GEOLOGICAL CONDITIONS AND ENVIRONMENTAL IMPACT OF THE MOHALE DAM, LESOTHO HIGHLANDS WATER PROJECT

By

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Abstract

A study of the roles played by geological forces in shaping man's interaction with the earth!

The Mohale Dam forms the second major dam to be built in Lesotho and is the main element in the Phase 1B of the Lesotho Highlands Project (LHWP). This 145-m high concrete faced rockfill dam (CFRD) is situated on the Senqunyane River immediately downstream of the confluence with the Likalaneng River. The Mohale Dam basin is connected to the Katse Dam Basin by the 32-km long Mohale Tunnel, which has been constructed under separate contract. The geotechnical investigations were undertaken in order to decide on whether the Mohale Dam should be a concrete faced rockfill dam instead of a roller compacted concrete (RCCD) or concrete arch dam (CAD).

Some of the reasons for selecting a CFRD have been the fact that such dams are very stable during earthquakes, as these are always anticipated for large dams; the availability of the construction materials, in this case the basalt rock sourced from the 2 quarries, located within 2 km of the embankment; the CFRD is less prone to construction time –delay risks than other types of dams, such as adverse weather conditions (The time schedule for construction for Mohale is five years) and the location of haul roads and quarries which minimized the environmental impact of the construction on the surrounding area. The availability and proximity of large quantities of rock played a pivotal role in the successful completion of the Mohale Dam. A broad ridge of basalt at the confluence of the Senqunyane and Likalaneng Rivers has been used as the main quarry. A broad colluvium filled oxbow in the quarry area made it unique because the bulk rock and the impervious clay materials have been mined from the same area and the same haul roads have been used for transportation. The impervious clay has been used for construction of both temporary and permanent cofferdams. It has also been used for a blanket purpose at the upstream toe of the dam. A 70-m thick coarse-crystalline jointed doleritic olivine basalt formed a secondary quarry on the steep slopes of the Likalaneng River valley. The rock has been mined for the purpose of producing a concrete aggregate for the dam and other concrete structures and for filter layers for the Mohale Dam. Another advantage of the CFRD is the fact that deep foundation excavations can be avoided but not compromised. Many outcrops in the areas have led to the plinth structure, which forms a contact between the rock foundation and the concrete face, being founded at shallow depths and hence avoiding high valley bottom stresses. The plinth structure has been placed on jointed competent doleritic basalt in the river. Consequently, there has been no deep excavation. Intrusive structures of mixed non-amygdaloidal basalt underlie the entire left abutment. A zone of breccia separates the basalt from the doleritic basalt. A typical sequence of basalt underlies the right abutment. There are some lineaments crossing the dam. These

lineaments have been treated with reverse filters and in some cases; joints have been performed in the plinth.

Two tunnels have been constructed on the left abutment for diverting the river. The provision of these tunnels is needed to allow construction to proceed in dry conditions. The concrete lined spillway is located on the right abutment. The purpose of this spillway is to allow anticipated floodwater to flow safely downstream when the reservoir is full. Topography has played an important role in keeping the spillway excavation very shallow.

The dam has required 7.8 x 10^6 $m^3$of material. The embankment of the Mohale Dam is a zoned rockfill dam. The rock has been mined from the two quarries mentioned earlier. The bulk of rock has been transported utilizing a fleet of 50-ton caterpillar 733 off-road dump trucks along access roads built to provide the most efficient trips from quarries to the dam. The development of the quarries has been organised in such a way that it has been the most economical in terms of transport costs and otherwise. The construction of roads has been ongoing as to the construction of the dam progresses. The bulk rock has been placed in 1 m layers on the upstream side of the embankment and in 2 m layers on the downstream. The graded filter layers below the face slab have been placed in 400 mm-thick layers, extending variably up to 4 m wide. The concrete face slab varies in thickness from 720 mm at the base of the dam to 300 mm near the crest. The upstream slope is built at a gradient of 1v: 1h, while the downstream slope is built at an overall slope of 1v: 1.45h, which includes the permanent access road down the face.

The Mohale Tunnel is located approximately 5 km upstream of the Mohale Dam wall. It comprises two intake structures: the lower and the upper intakes with operating levels at 2005 and 2050 m above sea level respectively. The tunnel has been excavated utilizing two tunnel-boring machines (TBMs): one from the intake side while the other advances from the outlet side.

The approach that has been adopted towards the environment aims at minimizing the adverse impacts on the Senqunyane River System while capitalizing simultaneously on the positive ones with the incorporation of the best compensation policies for the affected persons of the area. The adverse impacts put into perspective are such as, seismic activities possible during impoundment of any reservoir, possible breaching of the dam that can cause huge loss of human life and environmental damage; loss of land with its valuables due to the reservoir; permanent hazard posed by the reservoir; change in ground water behavior; displacement of communities and introduction of diseases. The positive impacts are introduction of all sorts of infrastructure (roads, clinics, shops, schools; etc.), provision of much needed jobs for poverty alleviation; increase in standard of education; and tourism boom. The environmental policies have been constructed in line with the world's experiences from other similar projects of the LHWP magnitude. While on the other hand taking cognizance that no two projects are the same, each is unique in its own right, the projects are governed by geological environment and socio-political situations of each country, specifically of the site of construction. The LHWP has tried very hard to always engage the civil society from all walks of life, in order to make it possible to achieve the LHWP goal, which has been to build dams and tunnels. However, the main challenge is to preserve the state of the environment during and after construction and impoundment respectively. The hydrological condition of the Mohale Tunnel during construction is appended in this work to highlight the hydrogeology of the basalt with a view to mapping out the possible impact of the Mohale Dam on to the groundwater regime. Measurements have been taken along the tunnel whenever water ingress has been encountered. The geology has been recorded and correlated with the surface. Spring measurements have also been appended, since they were carried out along the tunnel alignment, i.e. those located within 100 m on both sides of the alignment. Along the Katse shoreline, observations and monitoring of the groundwater behaviour have been undertaken and this has involved interviews with the local communities.

Below is the summary of the activities that this work has tried to cover with a view to realizing the aim and objective of this work:

- Description of work undertaken and terms of reference
- Sources of information (Literature information gathering and discussions with relevant people)
- Description of the site and surrounding area based on an inspection and walkover survey of the site
- Regional and site geology and any feature of the geology and known material properties which will is considered to impact on the project
- Groundwater conditions and hydrogeology/hydrology and climatological conditions
- Records of previous investigations undertaken at or close to the site, if any
- Site history and past uses of the site and adjacent sites (land-use in general.)
- Site characteristics and locality maps (topography/morphology)
- Records of quarrying, landfill, any impacts etc.
- Records of service and underground structures
- Field reconnaissance of the general area of the project noting particularly:
    1. Evidence of groundwater
    2. Location of surface waters (springs/streams/rivers) and evidence of flooding
    3. Behaviour of any existing structures on the site or neighbouring structures
    4. Exposures in quarries and borrow pits
    5. Area of instability (slope stability in rock/soil)
    6. History of the site
    7. Geology of the site (structural, lithological, etc.)
    8. Information from available aerial photographs
    9. Archaeological evidences
    10. Seismological history of the area. Type of field equipment used

- Clear conclusion regarding the conceptual geological and environmental impacts

- Overall strategy and objectives for subsequent intrusive investigation
- Purpose and scope of the investigation
- Names of all consultants and sub-contractors used
- Dates between which field and laboratory work was conducted, whenever possible
- A factual account of all field and laboratory work
- Exploratory hole records (boreholes, trial pits, window sample holes), including grid co-ordinates and ground elevation
- In-situ test results (site laboratory tests, etc.)
- Laboratory test results including any contamination test results
- Results of groundwater level monitoring and any geo-environmental monitoring
- Specialist sub-contract test results, (static or dynamic cone penetration test, geophysics, etc.)
- Site plan showing locations of exploratory holes (drillings, pits, quarries etc.)
- The provision of data in electronic format
- A review of the field and laboratory work
- Detailed description of all formation including their geological context, physical properties and their deformation and strength characteristics
- Comments on irregularities such as contact zones, podsols, depressions, cavities and boulders
- Sub-surface profiles showing the differentiation of the various formations
- Identification of geological, geotechnical or other hazards
- Depth of the groundwater table and its seasonal fluctuations
- The range and grouping of any derived values of the geotechnical data for each stratum
- Summary tables for chemical contamination data with listings of selected assessment criteria, wherever possible
- Tabulation, photographic and graphical presentation of the results of the field and laboratory work in relation to the requirements of the project and if deemed necessary histograms illustrating the range of values of the most relevant data and their distribution
- A review and summary of the derived values of geotechnical parameters
- Any proposals for further field and laboratory work, with comments justifying the need for this extra work and a detailed programme for the extra investigations to be done
- The assumptions, data and method of verifying the safety (environmental impacts) and serviceability of the geotechnical construction (e.g. method of calculation), including:
    1. A description of the site and its surroundings
    2. A description of the ground conditions
    3. A description of the proposed construction, including anticipated loading and any imposed deformations
    4. Site selection criteria
    5. Design values of soil and rock properties, including any necessary explanation for their selection
    6. Statements on any codes or standards used
    7. Statements on the suitability of the site for the proposed construction and the level of risk assumed in the assessment
    8. Geotechnical design calculations, photos and drawings
    9. Design recommendations
    10. A note of any items to be checked during construction or required maintenance or monitoring
    11. A statement of sequence of construction operations envisaged in the design. Alternatively, the design report may state that the sequence of construction is to be decided by the contractor

12. A plan of supervision and monitoring as appropriate for the type of project, stating acceptable limits for the results to be obtained by the supervision and specifying the type, quality and frequency of supervision

- Where appropriate records of preliminary prototype scale testing
- Records of ground conditions encountered
- Test records relating to additional geotechnical testing, materials testing and proof load testing, in particular piles and anchors
- Any non compliance records or similar raised during course of the project
- Monitoring records on the effect of the works on adjacent properties etc. (Environmental Issues)
- Where the observational method was used this shall be fully recorded
- Records of temporary works, particularly where these left in-situ or may affect future developments
- Detail of inspection regime including scope and frequency
- Proposed measures to be taken during the lifetime of the facility for maintenance of drainage, corrosion protection, etc.
- Long-term settlement records, water levels, piezometers, etc.
- Identification of critical elements of the structural system that may pose thread to the safety of the dam.

