieces of equipment.

The following figures assist the appreciation:-

Water Supply	Hardness Range
Hobart - Mountain System	18 - 27 mgl.
Lake Fenton System	9 - 30 "
Derwent River	17 - 30 "
Melbourne	12 "
Sydney (Warragamba Dam)	28 - 98 "
Adelaide	90 - 110 "
West Tamar	26 - 30 "
Nhill (Victoria)	310 - 345 "
Launceston	25 - 30 "

The only States for which comprehensive figures of water quality are to hand are Victoria and Queensland, and the hardness figures for the water supplies of the major towns of those States are as follows:-

VICTORIAN TOWNS (AS AT 1960).

Ararat	11 - 20	Castlemaine	25 - 60
Bairnsdale	14 - 62	Colac	17 - 35
Ballarat	41	Dandenong	7 - 31
Benalla	13 - 37	Echuca	16 - 26
Bendigo	30 - 57	Geelong	30 - 55

VICTORIAN TOWNS (Cont'd)

Horsham	12 - 21	Sale	25
Kerang	59 - 108	St.Arnaud	150 - 270
Kyneton	25 - 50	Shepparton	27 - 70
Mildura	28 - 87	Sorrento	36
Morwell	19 - 112	Frankston	16 - 37
Numurkah	16 - 29	Stawell	12 - 18
Ouyen	120 - 240	Swan Hill	32 - 115
Port Fairy	128	Traralgon	13 - 26
Portland	218	Wangaratta	12 - 38
Queenscliff	43	Warrackanabeal	80 - 178
Red Cliffs	31 - 62	Warrnambool	13 - 34
		Wonthaggi	18

QUEENSLAND TOWNS (AS AT 1960).

Atherton	5	Ipswich	185
Bowen	130	Kingaroy	275
Bundaberg	55	Longreach	55
Cairns	7	Mackay	80
Caloundra	120	Mareeba	35
Charleville	5	Maryborough	70
Charters Towers	85	Mt. Isa	100
Cloncurry	140	Mt. Morgan	305
Cunnamulla	7	Nambour	160
Dalby	315 & 120	Proserpine	55
Garton	670 & 65	Redcliffe	130 & 40
Gladstone	175	Rockhampton	130
Gordonvale	7	Roma	5
Gympie	85	South Coast	20
Harvey Bay	45	Stanthorpe	30
Ingham	50	Toowoomba	250 & 330
Innisfail	15	Townsville	7.5
		Warwick	140

The "International Standards" are -

Tolerable - up to 75 mgl.

Maximum - up to 200 mgl.

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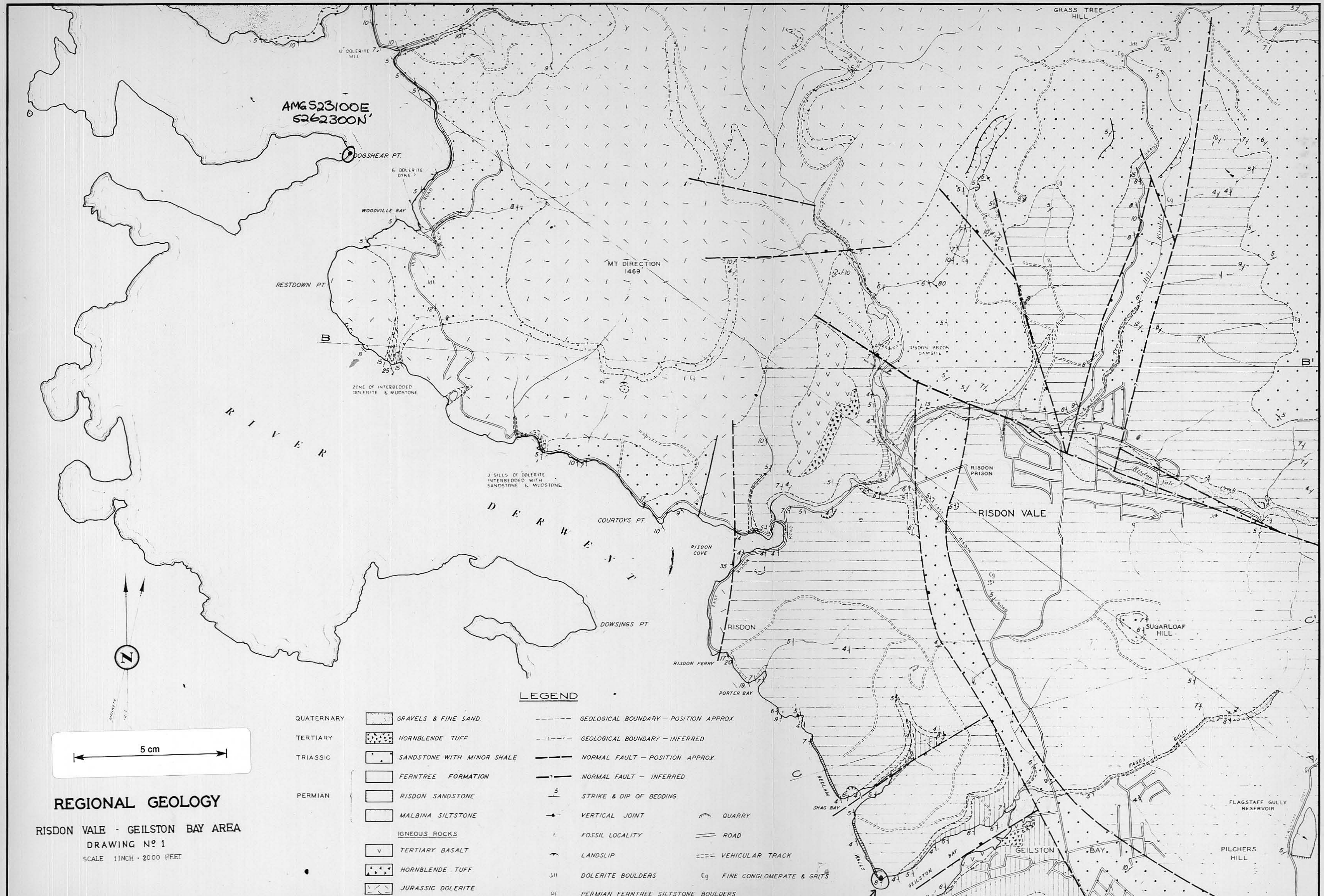
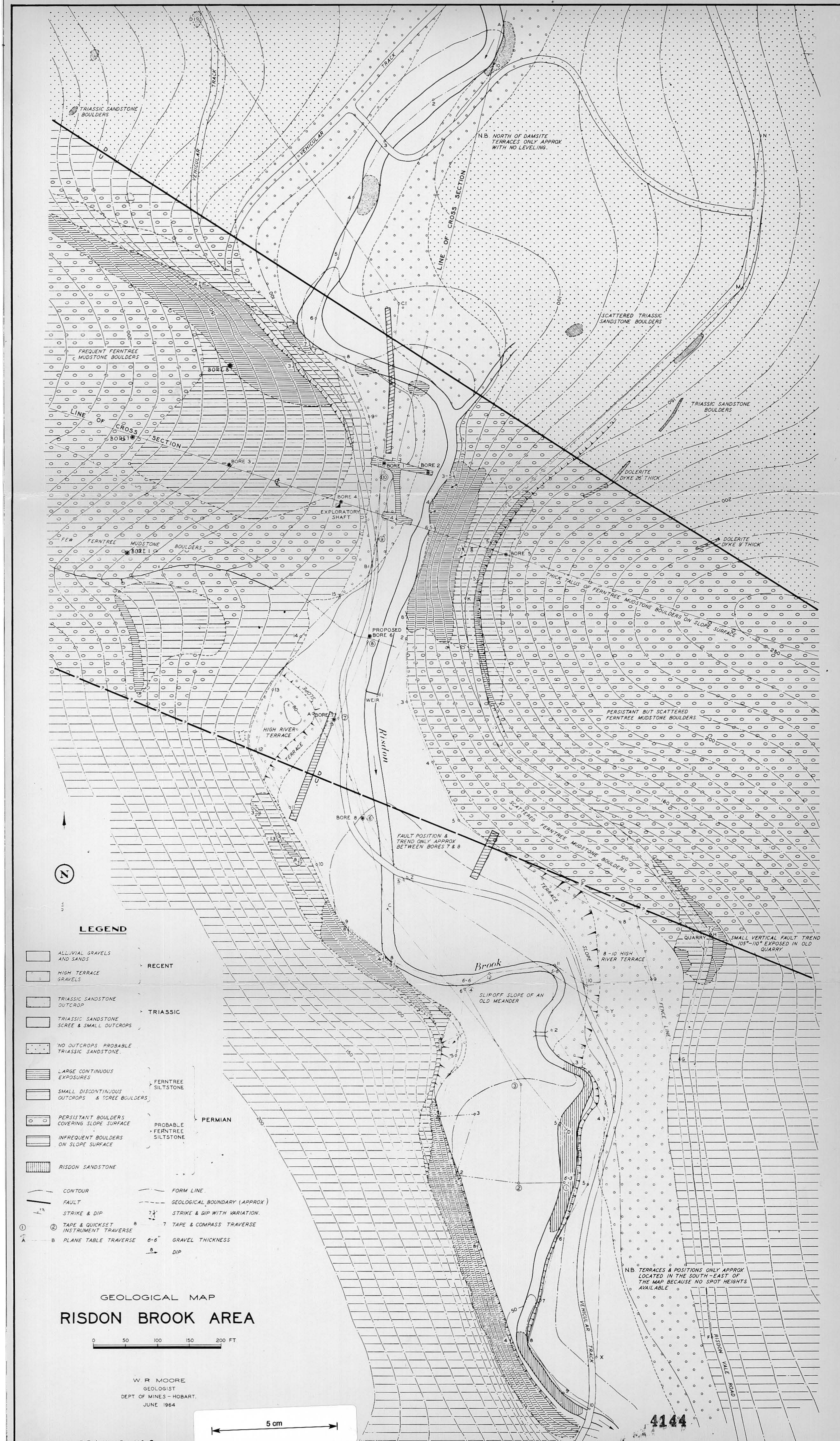


