Drakensberg pumped storage scheme: rock engineering aspects

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Synopsis

The Drakensberg Pumped Storage Scheme is a multi-purpose project being undertaken jointly by the Electricity Supply Commission and the Department of Water Affairs. The Scheme involves three major underground excavations for the pumping and generating plant and associated waterways and access. There is in addition an extensive series of inter-connecting tunnels to act as waterways and access. The general approach to the rock engineering aspects of the Scheme are described and details are given of the project feasibility considerations and subsequent exploratory rock mechanics and geotechnical work. The geological programme is outlined and details are given of the rock mechanics investigations involving sample and in-situ testing and evaluation of rock reinforcement and pneumatically applied concrete. The testing of an enlargement to the full cross sectional dimensions of the future machine hall and of a full scale penstock test chamber to determine the feasibility of concrete lined pressure tunnels are described. The inter-relationship of investigation, testing, analysis and design are considered in the Paper.

Introduction

1. Engineering description of the works

The principal source of water for Johannesburg and other centres in the Witwatersrand for both industrial and domestic purposes has traditionally been the westward flowing Vaal river (Figure 1). However, this source is no longer sufficient to provide an assured supply to meet the demands of this rapidly growing area. As a result, the Department of Water Affairs has proceeded with a scheme to abstract water form the upper regions of the eastward flowing Tugela river and to pump this over the Drakensberg escarpment into the Vaal catchment.

The first phase of this scheme was completed in 1974 and water is now being pumped over the excarpment from a pumping station at Jagersrust.

The predicted demand for water from the Vaal catchment would have made it necessary to duplicate this pumping scheme by 1980. However, as an alternative, studies were carried out into the possibility of developing a pumped storage station which could be used both for pumping water into the Vaal for water supply purposes and for storing electricity. These studies were undertaken jointly by the Department of Water Affairs and the electricity Supply Commission (Escom) as a result of which it was decided to proceed with the Drakensberg Pumped Storage Scheme (1).

Escom will use the Drakensberg Scheme primarily to store surplus off-peak energy from thermal power stations. For water supply purposes the Department will draw water from the upper reservoir (Direkloof).

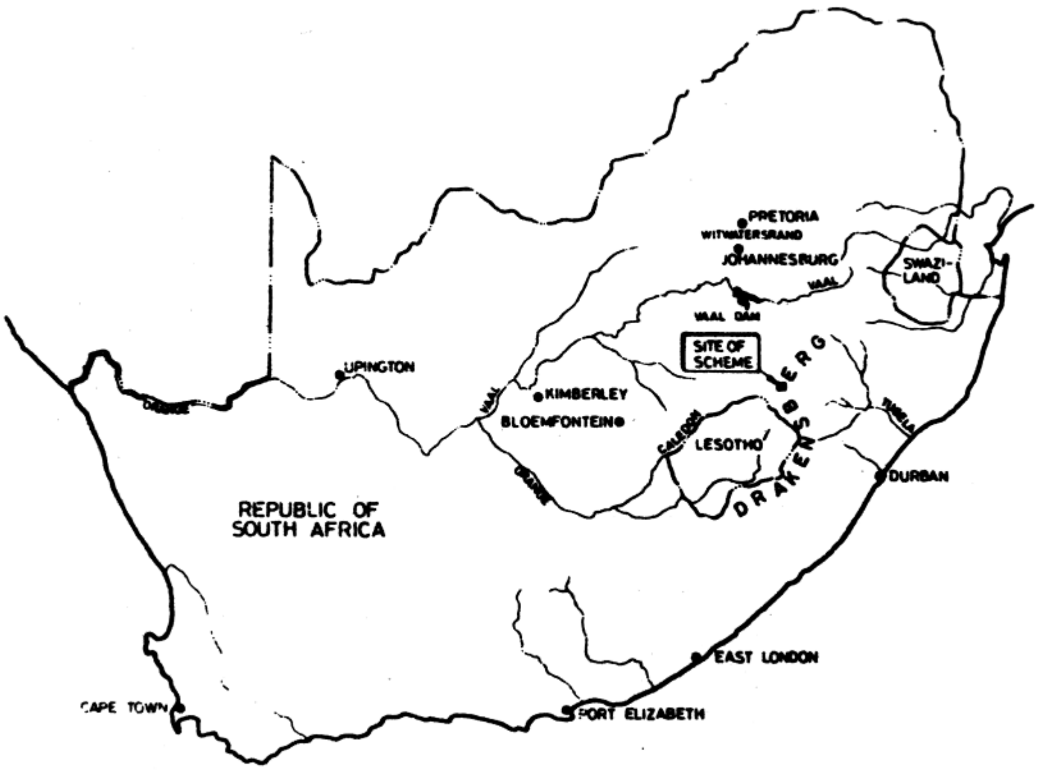

Figure 1. Local Plan

The pumping head will be approximately 500 m, one of the highest in the world for this type of scheme.

The Scheme, which is now in the early stages of construction, is located approximately 5 kilometers to the north of the existing Jagersrust pump station. The principal engineering structures include an underground power station to contain four reversible pump-turbines each driving a 250 MW reversible motor-generator, associated tunnels and shafts, surge chambers and access tunnels (Figures 2 and 3). These works will be carried out under contract to Escom. In addition, the Scheme involves the construction of two major embankment dams at Kilburn and at Driekloof and extensive open excavations. These excavations and dams are not discussed further in this paper.

An initial study of the rock mechanics aspects of the Scheme was made by the Council for Scientific and Industrial Research (CSIR). After this study, Escom appointed Gibb Hawkins and Partners as its consulting engineers for the design and site supervision of the excavation, stabilization and lining of the underground works. Gibb Hawkins and Partners in turn appointed Golder Associates of the United Kingdom as their specialist advisers for the rock engineering aspects of the Scheme.
Two exploratory contracts were awarded by Escom at the beginning of 1975. The first contract involved excavation of an exploratory adit and an exploratory shaft with interconnecting headings in the area of the future machine hall. In addition provision was

made in this contract for excavation of a test enlargement to the full span of the machine hall and construction of a trial length of concrete lined pressure tunnel. The second contract was for exploratory drilling both from the surface and from underground.

In August 1975, a preliminary contract was awarded for the excavation of the tailrace tunnel and main access tunnel (headrace and tailrace refer to the generating mode). Further contracts will be awarded by Escom for the remainder of the underground work.

This paper describes the rock engineering aspects of the Scheme including investigation, testing and design phases. Particular emphasis is placed on the power station excavations. Following a brief summary of geological and groundwater conditions, the main rock engineering aspects are discussed. At the time of writing (July 1976) the exploratory works are still in progress and full results are therefore not available.

2. Summary of Geological and Groundwater Conditions

2.1 Geology

The underground works lie mainly beneath the south facing slopes of the Drakensberg escarpment between elevation 1730 m at the top of the surge shafts and elevation 1214m at the invert of the tailrace portal (Figure 2).

The Scheme is sited mainly within rocks of the Beaufort Series of the Karroo System. Fossil and stratigraphic evidence indicates a continental depositional environment. Lateral facies variations are common and therefore the boundaries of the lithological untit are frequently diachronous.

A distinct division of rock types in the vicinity of the Scheme occurs below a prominent sandstone horizon which outcrops at approximately elevation 1550 m (Figure 2).

Below the marker horizon the rocks are primarily sandstones, siltstones and mudstones from the Middle and Lower Beaufort series. Siltstones and mudstones from these horizons vary in colour from greenish or bluish grey to dark grey.

Above the marker the rocks consist mainly of interbedded sandstones, siltstones and mudstones of the Upper Beaufort Series, the mudstones and siltstones of which have respectively and distinctive reddish-brown and greyish-green colouration.

In addition to the rock types mentioned above there are also occasional thin carbonaceous seams predominantly in those rocks below the marker horizon. Carbonaceous seams are usually thin, poorly developed fossil leaf remains within dark mudstones.