An appendix on information important structures in the dam and tunnels on the water ingress (groundwater condition of the site as derived from the inflows and springs)

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1 INTRODUCTION

Dams have been constructed for centuries for various reasons ranging from flood management, irrigation, and hydropower production to domestic and industrial water supply. Arguably, half the world's large dams have been built exclusively or primarily for irrigation and the dams have indeed made a significant contribution to human development and the benefits derived have been quite considerable. However, their geotechnical impact on the environment has raised a loud outcry from different sectors of the world population without concrete evaluation or exploration of individual projects. Concerns have recently been raised and have now let to global anti-dam campaigns to the point where future dam construction is in doubt. It is a reality that a dam built across a river indeed brings about certain environmental changes in ecosystems. These changes will be obvious in the river valley upstream of the dam, which will be flooded as the dam impounds a new reservoir behind it. A less obvious feature is that the river downstream will also be significantly changed. Large dam projects such as the Lesotho Highlands Water Project (LHWP) are highly unique in their design, geological setting and the construction materials used to build them. Their environmental impacts are also site specific. Some large dams in the world have created lakes hundreds of kilometres long in areas, which had large local populations. These projects bring about major impacts on the human, animal and plant population of the catchment area. These impacts have been well documented but it would be wrong to make an assumption that all dam projects have similar major impacts on the environment. The environmental impacts are heavily dependent on the geology of the site. This is clearly demonstrated by the huge budget made to buy expertise from all over the world to carry out the geotechnical investigations. From sufficient knowledge of local site conditions, gained from the investigation, the design can follow. These investigative procedures cover those important geotechnical investigations carried out at the LHWP sites and assessments of the geological conditions with respect to the environmental impacts. Specifically, this work deals with the environmental conditions of Mohale Dam and to a lesser extent with those of the Mohale Tunnel. However a detailed feature statistic analysis is herewith attached as appendix, to give the overview of the water ingress behaviour during tunnelling. The statistics cover the tunnel from the portal to kilometre 16. From the Katse Dam impoundment experience, seismic activities have played a major role in environmental impacts and this has led to a whole village (Mapeleng Village) being relocated and to a reassessment of the influence of impoundment on the water regime.

There is also the very important issue of the geology of the dam site including the foundation for the dam itself and the sites for other structures such as the spillway, diversion tunnels and outlet works. The investigations are based on the following concerns:

- Whether the dam foundation has sufficient strength and durability to support the type of dam proposed
- Whether the foundation is watertight,
- And if not, how much grouting is required
- Whether the spillway needs concrete lining.

Another very important issue relates to the geology of the area and includes the following very important points:

- Whether the basin to be impounded is watertight or whether there are areas of cavernous limestone, palaeosols, contacts between flows, fissures and banding in lavas present, this may lead to the dam not retaining water.
- Whether the slopes of the basin to be impounded are stable and, whether there is a possibility of a landslide into the reservoir that may cause a wave of water to be pushed over the top of the dam, as it was the case in a dam in Italy.

Another important consideration is the exploration of the sources of construction materials necessary to build the dam. Extensive site investigations are being carried out to give answers to the above questions. No two dam sites are identical as far as geology is concerned, hence each new dam construction project must be investigated individually. Some dam sites may appear to be uniform in their geology but each has its own site-specific geology. It implies that although the geology may be very similar for all sites, each site is explored individually. To achieve these site-specific characteristics the following are employed:

- Geological mapping of the surface rock outcrops and bedrock

- Topography of the Project Area

- Excavation of test trenches, and trial pits

- Geophysical surveys, for example, seismic refraction, which was used in the case of the Mohale Dam

- Diamond core drilling and core logging

The site-specific geological investigations are carried out to make it possible for the engineering geologist to construct a geological model of the site. The dam designers use this model as a basis on which they can design a safe and economic dam structure appropriate to the geology of that particular site.

Experience in engineering geology has shown that it is very important to realise that even the most comprehensive site investigation programme cannot hope to reveal all the significant geological features of the site. It is therefore of critical importance that the actual geological conditions revealed during construction be on the continual bases compared with the geological model of the site derived from the investigations. Quite commonly unexpected geological conditions are revealed during construction, which results in amendments or changes, which need to be made to the original design. A record of the site geology "as found" during construction is also very important. The Mohale Dam is also passing through the phases of geological/geotechnical investigations, design, and construction. The next phase, which follows is operation and maintenance, in which the geological records are of cardinal importance in these phases.

The Mohale Dam is built a few metres downstream from the confluence of Senqunyane and Likalaneng Rivers. The two quarry sites are located within the dam site. The river is diverted through two tunnels so that work can take place on the foundation. The following activities take place on the foundation before commencement of dam construction:

- Stripping of overburden along the plinth line
- Preparation of the dam foundation
- Treatment of the two linear features on the left bank
- Grouting of the foundation

The construction material has then been transported to commence with the building of Mohale Dam. The hauling distance is significantly short ca. 2 km to and fro. Basalt rock is dumped and compacted. Compaction is carried out to ensure that the fill, as placed, displays maximum possible density.

The Mohale Dam, when completed, will supply Katse Dam with water to meet the export demand and it will store water whenever Katse Dam is full. In this case the water will flow in either direction.

Assessing and managing the social and environmental impacts of a project of the magnitude of the LHWP is of utmost importance in order to ensure that the project is beneficial to the nation as a whole as well as to neighbouring countries, such as the Republic of South Africa (RSA).

1.1 Aim of the Study

The study takes the Mohale Dam, including its ancillary elements, as a case study to evaluate its geological and geotechnical conditions and impacts on the environment. The study focuses primarily on:

- Site geology
- Geotechnical site investigations
- Quarrying and quarries
- Construction and construction methods
- Environmental impact and proposed mitigation measures

1.2 Objectives of the Study

The objective of the study is to highlight the significance of geology and geotechnical operations and how both dictate site selection and hence the environmental impact. The study started as part of the LHWP during the researcher's geological engagement in the LHWP (1992-2003). Special attention has been given to the rehabilitation works as well, as to how the geological conditions will impact on the environmental decision making prior to, during and after construction of the tunnels and dams of the LHWP. Some fundamental principles of geology and current geological engineering are outlined in this study, which proceeds to examine the interaction of geology with the environment.

It is a well-known fact that every activity of construction either above or underground involves a total environmental change. Anderson and Trigg, (1976), describe the total environment to include the ecology of the site, of which man himself is a part, and its visual appearance, but from the point of view of safety, and largely of cost, the geological factors in the environment are the most important site considerations in civil engineering. In their words, the most significant of these geological factors are those affecting stability within the planned structure. They proceed to say that the basic task of the geologist is to collect and evaluate the data relating to the stability of the geological environment, taking into account the fact that this environment is itself changed by construction activities and the operation of the structure thereafter.

The presence of man-made structures in the environment may vary from negligible to critical depending on the size of the structure that is being constructed. On the geological time-scale, every site is unstable, that is, it is liable to be affected by geological processes. On the historical time-scale, some sites are geologically stable, that is, short-term processes affect them.

Geologists are invariably involved in environmental impact investigation. This study thus focuses on the investigation and evaluation of the consequences to the environment of Mohale Dam and Tunnel engineering works. This involves a description of the LHWP and its impact on the environment, particularly noting any positive and adverse effects. The author considers the involvement of geology as constituted in the control or reduction of the effects of geological processes or hazards.

The aim of this study is to demonstrate the fundamental role of geology in identifying, understanding and solving multidisciplinary, long term, environmental challenges brought about by the construction of large dams and tunnels, with special reference to the Mohale Dam and Tunnel and specifically slope stability and hydrogeology. This study provides examples of how geological surveys based on environmental issues can form a basis for employing geological criteria in the future planning for the LHWP.

1.3 Previous Works

The Government of the Kingdom of Lesotho appointed Lahmeyer MacDonald Consortium while the Government of the Republic of South Africa appointed Olivier Shand Consortium jointly to undertake the feasibility study. The study took place from 1983 to 1986. The Lahmeyer MacDonald Consortium had the task of undertaking the study for the Katse Dam and Transfer tunnels whereas Olivier Shand was responsible for the Mohale Dam.

Generally, the geotechnical and construction studies focused on the description of the nature and the engineering properties of the basalt of the Lesotho formations and the identification of possible construction sites.

The geotechnical studies at Mohale Dam concentrated on the geological mapping of the dam site and adjacent potential rock quarry and soil-borrow areas. The geotechnical investigation, along the Mohale Tunnel route and at potential quarry areas, was carried out to provide geotechnical information for the design and the construction of the tunnel. The geotechnical investigation for the Mohale Dam and Tunnel were carried out in three stages these included a feasibility study, planning study and tender design. The feasibility and planning investigations were undertaken effectively, and the technical feasibility of the selected site for the tunnel and dam types confirmed, respectively. The mapping procedures were a combination of airphoto interpretation as well as detailed field mapping of rock exposures. The steep-sided valleys revealed good outcrops.

The subsurface investigations were carried out by means of nine boreholes drilled at the likely dam and spillway sites, three boreholes drilled in potential quarry sites and one hole drilled at the portal site for the intake to the Mohale-Katse tunnel.

Fourty-nine test pits were dug to supplement the geological mapping of soil-borrow areas.

Three main types of soil deposits were identified in the Mohale dam basin. They were:

i. Relatively small and discontinuous deposits close to river level, containing predominantly alluvium and colluvial slope wash materials.

ii. Isolated bodies of variable size, generally occurring above river level, containing predominantly residual material with subordinate alluvial and colluvial materials.

iii. Abandoned raised oxbows, again of variable size, containing residual, colluvial and alluvial materials.