AMG REFERENCE POINTS ADDED

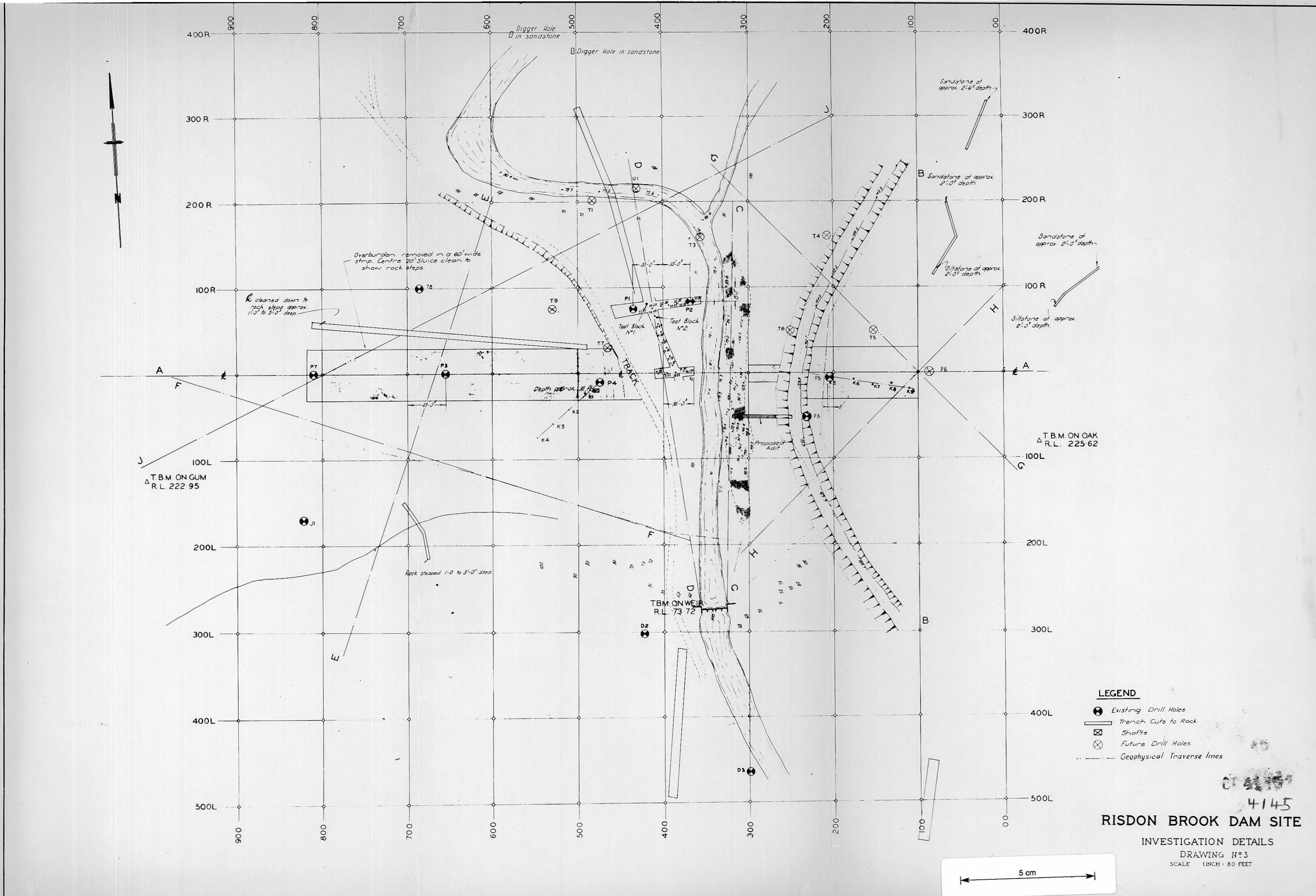
AMG 527150E
5256600N'