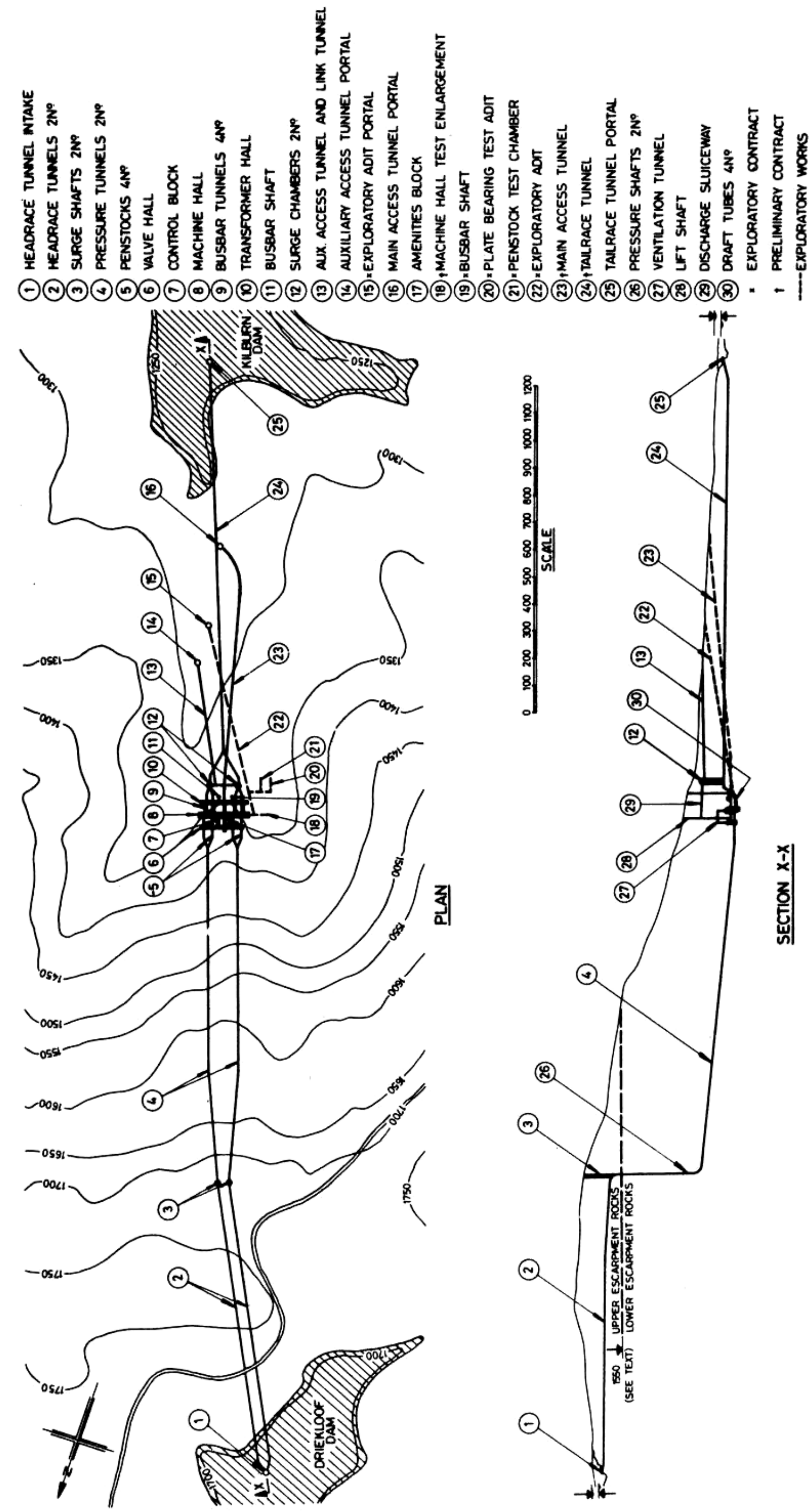

Figure 2. Longitudinal plan and section of works

Dolerite sills and dykes are also present. The dykes are typically near-vertical, from one to three metres thick, and often bounded by slickensided, serpentinised shear zones. A major sill having an upper elevation of 1133 m and over 70 m thick has been found beneath the proposed machine hall.

The strata within the Site area are essentially horizontal and bedding forms the dominant structural feature. Bedding plane spacing has been found to vary from less than 5 mm to up to 500 mm. Frequently cross bedding and other sedimentary structures occur within the lithological units.

Joint surveys have been carried out both on surface exposures and in underground adits. The surveys show three statistically significant near-vertical jointing trends striking at approximately 120º, 160º and 190º. In interpreting the significance of the jointing it should be remembered that the major axes of the main halls are east-west, the axes of the tailrace and pressure tunnels are approximately north-south. (All directions in this paper are with respect to magnetic north, 19º04’ west of grid north).

Faults are usually associated with dolerite dykes although displacements are generally limited to a few metres.

2.2 Groundwater

Borehole permeability tests carried out over the area of the Scheme show a range of permeability from 10^{-6} m/s to 10^{-9} m/s. In general, the sedimentary units are relatively impermeable. Permeable fractured zones up to 2 m thick have been found at dyke margins.

Piezometers were installed in selective boreholes to determine existing groundwater conditions, seasonal variations and the influence of tunnel excavation on groundwater pressures.

The presence of low permeability mudstone and siltstone seams gives ise in general to a preferred ground water flow direction parallel to the bedding within the more permeable units. The dykes and dyke margins modify this general flow pattern by dividing the rock mass into a series of reservoirs. Significant changes in peizometric head across dykes have been observed.

The groundwater table is generally close to surface. Significant departures from hydrostatic conditions with depth are expected in the vicinity of the escarpment.

Rock Engineering Aspects of the Scheme.

1. General Approach

The underground excavations required for a hydro-electric scheme such as the Drakensberg project have to be designed to satisfy a number of hydraulic, mechanical and electrical requirements. These requirements impose certain constraints upon the size, shape, depth below surface and orientation of the various excavations which make the underground complex. Within these constraints, rock engineering principles are applied to check the feasibility of the proposed layout (basic consideration of excavation size, spacing and geology). Further, they are used to design the detailed shapes, excavation sequences and support systems which are necessary to ensure that the excavations remain stable throughout the life of the project.

The scope of the investigation, testing and design processes depends largely upon the characteristics of the rock mass and the nature of the in-situ stress field. These factors will determine the degree of stability inherent in the roof, sidewalls and inter-section of the excavations and the necessary reinforcement to maintain required stability conditions permanently.

During feasibility studies for a project, only the most general type of geological information is usually available. At this stage, the potential behaviour characteristics of the rock mass are identified in very broad, general terms. Precedent and experience from other similar schemes are principal factors.

Such experience is related to the rock mass and excavation geometry by means of geomechanical classifications (2 and 3). Geomechanical classifications can be used to distinguish between different potential failure modes which may range form structurally controlled roof falls in jointed hard rock masses, to failure of intact rock material under stress in more homogeneous, weaker rock masses.

The potential behaviour and possible support requirements for the underground excavations are then estimated.

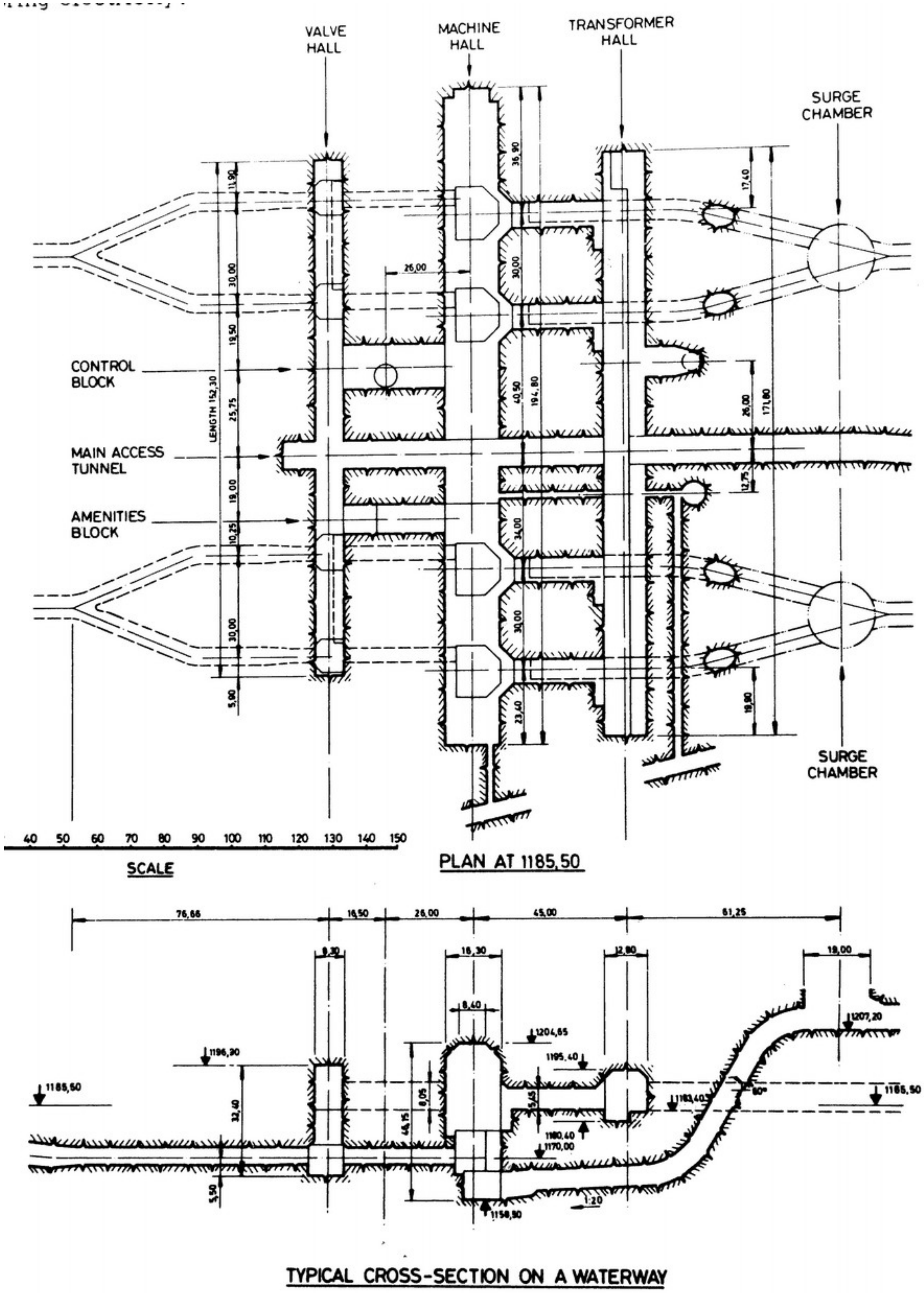

Figure 3. Plan and section of power station

During the exploratory phase the available geological information becomes more detailed and a final specific design is formulated. At this stage, the mechanical behaviours for the rock mass is considered in terms of the most dominant characteristics and the investigation, testing grand design processes are related to these specific parameters.