In all cases, it was found that the weathered rock beneath the surface material represents a significant proportion of the deposit. The alluvial and colluvial material have often weathered since deposition making it difficult to determine their origin and to differentiate between them and the underlying weathered rocks. A fourth soil deposit type of minor extent occurs as bars in the present river channels. These deposits consist of mixed alluvial sands and gravel.

In conclusion, the necessary investigations were carried out to a satisfactory level and the environmental protection procedures were put in place. The researcher has observed the broad geology of the area and has mapped out the link between geology and the environment, basing his findings on the geological investigations that were undertaken by LHWP and his fieldwork on the project.

The Vaal Augmentation Planning Study was undertaken in 1995 with a view to finding the alternative to the LHWP. A number of alternative schemes were considered:

- The Caledon Cascades Scheme to redirect water at the Lesotho border into the Caledon River and through a series of 19 dams transfers this water to the Vaal River System.
- The Canal Scheme with three canal options (Aloedal, Goedemoed and Upper Caledon) to transfer water of the Orange River into the Vaal River System.
- Water transfer from the Zambezi River
- Water transfer from the Tugela River
- Sea water desalination
- Reducing the amount of water used for irrigation etc.

The LHWP remained the most viable of them all. Another scheme that looked viable but smaller was the Oxbow Scheme. The scheme was envisaged to consist of five reservoirs with approximately 102 km of tunnels to transfer water via a hydropower station on the Hololo River at Tlhaka to the Kroonspruit River in the RSA. The scheme was later abandoned due to the exponential growth of water demand in the 1980s in RSA.

A similar study was carried out by Olivier, (1976). Although, he touched on "Some Engineering Geological Aspects of Tunnel Construction in Karoo Strata with special Reference to the Orange-Fish Tunnel", his thesis focused more on the engineering side whereas this thesis on "The Geological Conditions and Environmental Impacts of the Mohale Dam" emphasises on geology and environmental awareness.

The report titled the Landslide Loss Reduction: a Guide for the Kingston Metropolitan Area: Jamaica that was prepared by Ahmad et al., (1999); discuss the landslides and environmental impacts in Jamaica. This work is intended to promote an understanding of the potentially dangerous consequences of development and urbanization in geologically sensitive areas. Another similar paper is one that was prepared by the American Geological Institute, (1999) under the auspices of the Environmental Geosciences Advisory Committee, titled "The Role of the Earth Sciences in the National Institute for Environment".

Workshop 1, which the researcher attended at the Lesotho Sun Cabanas, Maseru, on the 25 and 29 May 1998 focused on the topic: "Review of the Current State of Knowledge of the Seismo Tectonic Setting of Lesotho and it's Significance in Predicting Seismic Design Parameters for the Katse and Mohale Dams and Further Phases of the LHWP". Fieldtrip inspections en route to Katse Dam and Mapeleng Village on the 26 May 1998 and en route to Thaba-Tseka, Mashai, Marakabei and Likalaneng on the 27 May 1998 completed the workshop.

Field inspections of alleged volcanic eruption sites, Mohale Dam, and environs were undertaken on the 28 May 1998. On the 29 May 1998, workshop 2 was held at the same venue as before. The researcher attended both workshops and all field trip sessions. The following papers were presented:

- A Review of Critical Aspects of the Structural Geological Setting of the Kingdom of Lesotho, Prof. I. W. Hälbich
- From Karoo Dolerite to Drakensberg Basalts: Structure, Volcanics and Seismicity, Dr L. Chevallier of the Council for Geosciences, Bellville
- Of Diamonds, Dinosaurs and Diastrophism: 150 million Years of Landscape Evolution in Southern Africa, Prof. T. C. Partridge of WITS
- A Review of Earthquake History and Interpretation of Seismotectonic of the Kingdom of Lesotho, Prof. J.H. Hartnady of UCT

- The Lesotho Volcanic Event of February 1983, R.R. Maud of University of Natal, T.C. Partridge of WITS, J.N. Dunlevey of University of Durban West-Ville

- Seismic Parameters used in the Design of Large Dams, and the Role of Reservoir-Triggered Earthquakes, Dr Clarence Allen of the Seismological Laboratory, California Institute of Technology, Pasadena, California

- Review of the Seismic Hazard Analysis and Selection of Seismic Design Parameters for Mohale Dam, R.J. Anderson of Harza Engineering Company Chicago, Illinois, USA

- A Summary Review of the Reservoir Induced Seismicity at the Katse Dam, Kingdom of Lesotho, Dr G. Graham of the Seismology Section of the Council for Geosciences, Bellville

- Brief Report on the Volcanic Activity under the Step-up Transformer of the Thaba Tseka Line, near Pony Farm of Molimo Nthuse, Dr G. Prasad, NUL

- The Seismic Response of the Katse Dam and its Appurtenant Works, Mr. D. Develay, Chief Katse Dam Designer of LHC

All the above papers are specialized. The researcher finds them to be very interesting and they address some of the issues that this thesis focuses on. As a result, they form part of this work due to their invaluable information. The workshop has also been very interesting while listening to geologists, engineers and seismic experts exchanging views on very important issues that may affect our environment with probable devastating results. The stops during the field trips have been made at 24 locations in total, from the 26 to the 28 May 1998.

The environmental studies undertaken by various groups on the LHWP and other similar projects elsewhere have been also studied. The researcher likes specifically to take into account the Phase 1B Environmental Impact Assessment (EIA) of 1996. The study has been prepared during the planning period and it is based on a number of studies specific to Phase 1B. It takes into account lessons learned from the implementation experience of Phase 1A. It is concurrently being prepared with the early detailed design stages of the various engineering components.

1.4 Methodology

The researcher uses the geological/geotechnical information and data gathered from excavation of quarries, trenches, test pits, adits, dam foundations, piezometric measurements, seismic refraction surveys, geological mapping of the dam site and along the Mohale tunnel route, tunnels and laboratory investigations. The field geological mapping is carried out to define the geological conditions of the Mohale Catchment and Dam Site and to confirm some of the structures seen on the maps. A substantial amount of information is also being collected along the Katse shoreline villages concerning the changes in the water regime of the project area. The researcher has been involved in the construction of the Katse telemetric seismic stations and their monitoring. He has also attended relevant workshops and field excursions provided by the Lesotho Highlands Development Authority (LHDA) in conjunction with the consultants employed. He has also been involved in the geological mapping of the Katse Dam foundation and access galleries. He has been employed as quarry geologist and he has been LHDA's supervisor of the geotechnical investigations of the Mohale Dam.

The following data and information has been used to address the main objectives:

i. Monitoring of water springs along Mohale tunnel route to establish the impact of the tunnel excavation on the springs (carried out by the environmental officer)

ii. Face mapping in the Mohale tunnel and feature analysis, core logging and tunnel-spoil logging to establish the encountered geology of the tunnel for payment purposes (carried out by the researcher and the engineering geologist)

iii. Monitoring of the LHDA telemetric seismic stations and analyzing seismic events. (Carried out by the researcher and seismic technician)

iv. Mapping of linear features at Mapeleng village during relocation of the whole village following the collapse of poorly constructed houses in the wake of the first reservoir-induced seismic activities (carried out by the researcher)

v. Performed point-load tests as control measures on the remainders of the cores sent for testing at the Council of Science and Industrial Research (CSIR) and the University of the Witwatersrand (Wits), (carried out by the researcher)

vi. Field trips along the Katse Reservoir Shoreline and around the Mohale Reservoir Site in order to make field observations on slopes accompanied by interviewing villagers along the traverse about any change in their environment (carried out by the researcher)

vii. Compiled seismic inventory map (carried out by the researcher)

viii. Photographing areas in and along the valley of the Mohale Reservoir (carried out by different engineers personnel)

ix. Reading and extracting information from relevant text books of engineering geology

This information and data were combined with environmental and geological engineering aspects.

1.4.1 Scope

- General geology of the basalts
- Main characteristics of Lesotho Basalt formation
- Type of hydraulic structures built and being built in Lesotho
- Conductivity of Lesotho Basalts formations in general
- Rock mass properties of Lesotho Basalt in general

Dams with Reference to Mohale Dam

- The general location merits and disadvantages
- Factors that influenced the choice of the location design method of construction and efficiencies.
- Hydraulic factors studied
- Studied rock mechanics parameters.
- Influence of basalt conductivity and siltation effects on design and structural life expectancy.
- Design justification of the spillway and efficiency.
- Environmental Impact

Geology

The study shall detail the geological aspects of all sites as well as the engineering geology aspects. The geological study shall include details of the physical effects on the mineral and of the rock mass formations.

Hydrogeology and Hydrology

The study shall detail all hydraulic features of the Lesotho basalt and shall go into more detail for site elaborating on the Mohale site particularities. The conductivity of basalt shall be addressed in detail especially when evaluating Mohale dam sites.

Sediment movement shall be discussed showing silting period and measures taken in design for cleaning the reservoir base and the lower outlet structure. Such an approach can be adopted to evaluate sediment transportation for the tunnel inlets. The catchment's basin shall be investigated for silt collection and sedimentation.

Rock Properties

The study shall describe the general properties of Lesotho basalt. Emphasis shall be on rock mass mechanics properties and the method of evaluation of such properties showing its merits and the uncertainty of testing and evaluation methods. The study shall also focus on measures taken for treating the foundations and materials used in the construction to ensure the safety of the Mohale dam.

The study shall show the monitoring-scheme adopted for the Mohale dam structure in terms of instrumentation. A comparison between the investigation-stage findings and the findings during construction shall be tabulated and a recommendation for future investigation plans shall be devised and explained.

Environmental Management

The study shall look into the mitigation measures in place during construction and after. The study shall look into the post-construction rescue measures adopted in case of any disaster.