4143



TRIASSIC SANDSTONE BOULDERS
VEHICULAR TRACK
N.B. NORTH OF DAMSITE TERRACES ONLY APPROX WITH NO LEVELING.
LINE OF CROSS SECTION
SCATTERED TRIASSIC SANDSTONE BOULDERS
TRIASSIC SANDSTONE BOULDERS
FREQUENT FERNTREE MUDSTONE BOULDERS
BORE 8
BORE 7
BORE 3
BORE 1
BORE 2
BORE 4
EXPLORATORY SHAFT
DOLERITE DYKE 26' THICK
DOLERITE DYKE 9' THICK
FEW FERNTREE MUDSTONE BOULDERS
BORE 5
THICK TALUS OF FERNTREE MUDSTONE BOULDERS ON SLOPE SURFACE
PROPOSED BORE 6
WEIR
HIGH RIVER TERRACE
SLOPE
TERRACE
Risdon
PERSISTANT BUT SCATTERED FERNTREE MUDSTONE BOULDERS
BORE 8
FAULT POSITION & TREND ONLY APPROX BETWEEN BORES 7 & 8
SCATTERED FERNTREE MUDSTONE BOULDERS
QUARRY
SMALL VERTICAL FAULT TREND 105°-110° EXPOSED IN OLD QUARRY
Brook
8-10 HIGH RIVER TERRACE
SLIPOFF SLOPE OF AN OLD MEANDER
FENCE LINE
N
LEGEND
ALLUVIAL GRAVELS AND SANDS
HIGH TERRACE GRAVELS
RECENT
TRIASSIC SANDSTONE OUTCROP
TRIASSIC SANDSTONE SCREE & SMALL OUTCROPS
TRIASSIC
NO OUTCROPS. PROBABLE TRIASSIC SANDSTONE.
LARGE CONTINUOUS EXPOSURES
SMALL DISCONTINUOUS OUTCROPS & SCREE BOULDERS
FERNTREE SILTSTONE
PERSISTANT BOULDERS COVERING SLOPE SURFACE
INFREQUENT BOULDERS ON SLOPE SURFACE
PROBABLE FERNTREE SILTSTONE
PERMIAN
RISDON SANDSTONE
CONTOUR
FORM LINE
FAULT
GEOLOGICAL BOUNDARY (APPROX)
STRIKE & DIP
STRIKE & DIP WITH VARIATION.
TAPE & QUICKSET INSTRUMENT TRAVERSE
TAPE & COMPASS TRAVERSE
PLANE TABLE TRAVERSE
GRAVEL THICKNESS
DIP
N.B. TERRACES & POSITIONS ONLY APPROX LOCATED IN THE SOUTH-EAST OF THE MAP BECAUSE NO SPOT HEIGHTS AVAILABLE
VEHICULAR TRACK
RISDON VALE ROAD
GEOLOGICAL MAP
RISDON BROOK AREA
0 50 100 150 200 FT
W. R. MOORE
GEOLOGIST
DEPT OF MINES - HOBART.
JUNE 1964
5 cm
4144
2434-82
DRAWING No 2