2. Project Feasibility Stage

As previously mentioned, the earliest studies on the rock mechanics aspects of the Drakensberg project were carried out by the CSIR and were based upon the use of a geomechanics classification system (2). These studies suggested that stable roof spans would be limited by the poor quality, horizontally bedded rock mass in which te major underground excavations would probably be located. Other schemes with similar rock conditions to Drakensberg were appraised. Table 1 gives the general rock mass characteristics and machine hall sizes for the Drakensberg, Poantina and Portage Mountain hydro-electric projects.

Based upon comparisons with the Poatina scheme in Tasmania (4), the use of a trapezoidal roof arch with a total span limited to 20m was considered. In order to minimise the effective overall height of the Drakensberg excavation and thus ensure greater inherent stability of the sidewalls, each of the four pump-turbines was planned to be located in individual pits separated by adequate rock pillars.

Following a detailed appraisal of the CSIR recommendations several major points emerged which affected the layout of the scheme and the scope of the subsequent exploratory works. These were as follows:

1. In accordance with the CSIR's recommendations, the pump-turbines would be located in a machine hall of the minimum possible cross-sectional dimensions. This would be achieved by separating components such as transformers and valves within separate halls, which would be located far enough from the machine hall to minimise interaction of the stress fields surrounding these excavations. Similarly, the downstream surge chambers where to be located as far away from the other major excavations as possible.

2. The initial design of the roof of the machine hall would be based upon a trapezoidal section (similar to that used at Poatina). The purpose of this shape is to limit the roof span in the horizontally bedded sequence by means of haunches. In order to check the validity of this concept to the rock mass conditions at Drakensberg and to permit the evolution of a rational roof support system, based upon the use of tensioned reinforcement and pneumatically applied concrete (PAC), it was proposed that a machine hall test enlargement should be excavated during the exploratory phase of the project. This test excavation would be

Project Name	Machine Hall		Rock Condition
	Dimensions LxWxH (metres)	Depth (metres)	
Drakensberg (Natal RSA)	193 x 16,3 x 45	150	Horizontal series of sandstones and siltstones and mudstones
Poatina (Tasmania, Australia)	92 x 13,7 x 26	152	Horizontally bedded mudstone. Horizontal stress approximately twice vertical.
Portage Mountain (B.C. Canada)	271 x 20,4 x 44	61	Interbedded sandstone, shale and coal measures dipping 15°. Horizontal stress approximately twice vertical.

Table 1.

carefully excavated in stages, monitored and the results used to ascertain the optimum roof shape and support system.

3. In order to take advantage of potential cost savings of using concrete instead of steel lining for the lower portion of the penstocks, it was proposed that a penstock test chamber should be constructed during the exploratory contract.

   This chamber, which would be lined in the same manner as the lower portion of the penstocks would be fully instrumented and tested to the full hydraulic head of the Scheme to check the interaction of the lining and the surrounding rock mass with respect to deformation and leakage.

In addition to the specific test openings mentioned above, a comprehensive geological and geotechnical investigation and testing programme were developed to determine conditions throughout the Scheme.

3. Exploratory Stage

3.1 Introduction

The purpose and scope of the various elements of the exploratory programme were established initially in terms of the design objectives. Adequate flexibility was built into each element to allow progressive refinement during the exploratory phase.

The following main factors were considered:

1. Geology

2. Groundwater

3. Mechanical characteristics of the rock mass.
    a. Sample testing
    b. In-situ testing

4. Rock reinforcement testing.

5. Evaluation of exploratory works.

These studies together with the machine hall test enlargement and the penstock test chamber are described in the following section of the paper.

The geological and groundwater conditions have been summarised earliest in this paper. The broad scope of the investigations are described in the following section for completeness.

The investigation and testing programmes were carried out to satisfy the following two main requirements:

1. Classification of stability/support conditions in tunnels and minor openings

2. Specific design of major halls, blocks and intersections.

For the first requirement, extensive use has been made of simple index tests on core to supplement geological data. For the second requirement more elaborate tests have been carried out at a representative scale and at locations relevant to the major works.

3.2 Geotechnical investigations

Figure 4 shows the locations of boreholes which were drilled primarily during the exploratory contract to investigate both the general geological conditions over the entire alignment and the detailed conditions in the vicinity of the major excavations. The rocks were classified according to argillaceous content as illustrated on Figure 5.

Most of the boreholes were drilled with 54 mm diameter double tube coring equipment. The core was generally wrapped in aluminium foil and waxed immediately upon extrusion from the barrel in order to minimise deterioration of due to weathering prior to testing. Permeability testing was carried out in representative boreholes.

Detailed geological logging was carried out on all core. An example of a typical geological log is given in Figure 6. An expanded scale of logging was used in the vicinity of the major cavern roofs.