The Domain of Comparison and Evaluation shall include:

- Historical records
- Maps, aerial photographs and aerial surveying
- Seismic surveying
- Preliminary investigation schemes
- Detailed investigation scheme
- Quality control measures applied during construction and any particular investigations by either contractor or consultants and the reason for such investigations.
- Methods of boring and diameter of boreholes
- In-situ testing
- Laboratory testing
- Post-construction monitoring schemes

The achievement of the purpose of the research shall be judged according to the degree of accuracy with which the study managed to construct the general view of the Lesotho basalt formations properties, the details of the Mohale Dam particularities and the conclusions relating to the geological conditions on the environmental impact.

1.4.2 Data Source

The study focuses mainly on the Mohale Dam. The observation/data gained from the Mohale Tunnel and Katse Dam as well as the catchment areas are also used. In all these areas of interest, the Lesotho formation dictates the type of techniques to be employed in excavations. The structures of the formation can be easily mapped due to little or no soil cover in the project areas of interest. The researcher extracts some of his information/data from the following reports:

I. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 1. General Project Description and Survey Data. Rodio (RSA), March 1997.

II. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 2. Water acceptance Tests. Rodio (RSA), March 1997

III. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 3. Test Pit Logs. Rodio (RSA), March 1997

IV. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 4. Laboratory Tests. Rodio (RSA), March 1997

V. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 5. Borehole Logs, Part A and B. Rodio (RSA), March 1997

VI. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 6.Seismic Refraction Survey Report. Rodio (RSA), March 1997

VII. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 7. Borehole Core Photographs. Rodio (RSA), March 1997.

VIII. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Data File B7 and B9 Volume 1. Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

IX. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

X. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Annex B7. Geotechnical and Construction Materials Studies-Mohale Dam. LMC and OSC, April 1986.

XI. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Annex B11. Geotechnical and Construction Materials Studies-Petrology and Durability of Igneous Rocks. LMC and OSC, April 1986.

XII. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Annex B7, Drawings. Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

XIII. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Data File B7 and B9 Volume 1. Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

XIV. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume A Text. Geotechnical Investigation Report. LHDA, June 1995.

XV. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume B Appendices. Geotechnical Investigation Report. LHDA, June 1995.

XVI. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume 1. Dam Type Selection. Geotechnical Investigation Report. LHDA, June 1995.

XVII. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume 3. Drawings. Geotechnical Investigation Report. LHDA, June 1995.

XVIII. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume A Text. Geotechnical Investigation Report. LHDA, June 1995.

XIX. The Lesotho Highlands Water Project, Contract 123 and 124 Tender Documents: Data for Tenderers for Katse Dam Volume 5, Geotechnical Report, Data for Tenderers for Transfer Tunnel South and Associated Files.

1.4.3 Field Evidence

Du Toit's extensive writings, culminating in the third edition of The Geology of South Africa, which appeared six years after his death in 1954, make it clear that he regards good field observation as the most powerful tool of the geologist (Partridge, 1997). Partridge emphasizes the importance of field observation in his address to a workshop held at Maseru Cabanas in 1997. He states that geology should be based primarily on observable facts, and models – so much the present fad in almost every branch of science, which is, in his opinion, useful in geology only insofar as they can satisfactorily replicate or explain such observations, or predict from them.

Pre-field work is the first step towards successful site-specific geological investigations. This involves going out to relevant institutions to find out if there is any relevant existing information regarding the study. Geological maps, aerial photos, and the LHWP reports have been examined as tabulated in 1.4.1. Ahmad et al., (1999, state that other pre-field work involves the gathering of the proper equipment needed for the investigation and this may be as simple as getting a notebook and arranging for transportation or as complicated as getting seismic and survey instruments and arranging for several days in the field.

The researcher has undertaken field traversing to check and verify the presence of geological features seen on geological maps and lineaments observed from aerial photographs. The soil cover on the slopes springs and vegetation has also been checked as well as the river bed-floor for water tightness. Villages along the tunnel route and on the shoreline of the dams have been visited to create public awareness on the environmental impacts of some geological features. Reservoir construction presents a unique means for inducing landslides (Ahmad et al., 1999). In their analysis, it is stated that the filling of the reservoir increases the pore-water pressure in the material on the reservoir slopes and possibly causes slope failure. They claim further that rapid drawdown of the reservoir due to pumped storage use or dam failure can cause landslides at the reservoir's edge as lateral support is removed faster than pore-water pressures can decrease in the soil or rock. On that basis the Katse Dam, shoreline has been observed with the view to collect information that has been used as a model for the Mohale Dam. Material excavated from the Mohale Tunnel and Dam has been geologically logged. Logging of cores, mapping and face mapping have been continually undertaken as the dam and tunnel construction advanced. Environmental monitoring has been also in place in conjunction with the environmental monitors.

1.5 Laboratory Techniques

Laboratories on site have a capacity to carry out laboratory tests such as determining durability tests, the potential concrete aggregate test, water sample tests and petrographic analysis, while geochemical analysis and strength and deformation tests, are performed at the laboratories of Wits University and CSIR. The number of cores are normally being divided equally, for example if, five cores are sampled, two are sent to Wits and two to CSIR and one is kept for reference purposes. Core logging was performed on site to characterise the geology. Logging of tunnel spoil, with a 10 m spacing was carried out to confirm the geology.

2 REGIONAL GEOLOGY

The geology of Lesotho is given in the literature of the data sources above. In addition, a short account is presented in this thesis.

2.1 General

The Mohale dam site and basin is situated in an area underlain by basalt of the Lesotho Formation (Drakensberg Formation). The geology of the area is described in all the information sources of this thesis and in other LHWP documents.

2.2 Geological History and Stratigraphy

The project area is situated within the Great Karoo Basin, defined as a shallow depression in the earth's crust covering a large area in Southern Africa. This depression is said to have developed in the Paleozoic period. It was subsequently filled with a sequence of predominantly terrestrial and lacustrine sediments up to 7 km thick and capped by volcanic outpourings of basalts. It took more than 100 million years for the Karoo Sequence to be deposited, that is, between the Late Carboniferous Era about 290 million years ago (Dwyka Formation with tillites) and the Jurassic Era about 160 million years ago (basalt of the Lesotho/Drakensberg Formation).

It has been recognized that the depositional environment in the Karoo basin changed from polar in Dwyka times to arid at the time of deposition of the Clarens Formation and the Lesotho basalts. The sediments are mainly detrital, ranging from fine-grained argillaceous sedimentary rocks to sandstones and gritstones, and have been deposited in extensive alluvial flood plains, river channels and ephemeral lakes. The uppermost formations (Molteno, Elliot and Clarens) indicate increasing aridity of the depositional environment, and much of the Clarens sandstone is interpreted as a wind-blown sediment deposited in the form of sand dunes in a desert environment.

Different authors are of the opinion that the outpourings of the Karoo basalts appear to have started in Upper Triassic times and cut short the deposition of Clarens sandstone. The dolerite dykes are believed to be feeders of the basalt flows. The numerous dykes criss-cross the Karoo basin, cut through the sedimentary rocks and die out at various levels within the basalt flows. Basalt has been extruded flow upon flow to a thickness of at least 1 450 m, and originally covered a much larger area in the central part of the Karoo basin than today as observed by Dingle et al., 1983. They refer to the flood basalts as the Lesotho Formation instead of the Drakensberg Formation used by the South African Committee on Stratigraphy (S.A.C.S.), 1980. The huge aerial extent of individual flows shows that the flood basalts were highly mobile.

The basalt plateau of the Lesotho Highlands as well as the surrounding mesas of the Clarens, Elliot and Molteno Formations in the project area, are relics that have resulted from 150 to 160 million years of erosional process. The erosion of the pile has resulted in the formation of peneplain, dissected by a wandering steep-sided river valley.

- The South African Committee on Stratigraphy (1981) recommends the use of the term "Drakensberg Formation" for the same rocks that the Lesotho Department of Mining and Geology name "Lesotho Formation" on the 1981 geological maps of Lesotho. The two names are therefore used synonymously in this study.

2.3 Regional Geological Structures

The regional structure of northern Lesotho and surrounding areas is an elevated plateau of nearly horizontal lava flows and sedimentary strata, which undulates slightly. Locally, dips of up to 10° have been recorded. The base of the strata is thought to be broadly basin shaped in the central part of Lesotho (Dusar, 1980 and Burley et al., 1982 cited in the LHWP Reports).

Imposed upon this overall structure are sets of faults, shears and joints. These features are often selectively followed by dolerite dykes, which are contemporaneous with the lava flows. The preferred orientations for these features are east-southeast and northeast. There is also a set of joints with an approximate east-west orientation.

Other major structural features that occur in the region are diatremes, which are roughly circular feeder vents to the lavas, and diamond-bearing kimberlite pipes. The latter features are vertical pipes that postdate the lavas and thus transect all the strata.

2.3.1 Lithology

The overall lithostratigraphic column for the project area is summarized in Table 2.3-1. This table indicates that the sedimentary and igneous rocks of the Karoo sequence underlie the project area. This sequence of rocks was formed during the period from about 290 to 160 million years ago. The Karoo rocks were deposited in a basin, which is deepest in the eastern Cape Province. The project area is located towards the northern margin of the main Karoo basin, where formations are considerably thinner than in the middle.

Table 2.3-1: Summary Lithostratigraphy of the Project Area

Geological name	Lithology	Maximum thickness
Recent alluvium and coluvium	Gravel, sands and silts	N/A
Lesotho formation	Dolerite dykes, sills, plugs Basaltic lavas	N/A 1 600 m
Clarens Formation	Aeolian sandstone	165 m
Elliot Formation	Siltstones and mudstones with subsidiary interbedded sandstones	120 m
Molteno Formation	Sandstones with subsidiary mudstones and siltstones	50 m
Beaufort Group	Mudstones with sandstones	1 000 m

Karoo Sedimentary Rocks

The basal geological formation of the project area is the Beaufort Group, of which only the Delivery Tunnel traverses the uppermost strata. The Beaufort Group formed about 230 million years ago (Dingle et al., 1983 cited in LHWP reports). No outcrop of Beaufort Group was found in the area of the tunnel, but was penetrated to a maximum of 90 m in boreholes drilled during the feasibility study. Mudstones with subsidiary sandstone horizons were intersected in the boreholes.