252060

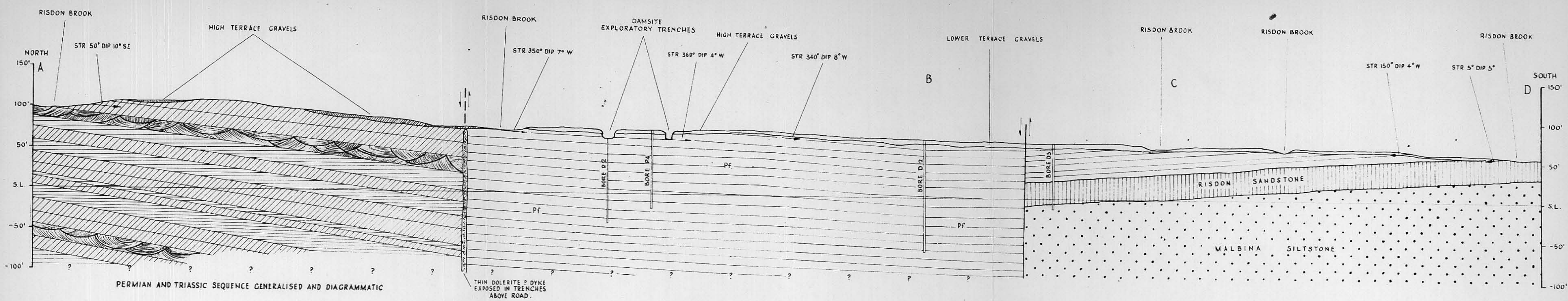


RISDON BROOK DAM SITE

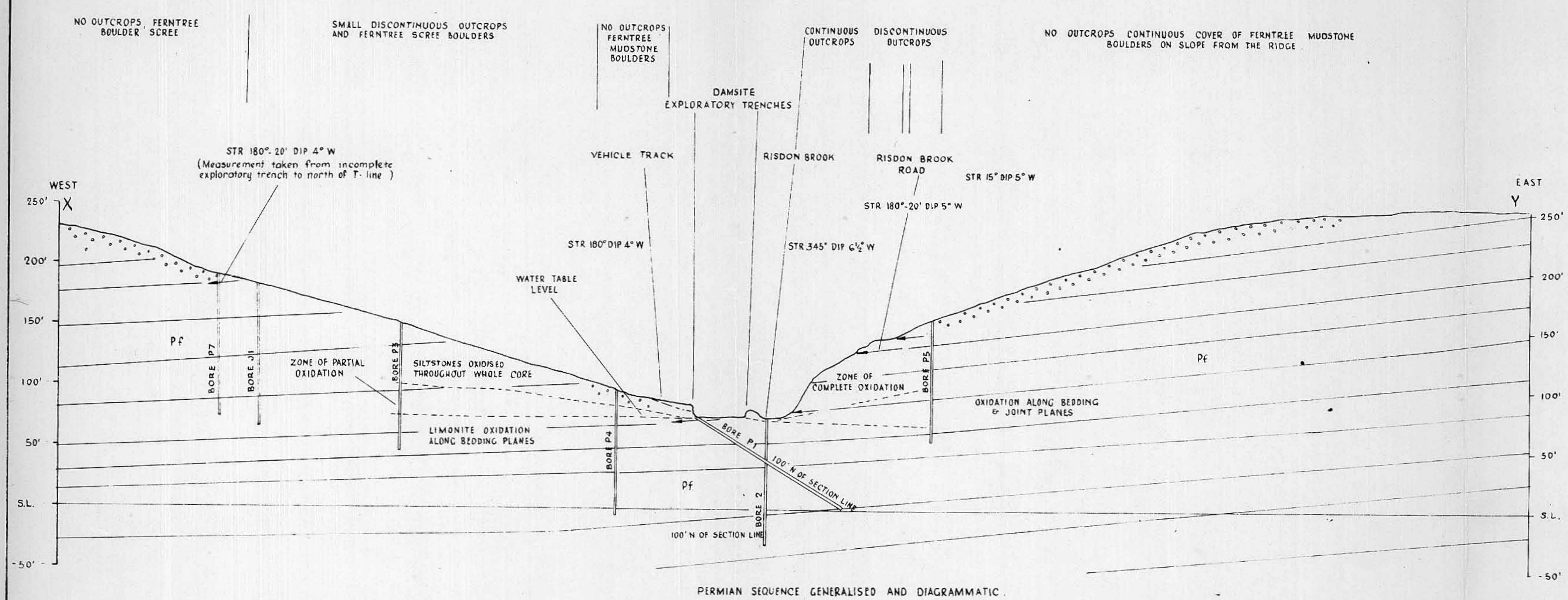
INVESTIGATION DETAILS

DRAWING Nº 3

SCALE 1 INCH = 80 FEET



LONGITUDINAL SECTION OF VALLEY



CROSS - SECTION OF VALLEY

GEOLOGICAL SECTIONS

RISDON BROOK DAMSITE

DRAWING No 4

SCALE : 1 INCH = 100 FEET

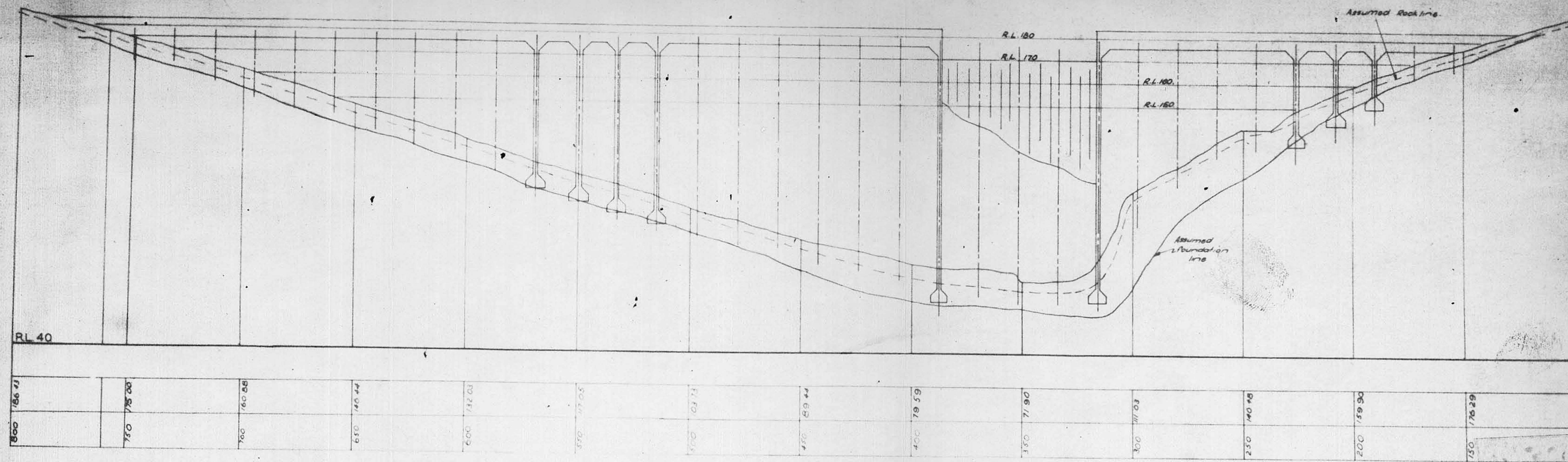
5 cm

4146

252062

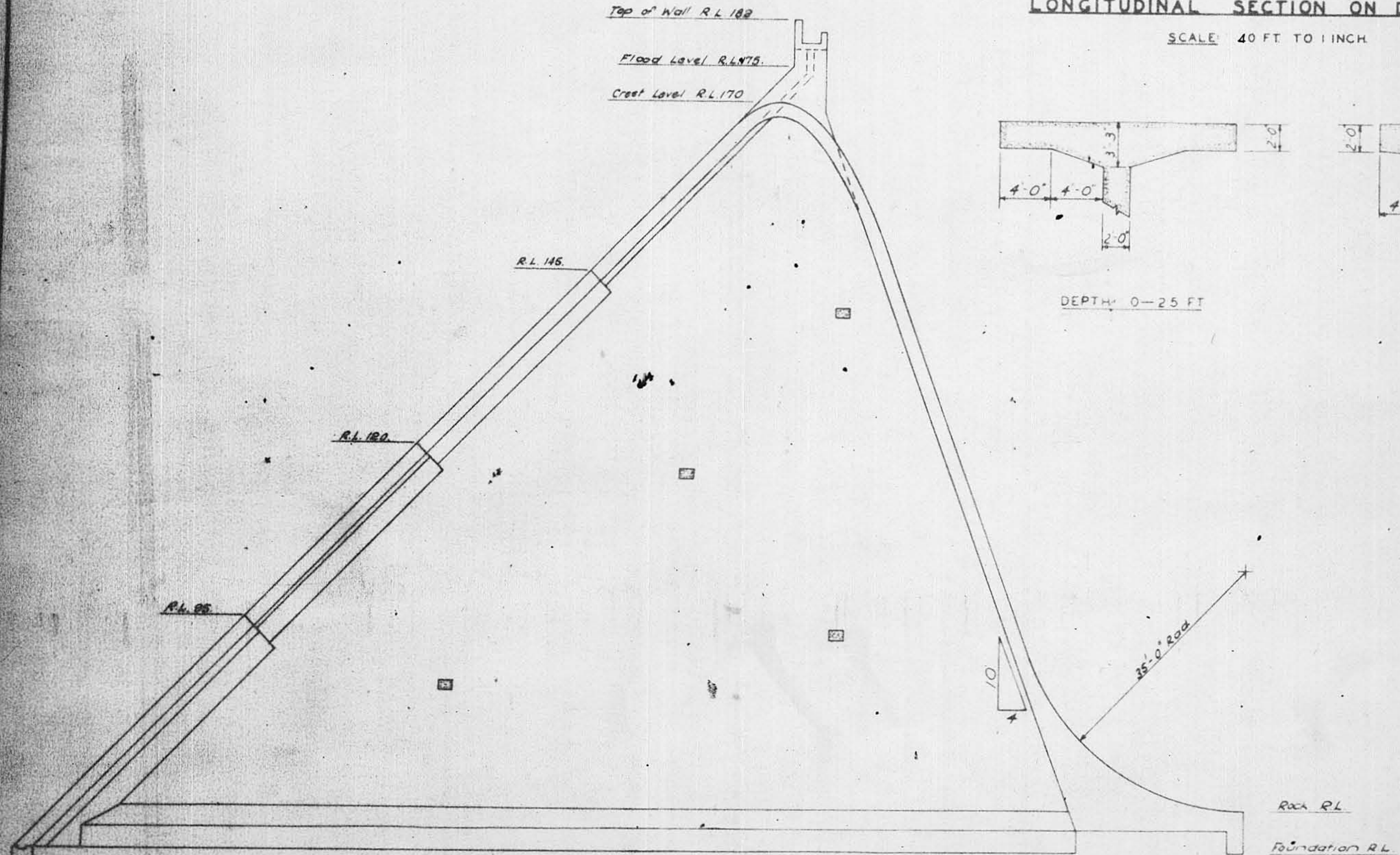

BORE LOGS
RISDON BROOK DAM SITE
VERTICAL SCALE 1" = 20 FEET
BORE P7
R.L. 190'
CORE RECOVERY %
PERMIBILITY FEET PER YEAR K (10 MIN)
BROWN GREY SILTSTONE
MOTTLED SILTSTONE
GREY MOTTLED SILTSTONE
SILTSTONE WITH LAYERS OF FIRM BROWN CLAY
HARD GREY SILTSTONE
HARD BLUE GREY SILTSTONE
BORE J1
R.L. 145'
MOTTLED SILTSTONE
LAYERS OF CLAY
SILTSTONE
MEDIUM GREY SILTSTONE
SILTSTONE LAYERS OF GREY SANDSTONE
LAYERS OF DECOMPOSED SANDSTONE
HARD GREY SILTSTONE
HARD BLUE GREY SILTSTONE
BORE T8
R.L. 150
MOTTLED SILTSTONE
HARD BROKEN MOTTLED SILTSTONE
CLAY BAND
BLUE GREY SILTSTONE
BLUE GREY SILTSTONE
BORE P3
BRECCIATED PUGGY BAND
QUARTZ FELDSPATHIC SANDSTONE
THIN VENEER OXIDISED CLAY
ZONE OF DARK MUDSTONE IN SILTSTONE
BRECCIATED PUG BAND
COARSE SANDSTONE WITH ROCK FRAGMENTS
MID-FINE CREAMY MASSIVE SANDSTONE
CLAY BAND REPORTED
BORE P4
MUDSTONE BAND
CREAMY SILTY LIMESTONE
BLACK MUDSTONE BAND
LAMELLAE BLACK MUDSTONE BEDS WITH PYRYTE
THIN PYRITE VEINS ALONG BEDDING PLANE EVERY FOR 18 ZONE
SHEAR POLISH ON DARK MUDSTONE
BORE P1
RECOVERY NOT ESTIMATED CORE SHATTERED
2" CLAY BAND POSSIBLY JOINT INFILL CLAY SUBVERTICAL JOINTING PRESENT
15' 4" OXIDATION OF SILTSTONE ALONG BEDDING PLANE CEASED
28' GREY COLOUR FINE GRAINED MUDDY CALCAREOUS SANDSTONE
9' FINE GRAINED MUDDY DARK LIMESTONE
4' CREAMY GREY GREEN CONCRETIONY LIMESTONE BAND
31' LIGHT GREY SILTY SANDSTONE AND SANDY SILTSTONE
N.B. BORE 1 DRILLED AT BEGINS AT 73.36' NOT 70.00' AS INDICATED ON SCALE.
SEA LEVEL
FIGURES BESIDE COLUMS REFER TO
1 DEPTH CORE PULLED 56' 9"
2 PERCENT CORE RECOVERED 88%
FINE SANDSTONE