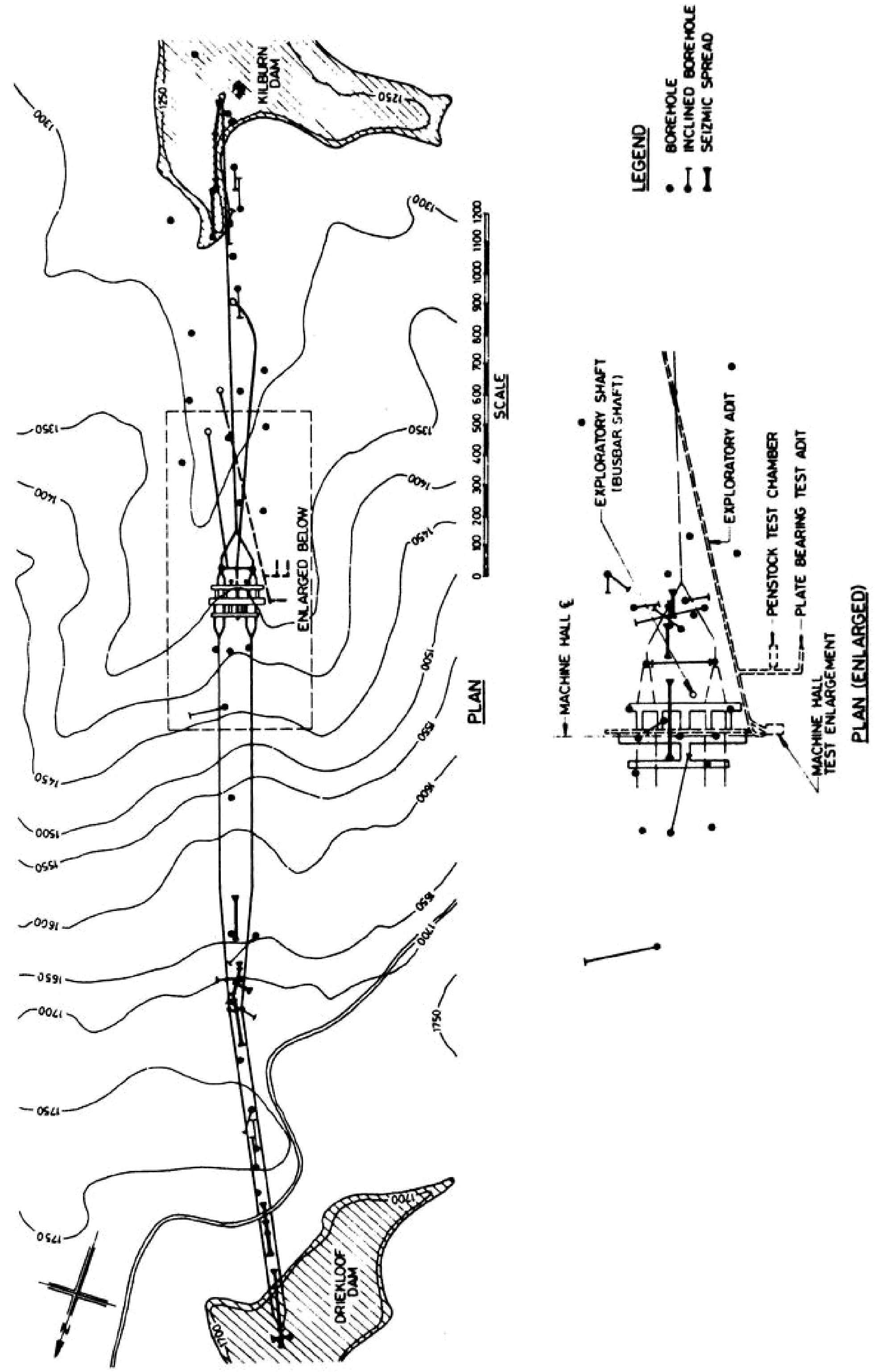

Figure 4. Location of exploratory boreholes and geophysical investigations

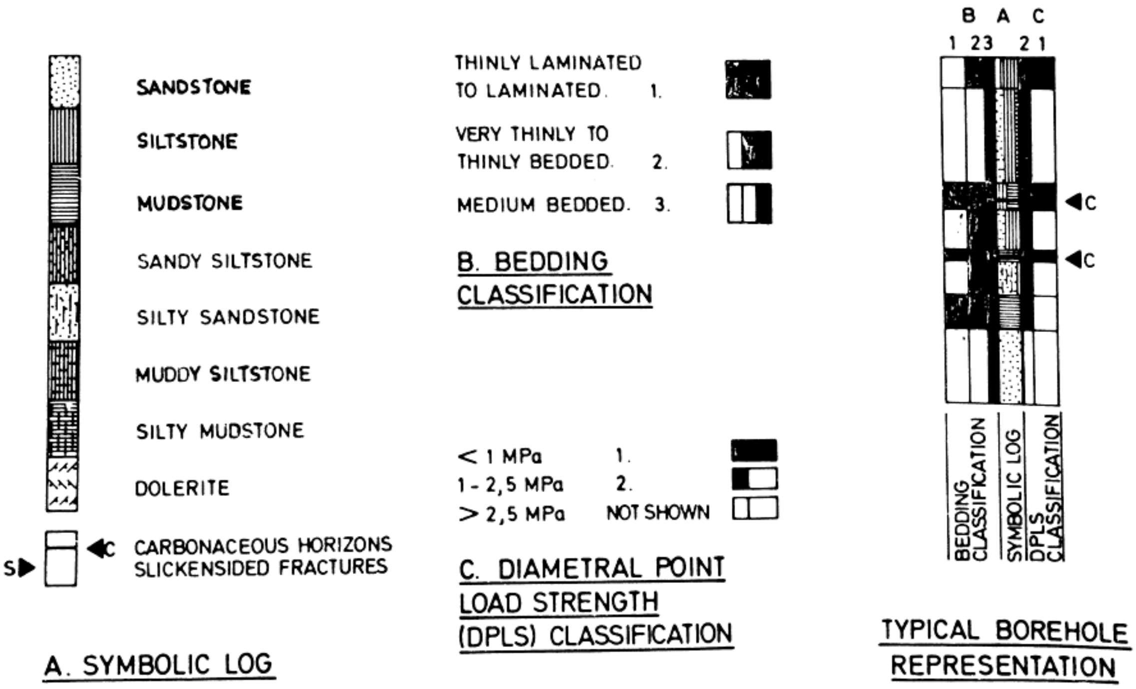

Figure 5. General rock classification scheme for tunnel roof conditions

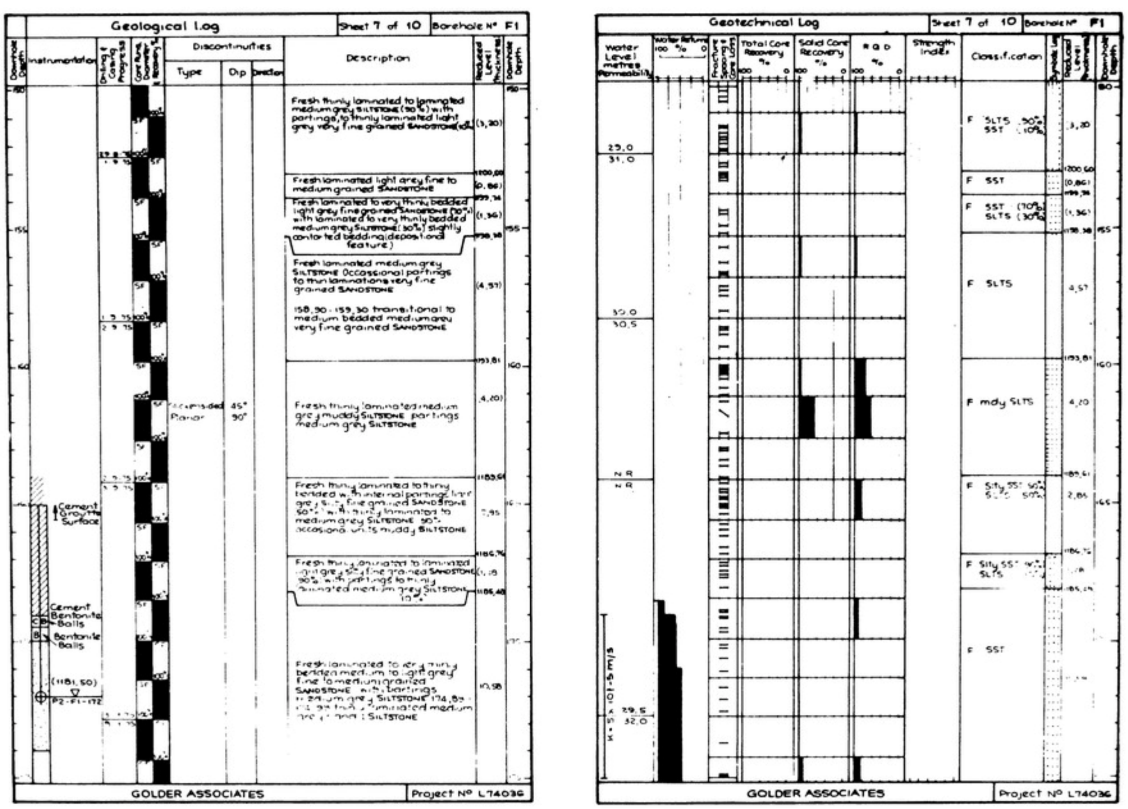

Figure 6. Typical geological and geotechnical logs

High quality colour photography of all core prior to testing was carried out in order to provide a permanent record.

Geophysical investigations were carried out along traverses as indicated in Figure 4 to check the depth of surface weathering and to identify anomalous zones between borehole locations. Surface reflections, refraction, up-hole and cross-hole shooting seismic techniques as well as magnetometer surveys were used. The measured seismic velocities were high for the rock types encountered (except in anomalous zones), an indication of the relatively unfractured nature of the in-situ rock mass. Magnetometer surveys proved to be useful in the location and mapping of dykes along tunnel alignments.

Sample testing

Tests on core samples were carried out for two main purpose:

1. To provide representative ‘intact’ strength and modulus values for the various rock types.

2. To provide index values for classification of stability/support conditions in tunnels and minor openings.

Small scale sample test results were not used as a basis for design of the major openings.

Early studies of various types of index tests indicated that the Point Load Index for core loaded diametrically (parallel to bedding) gave the most representative mechanical index for roof stability conditions.

This test was therefore carried out as a general index test for the entire works and also incorporated in to a Roof Stability Index for tunnel (limited span) sections. Correlation with lithology and actual roof conditions in the tunnels allowed three classes of diametrical point load strength values to be defined (Figure 5).

The susceptibility of the siltstones and mudstones to deterioration on exposure was studied using the Slake Durability Test. A series of visual tests, using dye penetrants on samples subjected to various wetting and drying cycles, was also carried out.

Uniaxial compression testing of cores with modulus measurements using the mechanical callipers was carried out on a large number of representative specimens. Results form such tests were correlated with lithology, inclination of bedding to the core axis and other parameters. Expected lower values of strength for increase in argillaceous content and at critical inclinations of the bedding to the core axis were determined. The sample preparation and testing techniques used were relatively simple and the results should be considered as ‘index’ values rather than absolute parameters.

A series of carefully controlled uniaxial compression tests on selected core samples from the various lithological groups was also carried out to determine laboratory values of strength, modulus and Poisson's ratio. Modulus and Poisson's ratio measurements were made using strain gauge techniques.

Shear testing of mudstone samples and specific bedding planes and joints was also carried out.

3.4 In-situ testing

The objective of the in-situ testing programme was to provide definitive design data for the major underground openings. The following tests were carried out:

1. Measurement of the in-situ stress field.
2. Measurement of rock mass moduli for strata within which the major caverns would be excavated.
3. Measurement of rock mass moduli for strata surrounding the penstock test chamber.

3a. In-situ stress determination

Measurements of in-situ stresses are necessary in order to determine likely stresses and displacements induced in the rock as a result of excavation. All such measurements were therefore concentrated in the vicinity of the power station. The objective of the tests was to provide average stress values in the vicinity of the main halls and in particular ratios of principal stresses.

It was recognised that stress measurements in the rock mass at Drakensberg would be difficult owing to the weak nature of the rock and the relatively limited depth below surface of the excavations. The CSIR triaxial strain cell overcoring method was considered to be the most suitable. The disturbed nature of the excavation sufaces did not favour the use of flat jacks (stress relieving slots) and bored raises were not available for larger scale over-coring methods.

Following an initial test to check equipment operation, a two stage testing programme was drawn up and carried out by the CSIR. The scope of the second stage was based on the first stage results.

Only tow measurements out of a possible total of ten yielded correlatable results from the first stage testing. The other tests were discounted due to flooding, lack of gauge adhesion or breakage of the overcored section. Preliminary results indicated the minor principal stress to be vertical and slightly greater than the overburden stress. The ratio of horizontal to vertical stresses (horizontal stresses about equal) was approximately 2.5:1.

The second stage of tests has been carried out but results are not yet available.

3b. Rock mass modulus determination – power station

Rock mass moduli in the vicinity of the power station were determined by Plate Bearing Tests at representative locations together with Borehole Modulus tests (Goodman Jack) in a number of boreholes in the roof and walls at the proposed machine hall location.

Plate Bearing Tests were carried out in exploratory headings at locations shown on Figure 7. The locations corresponded to the roof strata encountered in the machine hall test enlargement and the proposed machine hall enlargement was excavated prior to a final decision on the machine hall elevation). The test equipment is shown typically on Figure 8. A loaded area of 1.00 m^2 with a corresponding contact stress of up to 4.5 MPa was used. The maximum stress was determined following consideration of the stress changes induced during excavation.