Overlying the Beaufort Group on a disconformable erosion surface is the Molteno Formation. Sandstone is the dominant rock type in this formation, although subsidiary mudstone and siltstone beds are also encountered. The formation has a maximum thickness of 50 m in the project area and thins to less than 20 m in places,

The Elliot Formation lies conformably above the Molteno Formation and consists of siltstones and mudstones interbedded with subsidiary, though extensive, sandstone horizons. Individual beds range in thickness from less than 1m to more than 40m, and aggregate to about 110 m thicknesses. The finer grained rocks of this formation, and the mudstones of the Beaufort Group, exhibit a tendency to disintegrate rapidly when exposed to the atmosphere.

Conformably overlying the Elliot Formation are the mainly Aeolian i.e. windblown sandstones and siltstones of the Clarens Formation. The sands and silts were deposited in desert conditions as dunes with an undulating topography.

Karoo Basalt

The stratigraphic boundary between Clarens Formation sandstones and the basalt in the project area is sharp, generally flat lying and structurally continuous. Variations occur due to an undulating surface of the sands at the time of the first basalt outpourings, a post-depositional consolidation of the Lesotho Highlands area, and minor post-depositional faulting.

The basalts consist of a sequence of horizontally bedded flows, which have an accumulated thickness of about 1 500 m (range is 1 400 m to 1 600 m). Volcanic clastics and tuffs with volcanic breccias occur quite often within the lowermost basalt flows. Another frequent phenomenon in the lowermost parts of the basalts is intercalation of lenticular sandstones and siltstones, which represent the final phases of deposition of the Clarens sandstone facies. Intercalated sedimentary horizons have not been observed in the basalts higher than some 200 m above the general boundary of the Clarens and Lesotho Formations, and are not considered likely to occur at the higher elevations.

The thickness of basalt flows normally varies between 0.5 m and 20 m with an average of about 5 m. Exceptional flows up to 100 m thick occur, especially in the higher part of the whole sequence. Flow boundaries do not represent major discontinuities and are generally tight. Palaeosols have also been observed between flows high above full supply level of the reservoir as discontinuous layers, which form lenses in places.

Five types of basalt have been distinguished in the field during geotechnical investigations. The field descriptions that follow have been amplified from petrographic studies:

(i) Dense, compact microcrystalline basalt, which have a doleritic character. These basalts only occur in the centre of some flows that are usually greater than 5 m thick. They contain very minor or no amygdales. This type contains rare zeolites and minor quantities of expandable clays derived from deuterically-altered glass. Such basalts usually have a high strength

(ii) Dense basalt with disseminated soft mineral spots. This type usually occurs in the centre of flows and contains a varying but considerable amount of expandable clay minerals derived from deuterically-altered glass that is concentrated in spots. These basalts are not durable in ethylene glycol. Amygdales and zeolites may occur in minor quantities and the latter sometimes cause breaking of the rock material

(iii) Slightly amygdaloidal basalt (less than 1% of cut surface area).

(iv) Moderately amygdaloidal basalt (1 – 10% of cut surface area).

(v) Highly amygdaloidal basalt (more than 10% of cut surface).

The latter three types occur either at the base and top of flows, or within thin flows. These three types contain significant amounts of expandable clay minerals derived from deuterically altered volcanic glass and thus tend to react with ethylene glycol. The durability of all the basalts depends on the proportion and accessibility of free water to the altered glass. Zeolites are always in variable amounts, and cause some of these basalts to be crazed.

The five types of basalt occur in the following general proportions:

-	Dense and compact doleritic basalt	6%
-	Dense basalt with disseminated soft mineral spots	20%
-	Slightly amygdaloidal basalt	38%
-	Moderately amygdaloidal basalt	24%
-	Highly amygdaloidal basalt	12%

A full analysis and description of the petrography and durability of the basalts is given in most of the LHWP geotechnical documents.

Intrusions

Subsequent to the deposition of the sedimentary rocks and contemporaneous with the outpouring of the lavas, the strata were injected with feeder channels of lava which have solidified to form dolerite plugs, dykes, sills and inclined bodies of various sizes and shapes.

The dolerites are generally fine or medium grained, dense, very strong rocks of great durability. Occasionally the dykes coincide with faults (based on reported observations by several authors, who state that faulting is only present in the lower flows of the basalts and underlying formations). The dolerite rock material in the faults is sheared and slickensided. Details of dolerite petrology and durability are given in Feasibility Study Supporting Report B. The project area has also been intruded since Karoo times by diamond-bearing kimberlite pipes. Weathering is usually noticeable along contacts and where it penetrates deeper into the basalts.

2.4 Post-Karoo Deposits

The project area has undergone several phases of erosion since the outpouring of the lavas about 160 million years ago and the consequent tilting of the strata. The eroded material has largely been transported away from the project area by rivers, while colluvial soils have developed on the hill slopes and terrace surfaces. Localized alluvial deposits occur in the major rivers and in some minor stream valleys.

Seismotectonic and Seismicity

i.) Seismotectonics

The seismotectonics model for the area is based on limited source mechanism studies. It is believed that predominantly east-west compressive stresses in the basement rocks below the Karoo sediments cause deeper earthquakes. These can cause strike-slip movement along Precambrian 'megashear' zones in the basement rocks. In view of the tectonic stability of the area, the compressive stress field is considered to be relatively low. There are only a few major fault systems evident in Lesotho.

The ones that do exist are of the same age or older than the volcanic strata and are believed to be inactive. There is no known volcanic activity in the area and the last known volcanic events were the emplacement of kimberlite pipes and dykes during the Cretaceous Period. In general, earthquake activity cannot be related to surface geology.

Most earthquakes are believed to occur below the Karoo sediments. This believe is attributed to the possibility that these earthquakes occur due to stress relief associated with isostatic rebound following the erosion of superficial formations. Hence, for these earthquakes, differential vertical movements may be the primary source mechanism.

ii.) Seismicity

Lesotho and the surrounding areas are said to have experienced a relatively low level of seismicity. While the distribution of earthquakes in general appears random, some patterns are evident. There is no direct relationship with known tectonic features, which are exposed at the surface, has yet been determined for any recorded earthquakes. A total of about 100-recorded earthquakes have occurred within a 150 km radius of the Mohale dam site. One of these was a Magnitude (M) 5.5 event; five events were M 4.5 to M 5.0 and the remainder ranged from M 2.0 to M 4.5. The largest earthquake recorded within 150 km of the dam site was the M 5.5 Zastron earthquake, which occurred about 143 km south-west of the project area in 1957. Large events of

M 6.0 and M 5.9 occurred further west at Koffiefontein in 1912 and 1976 respectively.

The mining activities are thought to have induced the events at Koffiefontein in 1912 and 1976. Nevertheless, those events are indicative of crustal stresses and earthquake capability in the region. Three recorded earthquake epicentres occurred within a 30 km radius of the Mohale dam site. These events occurred at the following times and sites:

- M 3.8 on February 10, 1958; 23 km NE of the site
- M 4.2 on February 5, 1971; 13 km S of the site
- M 3.6 on May 3, 1976; 24 km from the site

Earthquake focal depths vary throughout the region from 0 km to more than 60 km. The majority of focal depths are, however, less than 33 km in depth. These are termed shallow earthquakes whilst focal depths greater than 33 km are referred to as deep earthquakes. The distribution of historical earthquake epicentres in Lesotho indicates that the majority of earthquakes occur in the southwest half of the country, including the Mohale site area. Ongoing studies will attempt to delineate precise seismic zoning. To date, however, no clear pattern of earthquakes has been proven in the Mohale area.

2.5 Physiography and Morphology

Climatic records in or near the Mohale area are limited to rainfall records at Marakabei and St. John the Baptist Mission, both situated south of the area, and a two-year data record from Blue Mountain Pass which lies west of the area. With a relative close correlation existing between temperature and altitude in the highlands, extrapolation from temperature data at Thaba Tseka (2160 m.a.s.l) and Butha Buthe (1680 m.a.s.l) forms a basis for a reasonable estimate of mean monthly temperatures in the study area.

The climate of the Mohale area is generally sub-humid, with about 85% of the precipitation occurs in the summer season from October to March when 10 to 12 rain days per month may be projected. The winters are normally dry. Hail occurs frequently and snow is common and may fall in any month of the year. Sudden weather changes are common, with temperatures falling rapidly within a few hours.

Summers are warm and winters are cold, with wide diurnal variations (Lahmeyer, MacDonald Consortium, 1986).

2.6 Hydrology and Hydrogeology of Lesotho

Lesotho Formation (Lower-Middle Jurassic) consists of massive amygdaloidal tholeiitic basalt with maximum thickness of 1600 m in the eastern part of the country with moderate to low groundwater potential. Well productivity and spring discharge from 0.1 l/s to 3.0 l/s. Perched aquifers are localized both in the uppermost and weathered zones of the formation. Indicative maximum depth of groundwater occurrence is 100 m. Intrusive rock represented by dolerite dykes and sills (lower Jurassic), including kimberlite (Cretaceous), with moderate ground water potential. Well productivity and spring discharge ranges from 0.3 l/s to 4 l/s. Aquifers are localised within the intrusion and often along the contact zones with the surrounding country rock. This is observable along the mountain slopes especially after rain.