SANDY SILTSTONE - SILTY SANDSTONE
SILTSTONE
CLAY OR MUDSTONE
LIMESTONE
FAULT PUG & OR BRECCIA
SHEAR ZONE
Pf FERNTREE FORMATION
Pr RISDON FORMATION
Pm MALBINA FORMATION
PERMIAN
BORE Nos P7, J1, T8, P8, & S2 FROM DRILLERS LOG ONLY
BORE P8 INCLINED ANGLE 75°
GREY MOTTLED SILTSTONE
SOFT CLAY SEAM
VERY SOFT LAYER
GREY MOTTLED SILTSTONE
BORE P2
R.L. 72.33
SLIGHT OXIDATION TO LIMONITE ALONG BEDDING PLANES
19' GREY MUDDY FINE GRAINED LIMESTONE
3' STONE DARK MUDSTONE PYRITE ALONG BEDDING
3' GREY GREEN CONCRETONARY LIMESTONE
7' FINE MUDDY SANDSTONE WITH PYRITE GRAINS
3' SILTSTONE
28' FINE SILTY SANDSTONE AND SANDY SILTSTONE
9' LIGHT GREY CALCAREOUS MUDSTONE
FOR DETAILS OF CORE DEPTHS & CORE RECOVERIES
BORE Q2
TERRACE GRAVEL MAINLY DOLERITE BOULDERS
CORE BROKEN BECAUSE HIGHLY JOINTED
BORE Q3
TERRACE GRAVEL MAINLY DOLERITE BOULDERS
FAULT CRUSH OR JOINT INFILL CLAY
FERNTREE CHANGES GRADUALLY FROM SILTSTONE TO SANDY SILTSTONE
TEXTURE OF FERNTREE MUDSTONE & GRAIN OF RISDON S.S. SIZE
QUARTZ FELDSPATIC
DARK SOFT MUDSTONE
NO CORE RECOVERED
SLICKENSIDES SEEN ON JOINT PLANES
SHATTERED MUDSTONE
SILTSTONE WITH LAMELLAE BEDDING SHOWING FERNTREE TEXTURE
BORE S2
R.L. 210
MUDSTONE
SANDSTONE WITH CLAY SEAMS
MUDDY SANDSTONE
SANDSTONE
VERY SOFT SANDSTONE
VERY SOFT SANDSTONE
SOFT SANDSTONE
SANDSTONE
SOFT SANDSTONE
SOFT SANDSTONE
MUDDY SANDSTONE
SOFT MUDSTONE
MUDSTONE
5 cm
DRAWING No 5


SCOTTS ROAD
GRASS TREE HILL ROAD
DAM
200
177
150
100
250
D2
D3
D4
D5
D6
U1
U2
U3
U4
S1
S2
S3
S4
S5
S6
S7
Q1
Q2
Q3
Q4
Q5
Q6
Q7
Q8
Q9
00
500
1,000
1,500
2,000
2,500
3,000
3,500
4,000
4,300
LEGEND
Future drill holes
Existing drill holes
Auger holes
Natural cuttings
Backhoe holes
Geophysical Traverse lines
5 cm
RISDON BROOK DAMSITE
REPORT Nº 8
VALLEY INVESTIGATIONS
DRAWING Nº6
SCALE: 1 INCH · 200 FEET
4148
252064



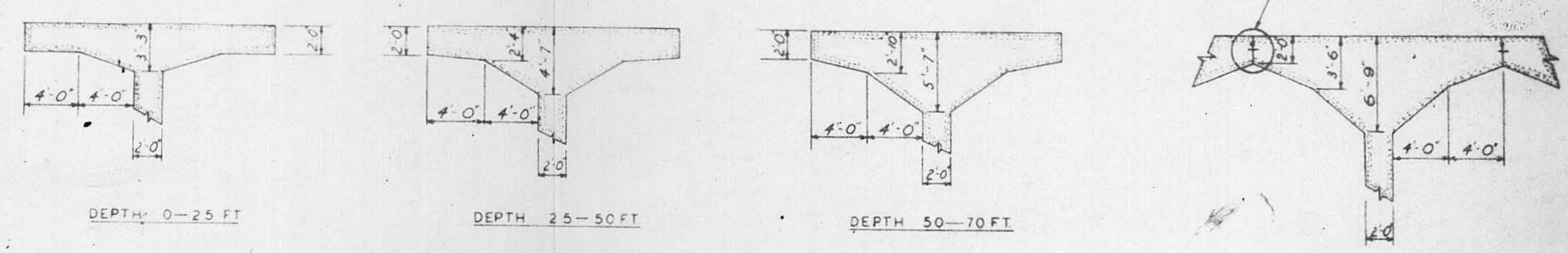
LONGITUDINAL SECTION ON DAM CENTRE LINE

SCALE: 40 FT TO 1 INCH



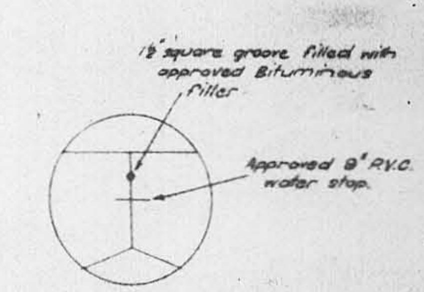
SIDE ELEVATION OF BUTTRESS

SCALE: 20 FT. TO 1 INCH



BUTTRESS HEADS.