Tests were carried out parallel and normal to the bedding. Rock mass displacements were measured relative to the loading plate on the loading axis. Three point extensometers were used at depths up to 6 m. Several loading cycles were carried out (generally 5) and a short term creep test was carried out at the maximum loading.

A typical test result is shown on Figure 9. The influence of rock relaxation at limited depths is clearly indicated. Modulus values are corrected to allow for the confining effect of the heading using results from a three dimensional boundary integral equation method study (5).

Bore hole modulus tests using a Goodman Jack were carried out in vertical boreholes into the proposed machine hall roof strata. The boreholes were spaced along the length of proposed machine hall as illustrated in Figure 7. Tests were carried out on representative strata identified during detail logging. Other tests were also carried out in the vicinity of the larger scal Plate Bearing Tests to compare the results obtained from each type of test. In addition, borehole jacking tests were carried out in horizontal holes in both a horizontal and vertical sense on selected strata to determine modulus anisotropy.

3c. Rock mass modulus determination - Penstock Test Chamber

Plate bearing tests were carried out in a special test adit alongside the penstock test chamber to determine the rock mass modulus at various points around the chamber. The tests were carried out on the structure representative of those exposed in the chamber excavation. All tests were radial to the surface and indicated in Figure 7.

Tests were carried out using a plate area of 0.5 m^2 and contact stresses up to 9.0 MPa. Such stresses are representative of the induced stress (pressure) changes in the penstocks during operation.

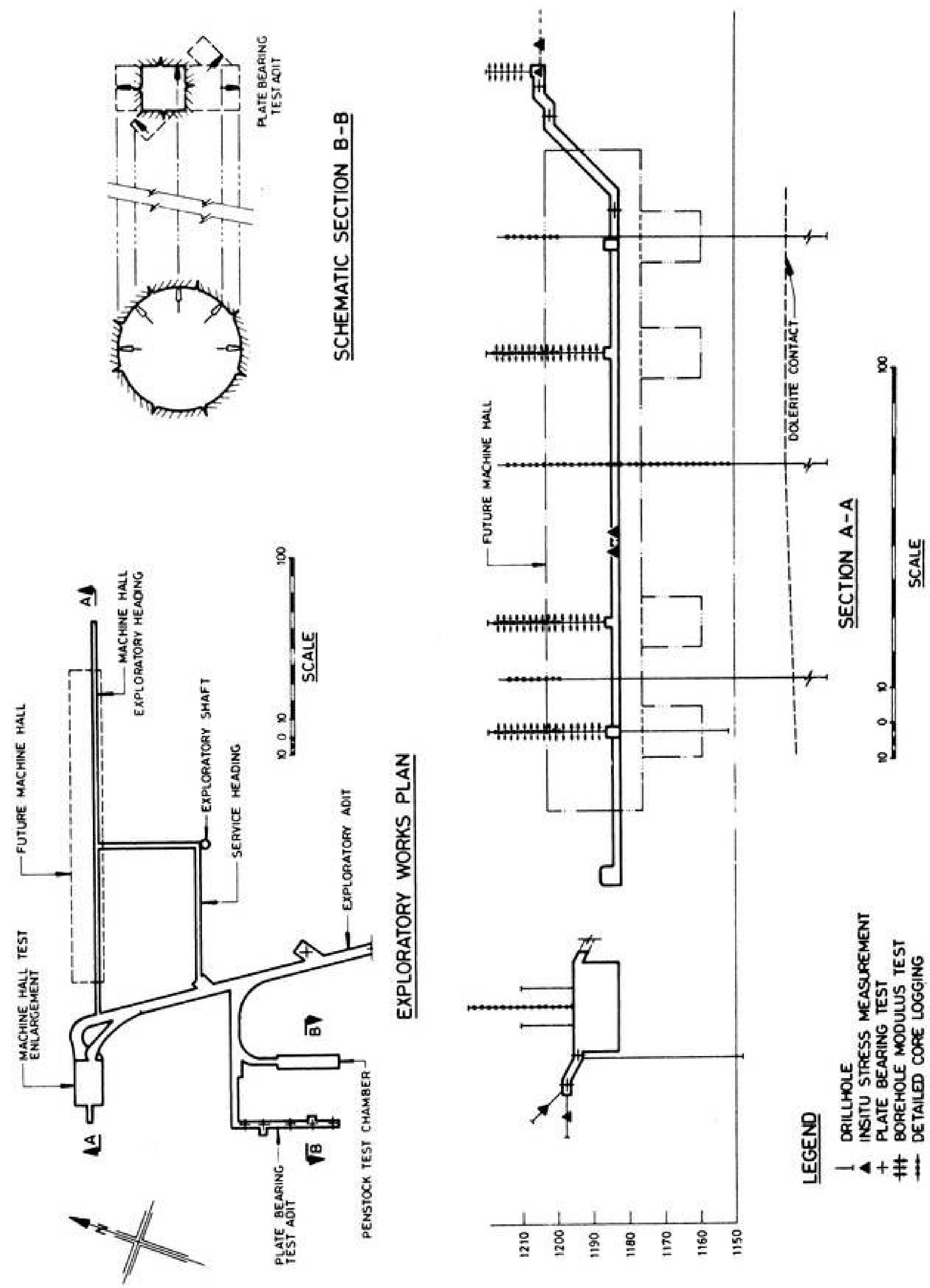

Figure 7. Test locations

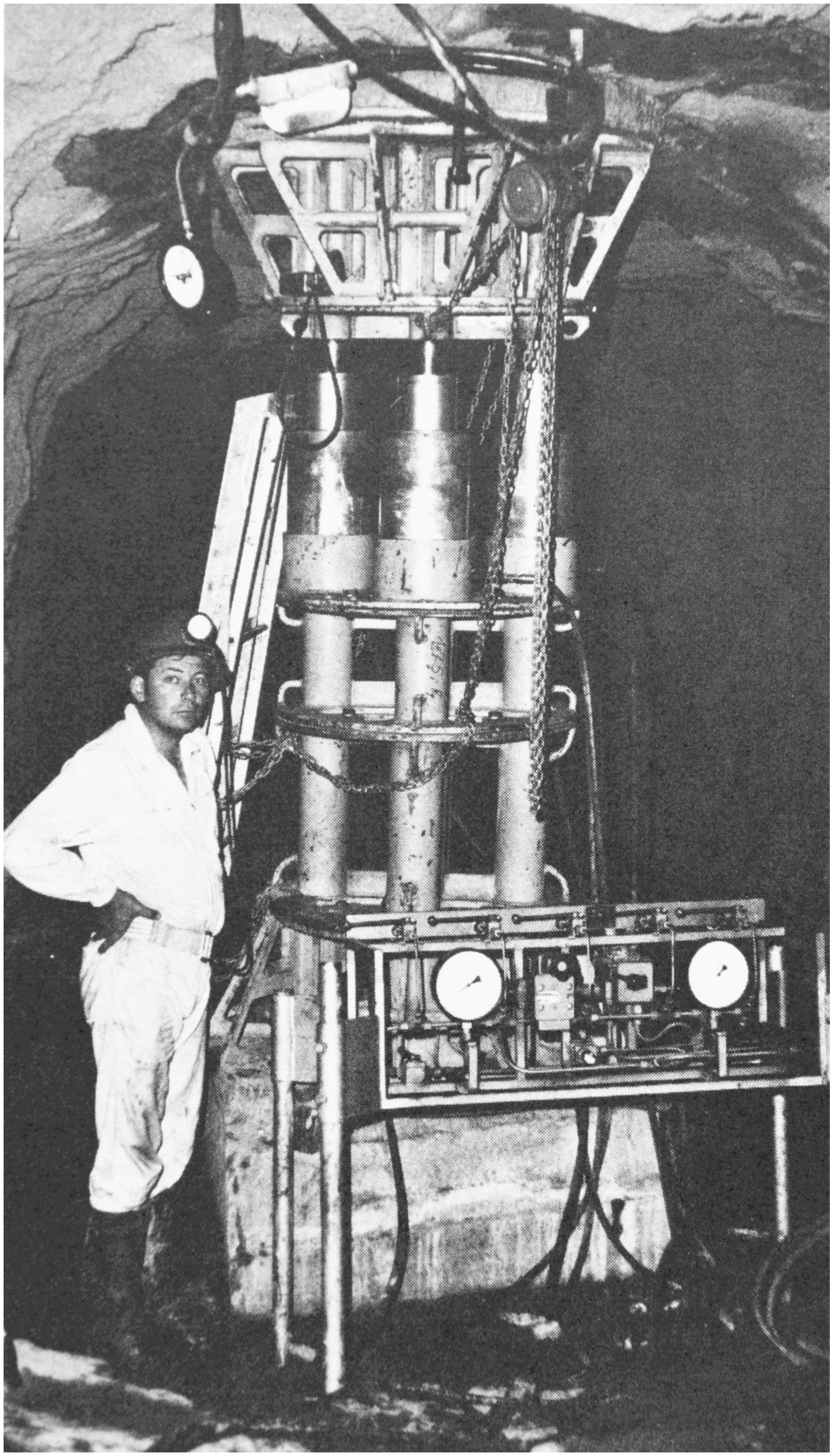

Figure 8. Plate bearing test equipment

The test normal to the invert of the penstock test chamber was repeated following grouting of the rock mass. The grouting process was designed to stimulate as closely as possible the actual grouting technique around the lining of the test chamber

3.5 Rock Reinforcement and Crane Beam Anchor Testing

Specific test were carried out on the type of reinforcement proposed that the main halls. It is intended that resin anchored bolts with adequate or continuous (threadbar) nut adjustment will be used. Secondary bonding of the free length after completion of final tensioning will be affected probably using concrete grout.