Water is the most valuable natural resource of Lesotho. The headwaters of the major drainage system in Southern Africa, namely the Senqu/Orange River system, are located in Lesotho (see Map of Southern Africa). The Senqu River provides about half of the total flow of the Orange River. The Senqu River has a mean annual discharge of approximately 107 m^3 /s at Seaka Bridge. The country (30 648 km^2) comprises two major morphological units, referred to as the Lowlands and the Highlands which have altitudes between 1 400 to 1 750 m and 1 750 to 3 500 m respectively. The Highlands cover more than 75% of the country and include the Senqu River and its main tributaries, mainly the Malibamats'o, Senqunyane and Makhaleng Rivers, flowing mainly on the basaltic rocks of the Lesotho Formation. The Mohokare River (Caledon River) and its principal tributaries, the Hololo, Hlotse, North and South Phuthiatsana Rivers, flowing mainly on the sedimentary rocks of the Karoo Super Group, incise the Lowlands.

Base flow, the groundwater contribution during dry seasons, also differs between the Highlands and the Lowlands. In 1985 (May-October), one of the driest years on record for the past 20 years, most of the Rivers and Streams in the Lowlands were dry, while those in the Highlands were still flowing, albeit at a very low discharge rate. Various factors explain this persistence of Highland's base flow:

- The runoff/rainfall coefficient varies essentially with rainfall and slope, with higher values in the highlands than in the Lowlands. It has been estimated that in the mountain catchment areas, runoff is approximately 22% of rainfall, whilst in the Mohokare catchment is 12%;

- The basins in the Highlands have a larger catchment area than those in the Lowlands.
- Occurrence of snow in the Highlands, particularly between the months of June and September, encourages groundwater infiltration;
- Basaltic rocks generally weathered to a depth of a few metres and covered by the bush vegetation overlie the Highlands, providing a temporary storage for groundwater. The sedimentary rocks of the Lowlands are less permeable than the volcanic rocks, encouraging proportionally more evapotranspiration and surface runoff at the expense of infiltration.

Table 2.6-1: Estimates for Lesotho Water Resources are as follows:

(Source: Dept. of Groundwater Studies)

Catchment	Area Km^2	Precipitation P (mm/y)	Total Runoff (mm/y)	%R	Flow (cub.mm/y)	Deficit (mm/y)	%P
Lesotho	30,648	775	158	20	$4.73*10^9$	621	80
Senqu	20,485	749	167	22	$3.42*10^9$	582	78
Makhaleng	2,911	844	199	24	$0.58*10^9$	645	76
Maphutseng	363	737	166	23	$0.06*10^9$	567	77
Mohokare	6,890	824	97	11	$0.67*10^9$	727	82

3 GEOLOGY OF BASALT

The information below is extracted from the works of Veevers et al. (Southern Africa: Karoo Basin and Cape Fold Belt, 1994) and other authors.

3.1 Lesotho Formation

This formation is synonymous with the Drakensberg Formation as referred to by (Dingle et al., (1983), Eales et al., (1984) and Marsh et al., (1997)) cited in several LHWP Reports, Visser (1984), Olivier (1976), Chevallier et al., (1997), and Duncan, (1997).

The tholeiitic basalts cover 80 % of Lesotho and represent the remnants of a large lava flood, the paleo-extent of which is unknown. It is speculated that the basalts covered an area similar to the one occupied by the dolerite dyke and sill outcrops of the Main Karoo Basin. On the other hand, according to Chevallier, (1997), geochemical (mantle plume), structural (the proximity of an inferred triple junction), stratigraphic (thicker sills beneath Lesotho) would argue towards a restricted extension of the basalt surface and eruptivity.

Veevers et al., (1994) and Stockley, (1946), describe the Lesotho Formation as a monotonous succession of amygdaloidal basalt lava flows. The thickness of individual flows differs slightly in the opinion of different authors. The range is 0.5 m to 70 m and the aggregate thickness to the exposed top is 1 400 m (range is 1 400 m to 1 600 m).

The basalt is tholeiitic. Rare silicic lavas in the andesite-dacite range are encountered in the lower part of the Drakensberg Group. Eales et al., (1984) cited in LHWP, write that in the northeast Cape therefore a feature of the overall development of the Karoo igneous events is that the earliest eruptions were characterized by diversity in style and composition of the erupted products. This later evolved into widespread and regular effusion of compositionally monotonous lavas, which built the bulk of volcanic pile.

The subvolcanic complex of dolerite dykes, sills, so called "bell jar intrusions" and irregular bodies that intruded the Karoo sediments represents the feeder channels for the overlying lavas. Within the limits of the dolerite line that skirts basement and the southern edge of the Karoo Basin, the greatest concentration of the dolerite (dolerite: sediment ratio of >0.5) lies in an elliptical zone that trends across northern Lesotho; the dolerite sheets, according to Winter and Venter, (1970) cited in LHWP, seem to terminate at a critical distance of a few thousand feet below the basalts. Dingle et al. (1983) cited in LHWP explain this observation and the stratigraphic distribution of the sills in the Beaufort and Molteno, dolerite plugs in the Molteno, tuff in the Elliot, agglomerate and lava in the Clarens, and pyroclastics and lava at the base of the Drakensberg Group and lava above, by the following sequence of events:

- Earliest upward movement of magma in the late Triassic to produce explosive activity
- During late Triassic-Early Jurassic Elliot and Clarens deposition, an increased scale of upward movement of the magma in sills 1 to 2 km beneath the surface, degassing of the magma, and contact with ground water produced explosive action and created diatremes
- Main eruption flood basalt by dyke injection in the Early Jurassic (193 ± 5 Ma), followed by a second major injection in the Middle Jurassic (178 ± 5 Ma), as dated by Fitch and Miller, 1984. Recent dating (Chevallier, 1997 citing Duncan, 1997) shows that the Drakensberg Lavas were extruded in a very short period of time 180 Ma.

The effect of zeolite mineral on the durability of Lesotho basalts:

Basalts intersected by tunnels of the LHWP have shown variable response with respect to durability and this has led to the decision to provide the full tunnel length with concrete lining. Rapid deterioration of the tunnel profile during construction could be ascribed to either overstressing or chemical deterioration, when in many cases a combination of these factors could be identified. Future construction works in Lesotho will be similarly affected and a better understanding of the deterioration may result in cost saving by selecting alternative alignments/sites and construction methods. It has been suggested that the most vulnerable mineral is the zeolite laumontite, and that the mineral is concentrated along specific horizons within the basalt succession.

3.2 Basalt Classification

Bieniawski, (1974), emphasises the classification of rock masses to be of cardinal importance in terms of the design of tunnel routes and cross-sections of a tunnel, estimation of construction duration and in terms of selecting the appropriate tunneling method. This opinion stands in the context of dam construction. Geological classification is an important data source and definition process since it is the fundamental means of understanding more or less all other sources (Drs. Sharp and Stacey special advisers to Lesotho Highlands Consultants (LHC, 1992)). They emphasise that it is also important to recognize that the extent and detail to which geological classification can be taken may be greater than what is observable in general during routine mapping.

The main characteristic of a lava flow is the presence of pipe amygdales at the base, non-amygdaloidal basalt in the central part, and amygdaloidal basalt towards the top of the flow. The amygdales are usually filled with secondary minerals (zeolites, calcite, clay minerals, rarely chlorite, quartz). Thinner flows are commonly amygdaloidal, while the basalt of thicker flows sometimes is doleritic at the centre with characteristic medium-crystalline texture and random joint orientation.

Table 3.2-1: Field classification of basalt

Description	Amygdales (% by area)	Clay (% by area)	Comment
Doleritic basalt	0	<2	Some clay may be visible in this rock type
Non amygdaloidal basalt with dark soft spots (DSS)	0-1	>2	
Moderately amygdaloidal basalt	1-10	Variable	
Highly amygdaloidal basalt	>10	Variable	

Doleritic Basalt (DB)

Doleritic basalt is similar to medium-crystalline doleritic intrusions and is generally only found in thicker flows. It contains very small amounts of smectite clays.

Non-Amygdaloidal Basalt (NAB)

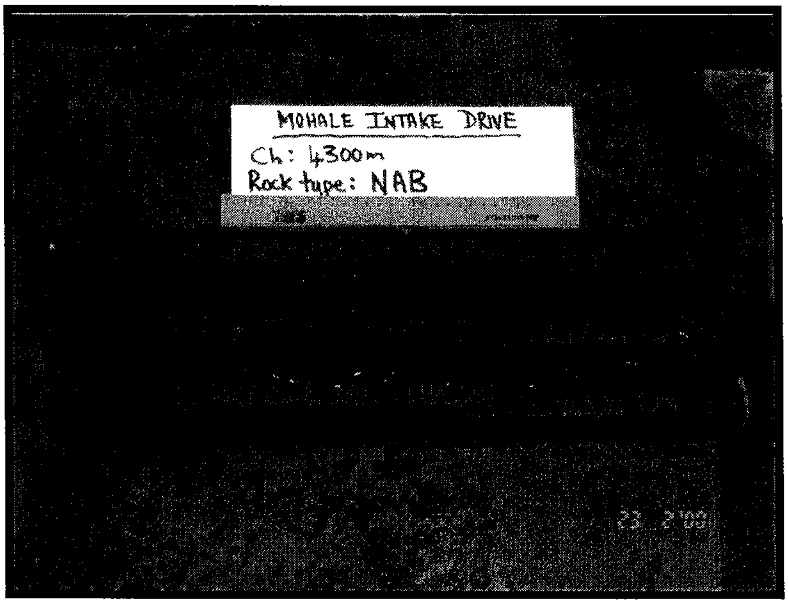


Figure 3.2-1: Cores of NAB. They are selected for WITS and CSIR for Laboratory. Tests (UCS and TS)

NAB is gray to dark gray; (fig. 3.2-1), often-speckled red or green, medium crystalline and usually occurs in the centre of the flows, which are more than two metres thick. It contains smectite clays (DSS), which occur in varying concentrations, (generally 2% to 25%) and range in size from 1 mm to 7 mm in diameter. Isolated amygdales do occur and occupy about 1% of the total cross section. The olivine rich basalt is a slightly coarser crystalline NAB and is found in thicker persistent flows.

Moderately Amygdaloidal Basalt (MAB)

MAB is usually grey to dark grey, fine to medium crystalline, and contains between 1 and 10% amygdales, (fig. 3.2-2). The upper contact, with the highly amygdaloidal basalt, is gradational. It contains less smectite clays than the NAB. MAB is commonly about 2 m thick.