SCALE 10 FT. TO 1 INCH



DETAIL A

SCALE 1"=1'-0"

Notes:-

1. Sections based on Danjera Dam.
2. Estimated quantity:- 100' high 25,200 cu. yds. Foundation RL 55
   80' high 15,900 cu. yds.
3. Spillway capacity 3,000 cusecs.
4. Stresses :- F'_c = 3000 P.S.I.
   Allowabe Tensile Stress at Face = 200 P.S.I.
   f_s = 20,000 P.S.I.

RISDON BROOK DAM

REPORT Nº 8

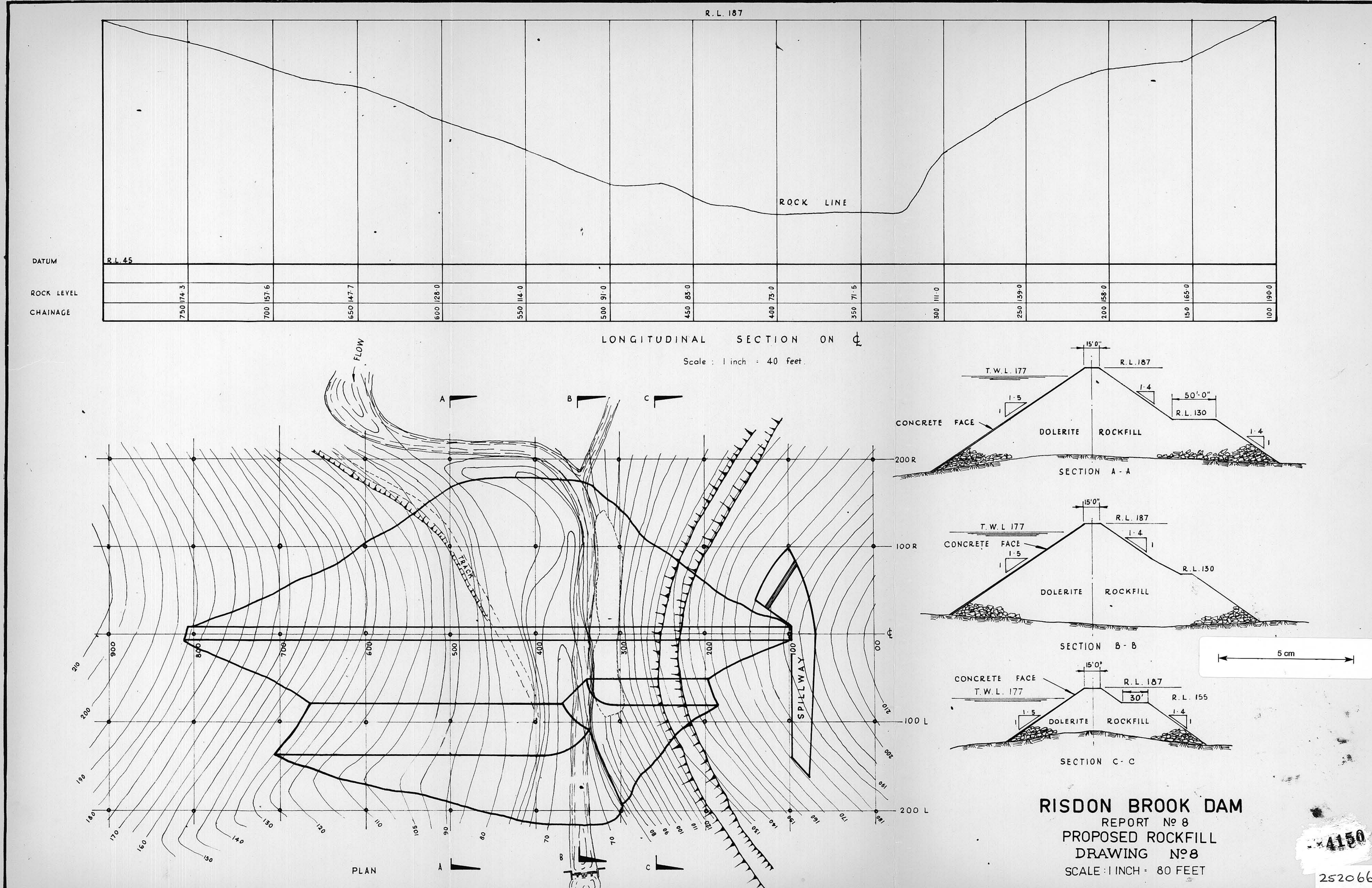
PRELIMINARY BUTTRESS SECTIONS

DRAWING Nº 7

4149

5 cm

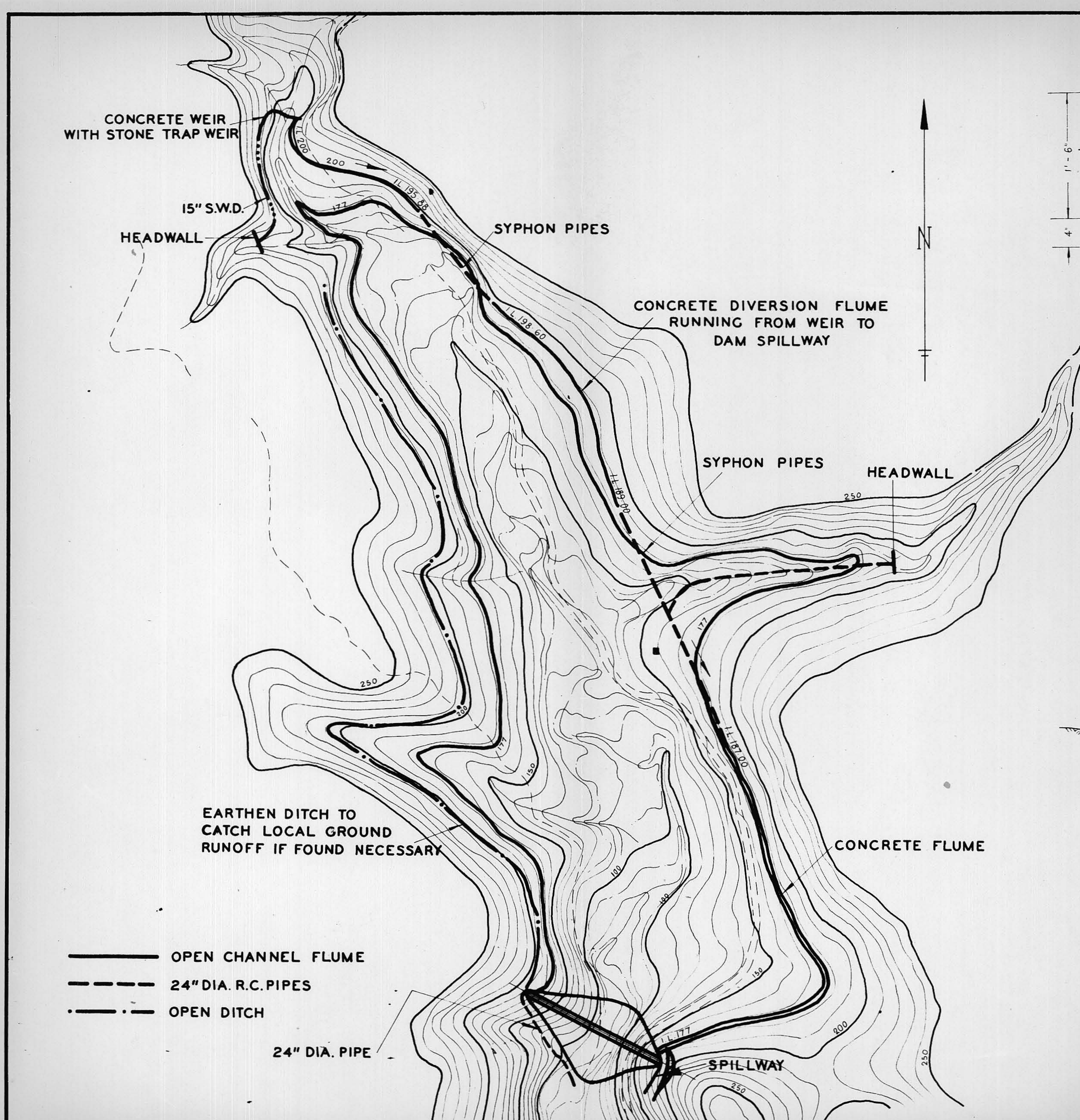
252065

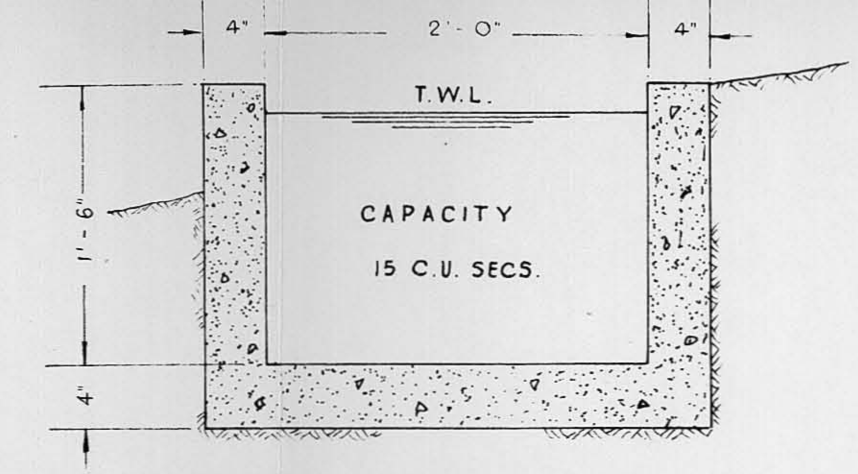


RISDON BROOK DAM
REPORT No 8
PROPOSED ROCKFILL
DRAWING No 8
SCALE : 1 INCH = 80 FEET

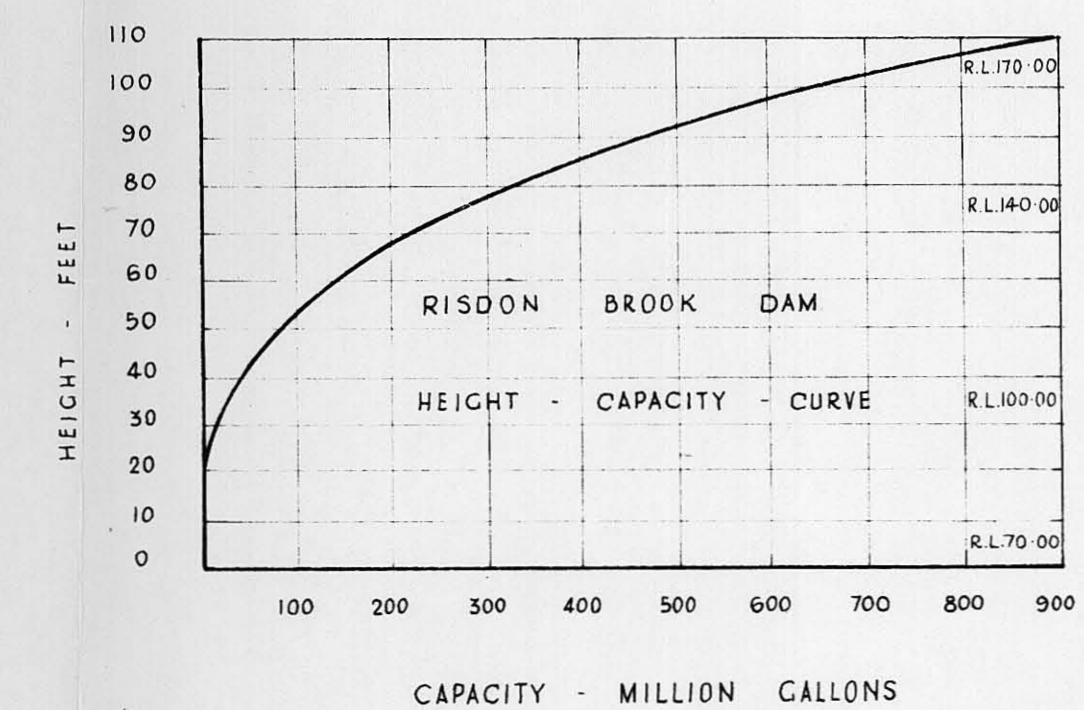
4150

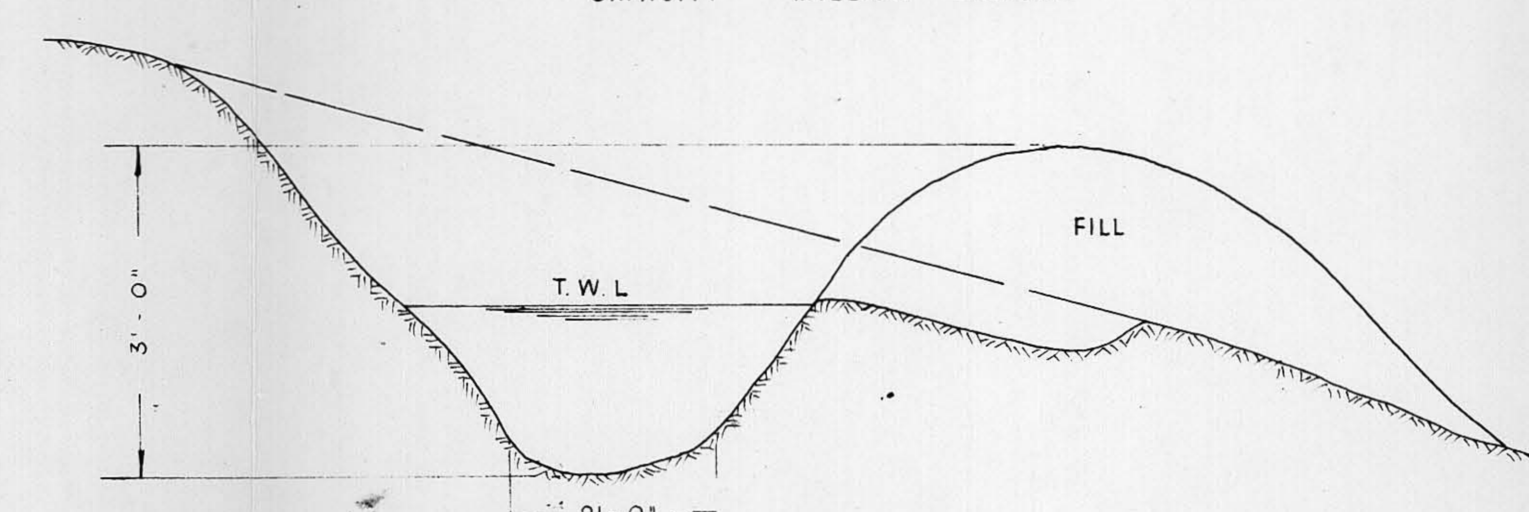
252066.