It is intended that the crane beams in the machine and valve halls will be supported by stressed rock anchors, each with a working load of 90 tons. Tests will be carried out to determine the anchorage characteristics of the rock strata in which the anchors will finally be located. A cement grouting anchorage will be used. During the tests each anchor will be fully loaded and unloaded several times and in addition short term creep tests on anchorages of varying length will be carried out.

3.6. Evaluation of PAC (Pneumatically Applied Concrete)

As currently proposed all excavations will be permanently lined with mesh reinforced PAC other than where placed concrete lining is required for hydraulic or internal structure of reasons. The PAC lining will be applied generally in two stages with mesh reinforcement being placed after the initial PAC application. Apparently PAC as used in the exploratory contract and preliminary contract works has been evaluated.

PAC spalling has been recorded and the results related to the particular rock strata/bedding weakness. Only limited trials using mesh reinforcement have been carried out. The following conclusions involving the use of PAC for the permanent lining of rock strata such as encountered at Drakensberg are as follows:

1. The rock surface should be sound (controlled blasting an adequate scaling) an inspection immediately prior to PAC application is required.

2. Specialised PAC mixing and application equipment capable of spraying PAC at specified distance from the face are required. This implies equipment with considerable nozzle/operator reach but the main halls.

3. Only trained operatives with adequate experience should be used.

4. Adequate peeling back of mesh to the first PAC layer to ensure minimal clearance is required.

Since the process of PAC application is highly dependent upon the method used, final trials will be carried out after the main contract award.

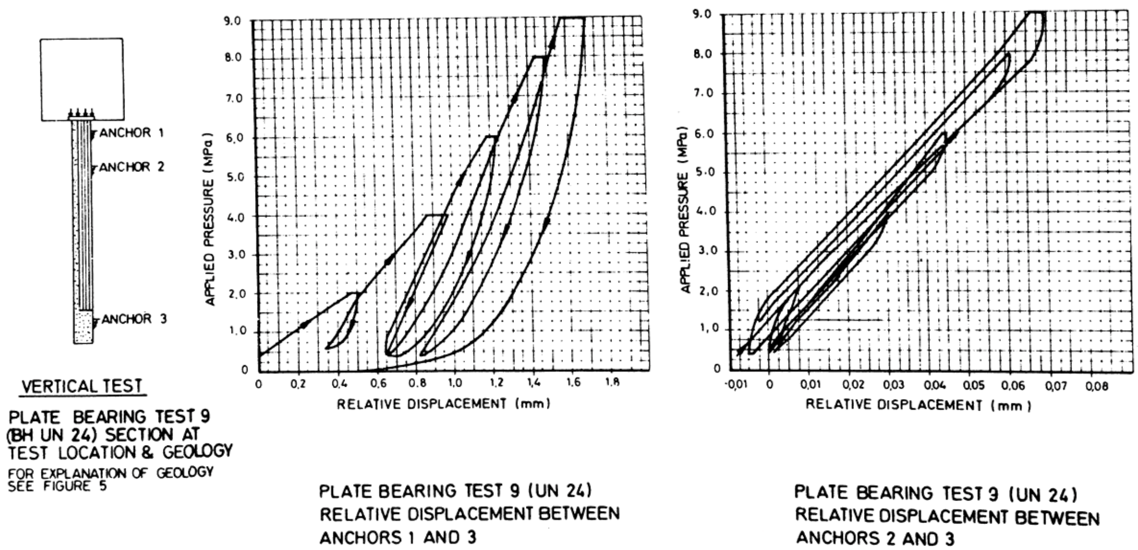

Figure 9. Typical plate bearing test results

4. Machine Hall Test Enlargement

The purpose of the machine hall test enlargement (MHTE) can be summarised as follows:

1. To confirm that the roof span required for the main halls is feasible.

2. To demonstrate that haunches (required to limit the span) can be effectively excavated and reinforced.

3. To determine the necessary level of reinforcement for the roof span.

In order to develop a realistic loading condition for the roof span, the effect of sidewall excavation was simulated by excavating slots to a depth equal slots to a depth equal to about half the final sidewall height. The principal stages of excavation are shown on Figure 11.

The influence of excavation to the full cross-section on the stresses in the roof strata will be determined by a staged stress analysis using both the results of monitoring from the actual test excavation and the results from the in-situ stress measurements and rock mass properties.

The elevation of the MHTE was chosen such that the haunches and roof were positioned in the weakest possible strata (as determined from the drilling from surface) for the 15 m range in possible level for the proposed machine hall. This level was only finally selected in July 1976, based on the required setting for the pump-turbines. The axis of the MHTE is parallel to the proposed machine hall.

The length of the MHTE (approximately 1.5 times the span) was chosen to allow for a central 10 m section which would be relatively unaffected by end constraints.

The excavation sequence for the test enlargement was slightly more complex than that for the proposed machine hall (5 slices instead of 3) because of the need to install crown instrumentation prior to significant rock deformation.

Reinforcement/support was provided by two means:

1. Rock bolts estimated on the basis of precedent and simple numerical analyses

2. Temporary hydraulic props along the centreline of the enlargement (5 x 100 ton capacity).

The purpose of the temporary props was to provide a calibrated stiff support along the centreline as well as a temporary support during excavation. Prior to increasing the span at a given stage the props were set to a nominal load (10 ton) and the increase in load with span monitored.

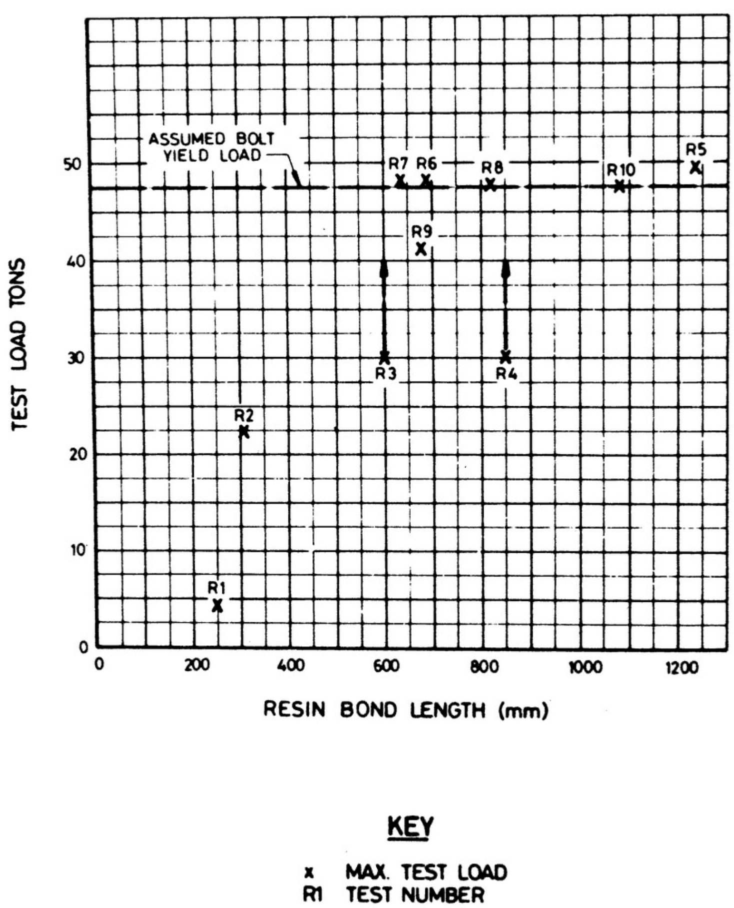

Figure 10. Rock reinforcement test results

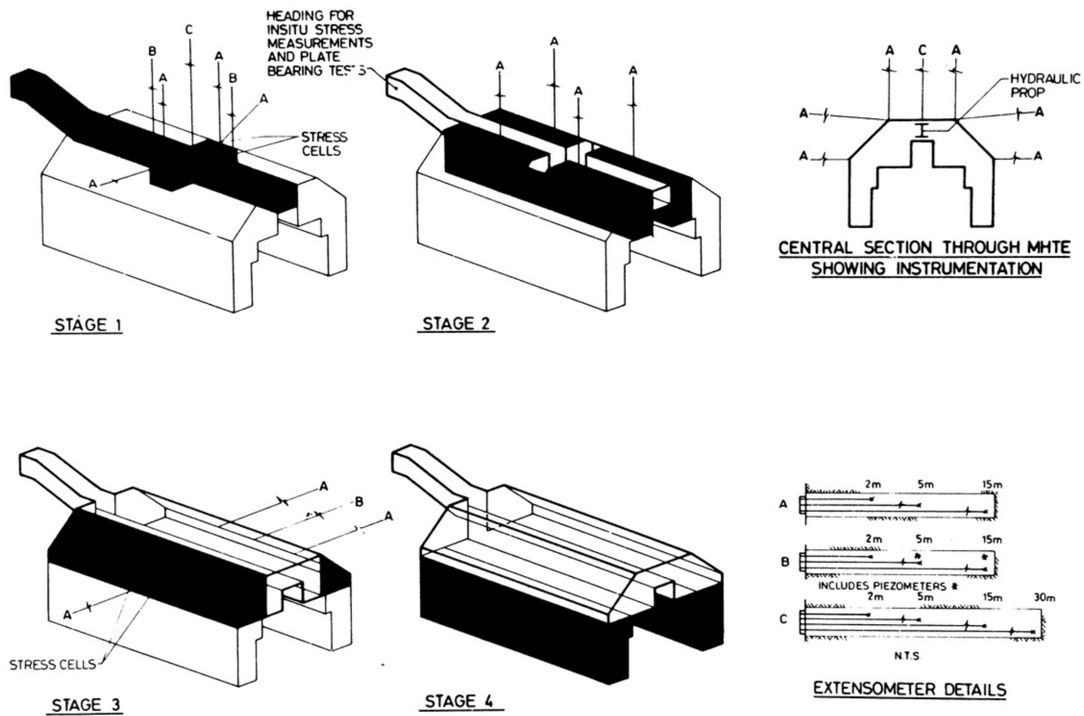

Figure 11. Machine hall test enlargement: stage excavation

The load carried by the props was then distributed to the primary reinforcement as an installed 'calibrated' load and the props subsequently unloaded. It was thus possible to observe potential support requirements under acceptable displacement conditions (as monitored by the extensometers). Unfortunately, the excavation of Stage 2 (Figure 11) caused considerable damage to the pillar supporting the props and the stiffness of the pillar was significantly reduced. Following an evaluation of this condition, additional load was placed in the primary reinforcement for safety reasons.