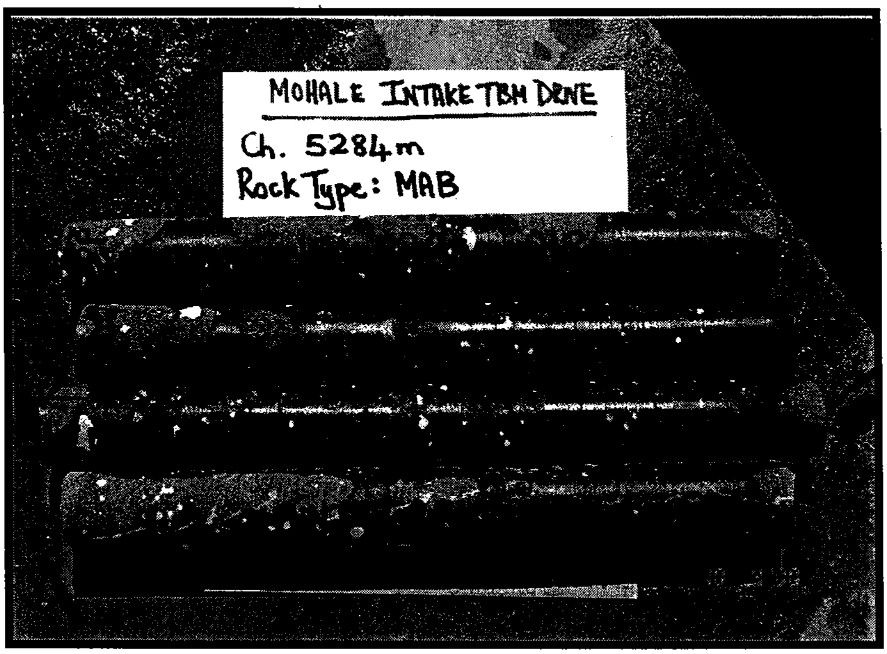


Figure 3.2-2: Cores of MAB. They are selected for WITS and CSIR for Laboratory Tests (UCS and TS)

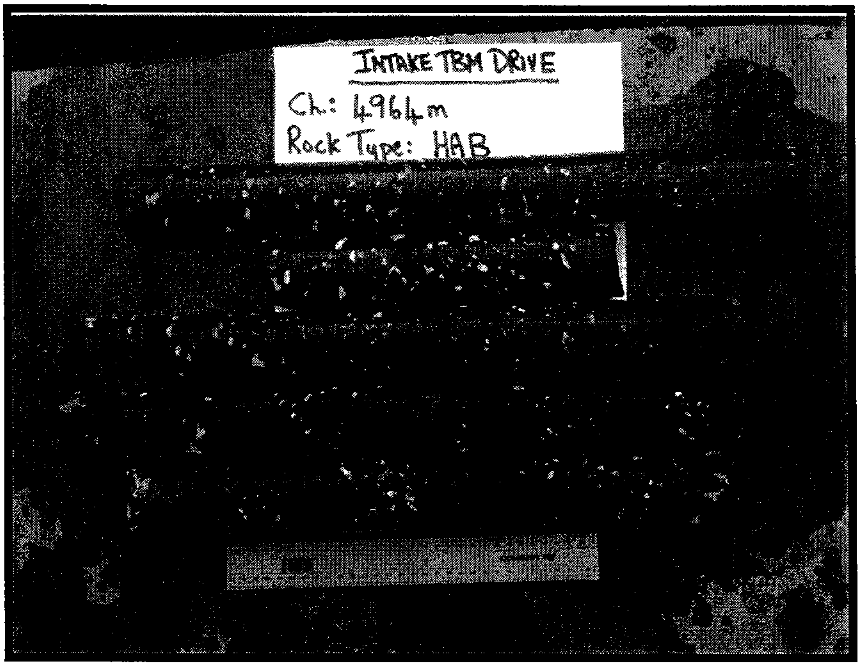


Figure 3.2-3: Cores of HAB. They are selected for WITS and CSIR for Laboratory Tests (UCS and TS)

Highly Amygdaloidal Basalt (HAB)

HAB varies in colour from red, reddish grey to greenish grey, is fine to very fine crystalline, figure 3.2-3. It occupies the upper zones of flows or areas near the base of flows in the pipe amygdale zone. HAB contains more than 10% (50% in the uppermost part of the zones just below the flow contact) amygdales.

3.3 Intrusive Rocks

Dolerite dykes as described in several LHWP-documents usually form intrusive rocks, which generally occur as near-vertical structures bisecting pre-existing strata and in most cases can be followed for several kilometres in plan. Several authors take dykes to be feeder channels for the basalt outpourings and consequently extend upwards through successive lava flows as deposition occurred. However, dykes die out at various levels of the lava pile. This implication is that a greater number of dykes will be encountered in the lower levels of the lava pile without outcropping. This has been seen in studies elsewhere which indicate a greater frequency of dykes outcropping at lower elevations. This kind of behaviour was often experienced during driving of the LHWP tunnels. This phenomenon was termed unforeseen ground conditions in tunnels. Dolerite dykes tend to be following the predominant lineament trends, which strike west northwest to east-southeast or northeast to southwest. Few dolerite dykes have been identified along the Mohale tunnel route. Seven dykes that cross the tunnel axis have been identified in the field, two of which are associated with joint zones. These dykes range in width from 1,0 m to 12,5 m. Dolerite sills are infrequently observed in outcrops in the Lesotho basalts. These linear features have also been identified across the Mohale Reservoir area (Fig. 5.4-1).

Regional maps and visual observations indicate a relatively high density of dykes to the east of the Malibamats'o River. This may be indicative of the presence of an impermeable barrier at depth (such as a sill) near the Mohale Tunnel, which has prevented dykes penetrating up into the overlying sequence (LHWP). It is thus expected that the tunnel may possibly intersect one or more dykes or sills. It should be noted that the drilling investigation programmed for the Mohale Tunnel has not been extensive enough to predict the presence of dolerite sills at tunnel elevation. Boreholes have been spaced at intervals of between approximately 2 km and 22,5 km. Rock cover above the tunnel ranges between approximately 400 m and 700 m over a substantial length of the Mohale Tunnel. Therefore drilling closely spaced boreholes to determine the presence of sills would have been impractical. The excavation of the Mohale tunnel however has produced valuable information relating to the above discussion.

Dolerite is very strong, ranges from fine to medium crystalline and is substantially harder than the basalt host rock (fig. 3.3-1). Dyke contacts are generally very fine crystalline and field observations have indicated a range of conditions varying from tight and welded to open with weathered rock. The encountered dykes in the Transfer Tunnel (deliver water from Katse to Muela) indicate a range of joint spacing varying from very close to medium spaced joints to those dykes with medium to very wide joint spacing. There are usually two jointing sets either observed, of a random attitude or comprising three sub-perpendicular joint sets. Instability may result where a regular joint system implies blocky conditions. These blocky conditions were one of the time delays courses during the excavation of the Mohale tunnel (Appendix B). Joints range from tight to those containing infill, which is common in dykes and may be emplaced as joint infill and/or as anastomosing veins. Dykes are often associated with faulting and slickensiding can be observed on some dykes, as it was the case with the dolerite dykes intersected by the Mohale and Transfer Tunnels. Figures 3.3-1, 3.3-2 and 3.3-3 below, show this particular type of dyke, which has been examined.

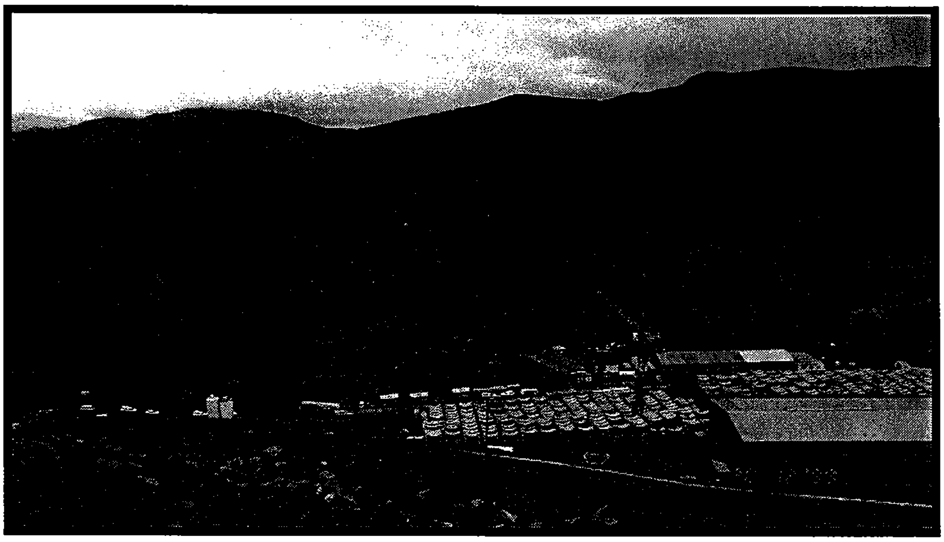


Figure 3.3-1: Dolerite Dyke. It is about 100 m from the Mohale Intake Structures. This Photograph was taken looking to the north over the Senqunyane River. In the Background on east Bank is the Precast Yard.