TYPICAL FLUME SECTION





TYPICAL EARTH DITCH SECTION

5 cm

RISDON BROOK DAMSITE

REPORT Nº 8

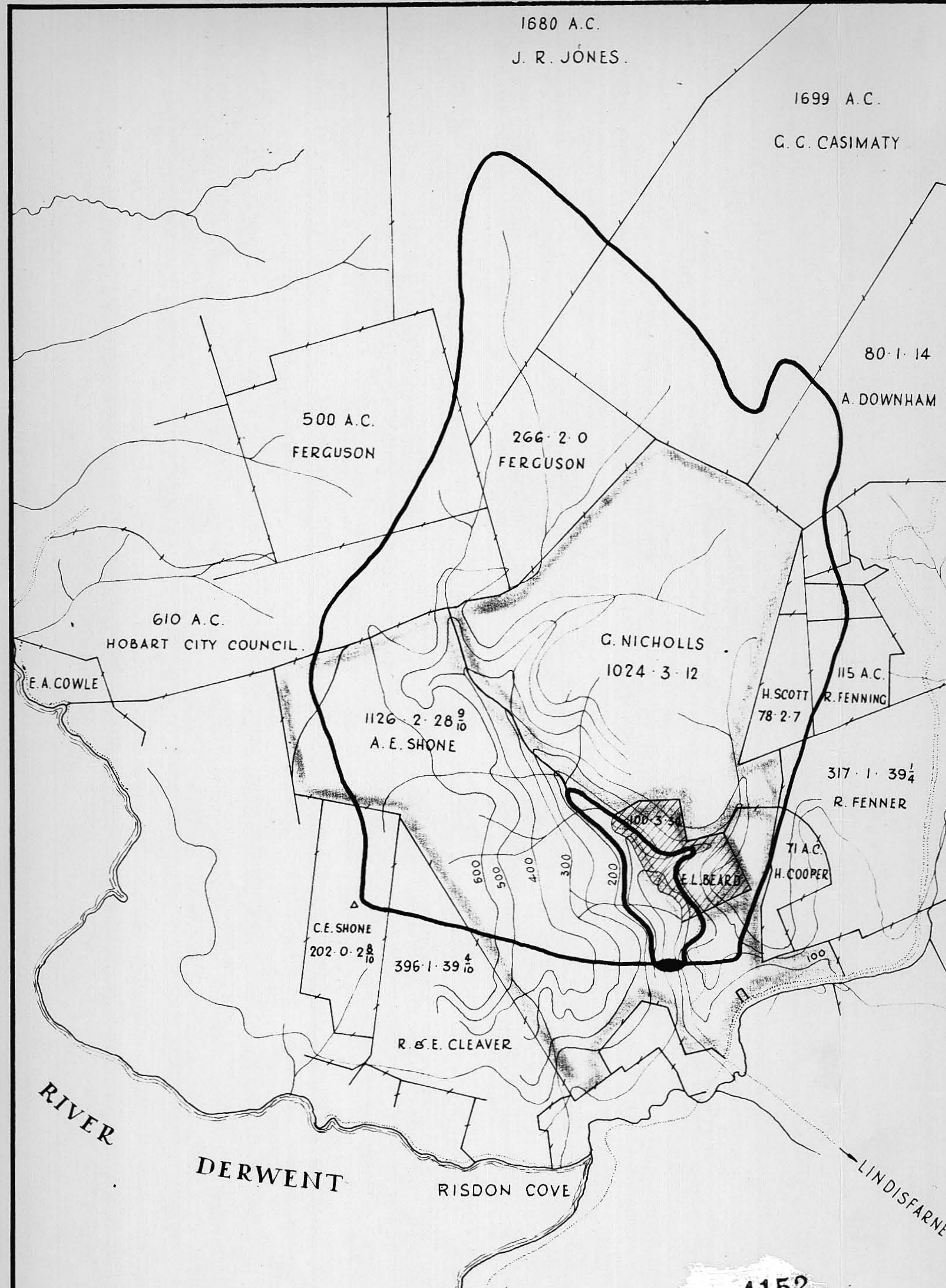
STREAM DIVERSION

DRAWING Nº 9

SCALE: 1 INCH = 400 FEET

-4151

252067



4152

NOTE :- BOUNDARIES SHOWN THUS :
SHADED AREAS HAVE BEEN PURCHASED.

5 cm

RISDON BROOK DAMSITE

REPORT No 8

CATCHMENT & STORAGE AREAS

DRAWING No10

SCALE ½MILE = 1 INCH

252068


RISDON BROOK SCHEME
REPORT No 8
DISTRIBUTION TRUNK MAIN
DRAWING No 11
SCALE : 1000'-0" - 1 INCH
PROPOSED TRUNK MAIN
PROPOSED FUTURE TRUNK MAIN
EXISTING OR APPROVED TRUNK MAINS
RISDON BROOK RESERVOIR T.W.L. 177
PUMP STATION 6 M.G.D.
24" DIA. RISING MAIN
FUTURE RESERVOIR
5·4 M.G.D.
RISDON BROOK DISTRIBUTION MAIN
27" DIA.
SOUTHERN REGIONAL WATER SUPPLY LINE 3·8 M.G.D.
18" DIA.
PILCHERS HILL RESERVOIR
RATE CONTROL VALVE
FUTURE EXTENSION
TO TORRAH RESERVOIR
12" DIA.
15" DIA.
FROM WEST DERWENT WATER SUPPLY 3 M.G.D.
RICHMOND MUNICIPALITY
CLARENCE MUNICIPALITY
RISDON VALE
LINDISFARNE
WARRANE
BELLERIVE
GEILSTON BAY
LIMEKILN POINT
LINDISFARNE POINT
MONTAGU BAY
ROSNY
ROSNY POINT
KANGAROO BLUFF
SECOND BLUFF
Bedlam
Walls
AMG 528500E 5260200N
AMG 528150E 5256300N
5 cm
AMG REFERENCE POINTS ADDED
4153
252069