The instrumentation comprised the following types:

1. Displacement monitoring (multi-point rod extensometers, convergence measurements, precise levelling)

2. Stress change monitoring (embedded mercury filled stress cells)

3. Piezometric monitoring

4. Load monitoring in reinforcement and central support jacks.

The instrumentation was primarily located on three sections; a central section and sections spaced 5 m either side. Typical layouts are shown on Figure 11.

Displacement monitoring using the extensometers proved to be reliable except where blasting damage occurred. Typically a resolution of 0.01 mm was achieved and both elastic and irrevocable deformations were monitored. The monitoring results at each stage were compared with displacement data predicted prior to excavation from finite element analyses.

A typical plot of displacement versus time/excavation stages is shown on Figure 12. both closure measurements and precise levelling were an order of magnitude less accurate than the extensometers.

Stresses monitored during the excavation stages were very sensitive to changes in excavation geometry and could be usefully correlated with the displacement records (Figure 12).

Piezometric monitoring in the roof strata indicated relatively high pressure (up to 15 m head) relatively close to the excavation face. These measurements further indicated the low transverse permeability of the siltstones. Consideration of likely loadings due to groundwater pressures on roof strata will be made during the design stage.

Load monitoring in primary reinforcement has yielded few results to date (end Stage 3) as deformations have been small since the reinforcement was installed (Figure 12). Significant load increases (of the order of 40 tons) were observed in some support props during Stage 3 excavation. These load increases occurred where the pillar was relatively undamaged and were used in assessing Stage 3 primary reinforcement loads. At all stages of the development a continual evaluation of displacement, stress and load changes was carried out.

At the time of writing Stage 4 (sidewall) excavation remains to be completed.

5. Penstock Test Chamber

The penstock test chamber has been constructed to check the suitability of concrete as a lining for the pressure tunnels. The chamber has been located in an area representative of the weaker rock conditions expected along its alignment. Attention was given in the choice of site to locating the chamber in strata having a considerable modulus variation over the height of the chamber.

A principal objective of the test was to determine the relative behaviour of the concrete lining, grouted rock and surrounding rock mass to demonstrate that an effective transfer of stress into the rock will occur under acceptable deformations and leakage.

The chamber was concreted in three bays approximately 10 m long having a finished internal diameter of 5.5 m and a nominal lining thickness of 0.6 m. the three bays are separated by conventional waterstops. Each end of the chamber is terminated by a concrete plug.

The central bay is considered to be representative of tunnel operating conditions and contains all the instrumentation. The instrumentation is arranged on 5 sections as illustrated in Figure 13 with particular emphasis on monitoring the central section.

The principal instrumentation can be summarised as follows:

1. Diametral changes across the tunnel section monitored by means of internal closure measurements

2. Radial deformation of the lining, grouted rock and surrounding rock mass monitored by means of multi-point borehole extensometers which are tied in with the diametral closure measurement (1)

3. Strains in the lining measured by means of embedded and surface vibrating wire gauges (radial and tangential strains monitored)

4. Stresses across the lining/rock interface monitored by means of mercury filled embedded stress cells (radial and tangential stresses monitored)

5. Water pressures behind the lining and in the surrounding rock mass monitored by means of hydraulic piezometers.

Remote read-out of instrumentation under high fluid pressures (up to 7.5 MPa) necessitated special developments particularly of remote sensing elements. Temperatures are monitoring at various points in the test zone and invar rods/wires are used for reference displacement monitoring. Stress cells are designed to allow for compensation from within he chamber.

As previously discussed a series of plate bearing tests was carried out to determine radial modulus conditions within the rock mass as well as the effect of grouting.

High pressure grouting behind the chamber lining will be monitored by means of the installed instrumentation. Particular attention will be paid to uniformly of deformations and stresses indicating that a complete grout ring has been established. Residual stresses induced in the lining will be carefully monitored.

The chamber pressurization programme has not yet been finalised. It is intended however to pressurize in stages and carry out tests at constant pressure as well as under cyclic conditions.

A theoretical study of likely deformations/stresses will be finalised prior to commencement of testing. The test chamber is located at a shallower depth than the proposed pressure tunnel alignments. It will thus be a valid acceptance test from overall pressure/rock cover considerations.

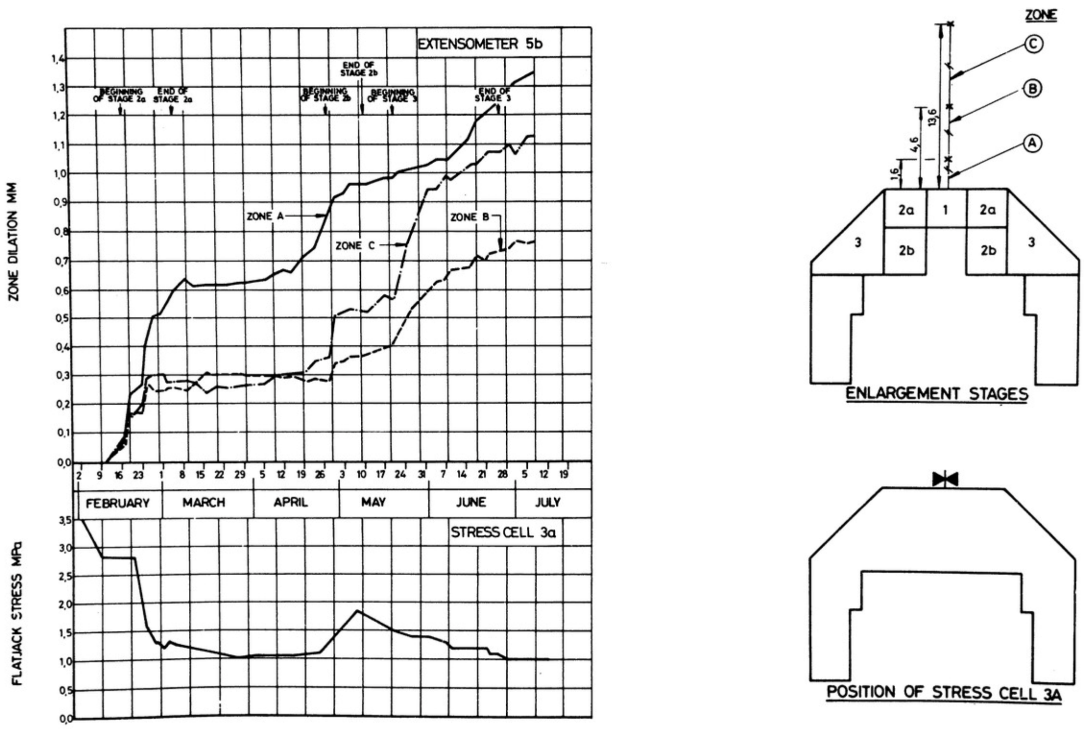

Figure 12. Typical roof displacement and stress change characteristics
– machine hall test enlargement

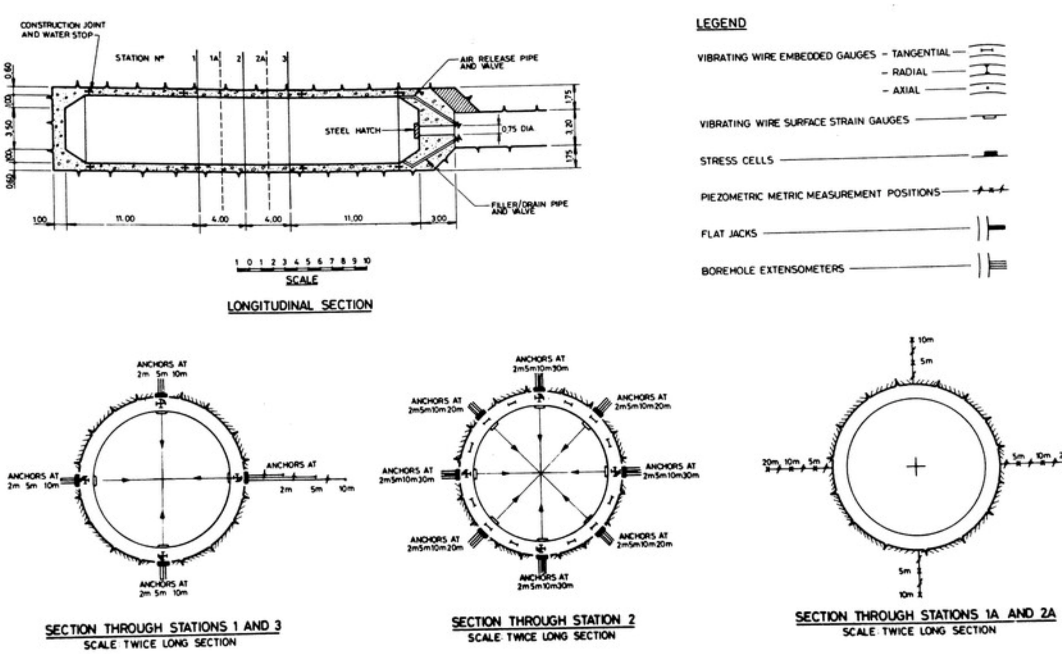

Figure 13. Penstock test chamber

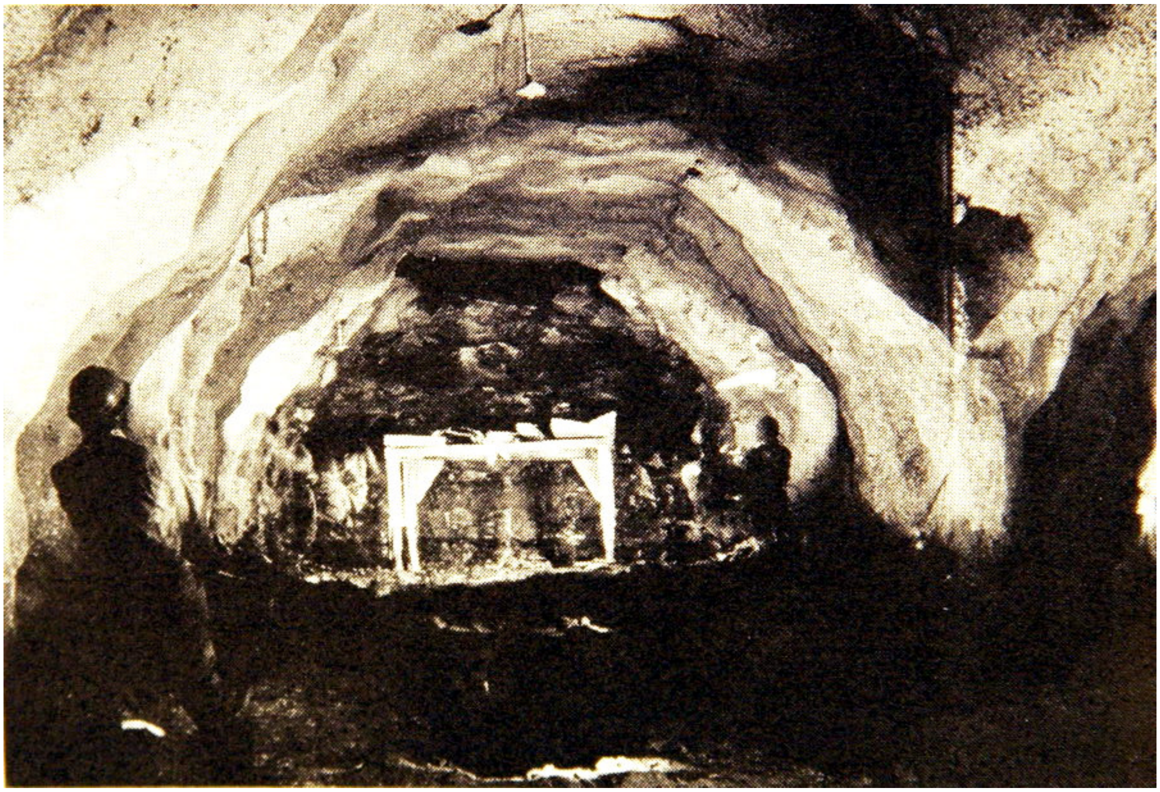

Figure 14. Penstock test chamber – excavation of upper bench

6. Proposed Design Studies

The design process for the excavation and reinforcement requirements of the main underground works was initially considered during the formulation of the investigation and testing programme which was consequently arranged to provide relevant parameters for the design requirements.

As already outlined, considerable attention has been paid to testing rock conditions at a representative scale. Both the machine hall test enlargement and penstock test chamber are full scale tests. The information obtained from the investigation and testing stages will e used primarily to extrapolate observed underground conditions to the overall station layout using recognised stress analyses techniques and a structural evaluation of the rock mass. Optimisation of the detailed underground layout and rock reinforcement with respect to localised geological conditions will then be carried out.

7. Principal design aspects are as follow:

8. Excavation method to achieve specified profiles

9. Shape and precise elevation of major roof spans taking into account predominant bedding features, haunch geometry and geology.

10. Reinforcement of major roof spans including haunches

11. Reinforcement of sidewalls

12. Excavation sequence for main halls

13. Excavation sequence, shape and reinforcement of major intersections

14. Reinforcement of minor galleries and tunnels in the vicinity of the main works

15. Excavation sequence, shape and reinforcement of surge chambers

16. Penstock linings (both steel and concrete)

17. Drainage of the rock mass in the vicinity of the penstocks and upstream wall of the valve hall

18. Rock reinforcement details (inclination, timing, installation, tensioning, secondary grouting, corrosion protection)

19. Crane beam anchors (installation, tensioning, grouting, corrosion protection)

Principal methods of analysis are as follows:

1. Two and three dimensional stress and deformation analyses finite element and boundary integral equations methods)

2. Kinematic check of prevailing geological structure to identify potential failure modes caused by movement of blocks of rock

3. Analysis of structurally controlled failures taking into account rock stresses and reinforcement loadings (production of detailed reinforcement requirements and likely rock deformations).

A final design will be evolved form the results of the full scale tests, an evaluation of the rock mass characteristics and an analysis of the final station layout in terms of the measured parameters.

The inter-relationship of investigation, testing an analysis and design for the underground works at Drakensberg is summarised on Table II.

KEY:
○ - MINOR INFLUENCE
● - MAJOR INFLUENCE

	GEOLOGY GROUNDWATER			SAMPLE TESTING					IN SITU TESTING						FULL SCALE TESTS		ANALYSIS		
DESIGN	GEOLOGICAL INVESTIGATIONS	PERMEABILITY MEASUREMENTS	GW PRESSURE MEASUREMENTS	POINT LOAD INDEX TEST	UNIAXIAL COMPRESSIVE STRENGTH TESTS	MODULUS DETERMINATION	SHEAR TESTING	DURABILITY TESTING	INSITU STRESS MEASUREMENTS	ROCK MASS MODULUS PLATE BEARING TEST	ROCK MASS MODULUS BOREHOLE JACKING TEST	ROCK REINFORCEMENT TESTING	CRANE RAIL ANCHOR TESTS	PAC EVALUATION	MACHINE HALL TEST ENLARGEMENT	PENSTOCK TEST CHAMBER	STRESS / DEFORMATION ANALYSIS	KINEMATIC CHECK	STABILITY ANALYSES REINFORCEMENT DESIGN
EXCAVATION METHOD / PROFILE	○							○						○	●				
SHAPE/ELEVATION MAJOR ROOF SPANS	●			○	○	○	○		●	●	○				●		●	●	●
REINFORCEMENT MAJOR ROOF SPANS AND HAUNCHES	●	○	○	○	○	○	○		●	●	○	●	○	●	●		●	●	●
REINFORCEMENT OF SIDEWALLS	●	○	○	○	○	○	○	○	●	●		●	●	●	○		●	●	●
EXCAVATION SEQUENCE FOR MAIN HALLS	●								○	○		●	○	●	●		●	○	●
EXCAVATION SEQUENCE, SHAPE AND REINFORCEMENT · INTERSECTIONS	●			○	○	○	○		●	●		●	○	●	○		●	●	●
REINFORCEMENT OF TUNNELS IN VICINITY OF MAIN WORKS	●	○	○	●	○	○	○	○	○										
EXCAVATION SEQUENCE, SHAPE AND REINFORCEMENT · SURGE CHAMBERS	●		○	○	○	○	○	○	●	●	○	●		○	○		○	●	●
PENSTOCK LININGS	●	●	●	●	○	○	○	○	○	●		○		○		●	○	○	○
DRAINAGE OF PENSTOCK AND UPSTREAM WALLS OF HALLS	●	●	●						○	○				○	○	●	○		
ROCK REINFORCEMENT DETAILS	●					○		○		○	○	●		○	●	○			
CRANE BEAM ANCHORS	●					○		○		○	○	○	●						
PAC AND MESH REINFORCED PAC	●		●	○		○		●	○	○		○		○	●	○			

Table II. Inter-relationship of investigation – testing – analysis – design

Conclusions

The determination of rock mass properties at Drakensberg in relation to the design of underground excavations has been reviewed. The investigation methods and testing techniques have been outlined and the reasons for their selection given.

The particular role of full scale testing has been highlighted. The relevance of testing at other scales, both in-situ and on a sample scale, has also been discussed in relation to the final design process.

The current status of the investigation/testing works precludes the reporting of detail results at this stage. A full evaluation of all data is expected by early in 1977.

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