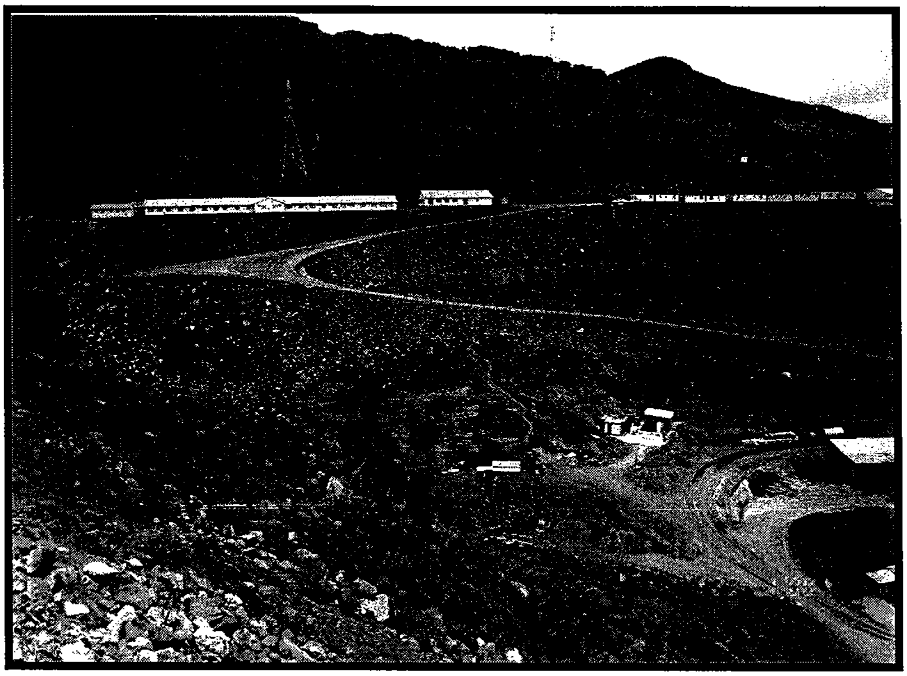


Figure 3.3-2: Dolerite Dyke. It is on the eastern Side of the River. Photograph was taken looking southeast from the Intake Structure. Engineers and contractors offices and access roads can be seen.

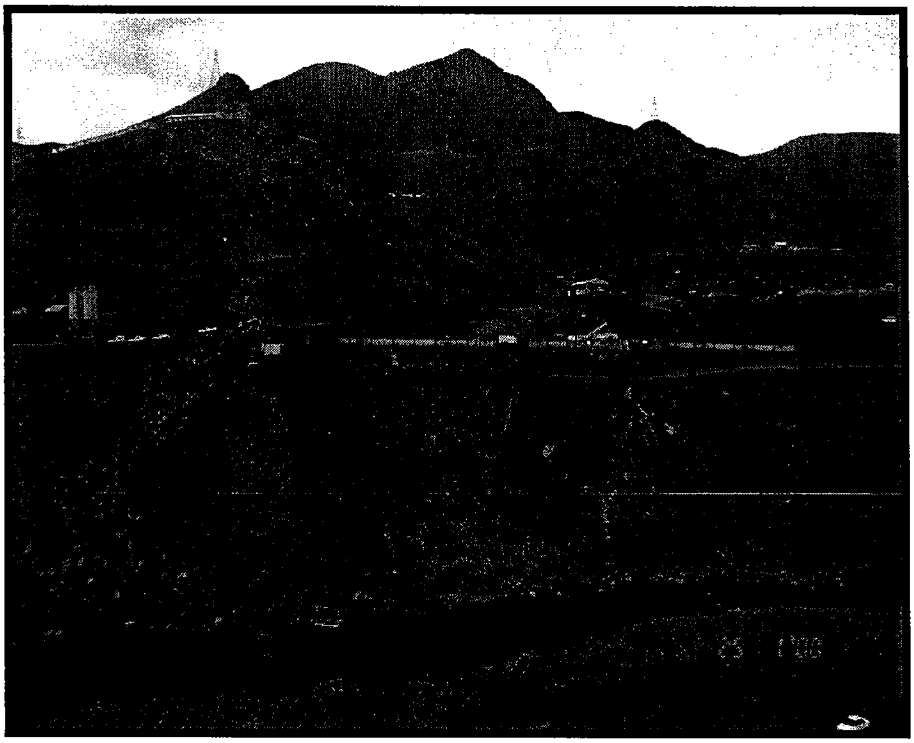


Figure 3.3-3: Dolerite Dyke. It is on the eastern Side on the River Bank. Photograph was taken looking east from the west bank.

4 SITE ASSESSMENTS AND SELECTION OF TYPE OF DAM

4.1 General Site Appraisal

The following points are considered very important for a general site appraisal

- A satisfactory site for a reservoir must fulfill certain functions and technical requirements.
- Functional suitability of a site governed by the balance between its natural physical characteristics and the purpose of the reservoir.
- Catchment hydrology, available head and strong volume etc. must be matched to operational parameters set by the nature and scale of the project served.
- The presence of a site (or sites) for a dam, the availability of materials suitable for dam construction and the integrity of the reservoir basin with respect to leakage (all these sum up to what is termed technical suitability dictation).
- The geological and hydrogeological or geotechnical characteristics of the catchment area and site are the primary determinants establishing the technical suitability of a reservoir site. To these an assessment of the anticipated environmental impacts of construction and operation of the dam or tunnel is added.

Investigation of the reservoir shoreline is conducted as required to confirm the stability of the potentially vulnerable areas in the vicinity of the intended dam. The construction material availability, especially suitable fills and aggregate sources are investigated and evaluated thoroughly. The hydrological studies are continual to confirm the initial investigation extent and results.

4.2 Type of Dams

Types of dams are summarized as follows:

- Rockfill Dam: rock foundation preferable; can accept variable quality and limited weathering. Cut-off to sound horizons is required. It is suited for all weather placing. Requires material for core, filters etc.

- Gravity Dam: suited to wide valleys, if excavation is less than ca. 5 m. check discontinuities (jointing/pattern of joints) in rock mass with regard to sliding. Moderate contact stress. Requires imported cement.
- Buttress Dam: similar to gravity dam, but higher contact stresses require sound rock. Concrete saving relative to gravity dam is from 30 – 60%.
- Arch and Cupola Dams: suited to narrow gorges, subject to uniform sound rock of high strength and limited deformability in foundation and most particularly in abutment loading. Concrete saving relative to gravity dam is 50 – 80%.

4.3 Design Features of Dams

Below follows a list of the design features of dams, which can have major implications with regard to programming and costs as well as to the environmental impact:

- Cut-off
- Spillway systems, including channels and stilling basins
- Internal drainage systems
- Internal culverts, galleries
- Foundation preparation, including excavation and grouting
- Construction details, e.g. transitions or filters in embankments or construction joints details in concrete dams.

4.4 Selection of Type of Dam

The optimum type of dam for a specific site is determined by estimates of cost and construction programmes for all design solutions, which are technically valid. Where site conditions are such that viable alternatives exist it is desirable that options are kept open, until a preferred solution is apparent. It may also be necessary to take into account the less tangible sociopolitical and environmental considerations in the determining of that solution (site selection).

Four important considerations are listed below:

- Hydraulic gradient: the nominal value of hydraulic gradient ? for seepage under, around or through a dam varies by at least one order of magnitude according to dam type.
- Foundation stress: nominal stresses transmitted to the foundation vary greatly with the dam type.
- Foundation deformation: certain types of dams are better able to accommodate significant foundation deformation and/or settlement with serious damage.
- Foundation excavation: economic considerations dictate that the excavation volume and foundation preparation should be minimized.

In general, foundation investigations are about foundation competence of the dam that is assessed in terms of stability, loading capacity, compressibility (soils) or deformability (rocks), and effective mass permeability. This means that the foundation should be competent enough to support the weight of the dam for safety, thereby reducing any chances of dam failure. The investigative techniques to be adopted depend upon the geomorphology and geology of the specific site. Nowadays, environmental issues play a pivotal role in the decision making process. There should be a full comprehensive study before the approval of any kind. The study encompasses the physical and social environment.

4.5 Dam Site and Geology

More than any other form of civil engineering, the construction, and adequate geological knowledge of dam sites conditions is an essential consideration (Duncan, 1969). Bell, (1980) and Wahlstrom, (1974) shares this point of view. They are of the opinion that if at some future date there is any failure of the dam, its foundations or its reservoir banks, the forces, which will be unleashed, are enormous and that geology is the first area to be investigated. Wahlstrom, (1974): "No matter how much preliminary investigations may have been addressed to the problem, it is never certain what geological features will be discovered when a dam site is excavated and even years later, unforeseen and unpredictable weaknesses may appear. Most uncertainties have been related to the geology of the site rather than to engineering design and workmanship". For instance, Wahlstrom cites Gruner, (1962) as listing the causes of dam failures as follows:

• Foundation failure	40%
• Inadequate spillway	23%
• Poor construction	12%
• Uneven settlement	10%
• High pore pressure	5%
• Acts of war	3%
• Embankment slip	2%
• Defective materials	2%
• Incorrect operation	2%
• Earthquakes	1%

The vast sums of money spent upon construction of dams make it necessary to ensure that they will function efficiently for long periods. Comprehensive studies are necessary, therefore, and these fall into three groups (Duncan, 1969):

- Studies at the dam site
- Studies of the reservoir area
- Studies of the catchment area.

The possible environmental impact can be elucidated timeously by impact studies. In this case, the environmental impact on the dam site and the catchment area create the opportunity for timeously remedial procedures.

4.6 Site selection

One of the major tasks of geological engineering is site investigation (Duncan, (1969); Thomas, (1976); Wahlstrom, (1974); Bell, (1980)). It is applied to gain information about the subsurface by employing both traditional methods such as classical geological mapping and drilling, and geophysical surveying methods. Other tasks, which must be undertaken, are the assessment of the stability of foundations, dams, slopes, underground structures (e.g. tunnel construction), the prediction of natural hazards (e.g. seismic activities), geohydrological analysis (e.g. watertightness of a reservoir), and environmental impacts.

Every reservoir has unique geological characteristics. Gaining a thorough understanding of these characteristics is very expensive and time-consuming according to (McCully, (1996); Duncan, (1969); Bell, (1980); Thomas, (1976)). They claim that millions of Rands may have to be spent on a geological survey before it determines whether a site is suitable for the construction of a dam. Hence, they agree that it is normal for the dams to be designed with a partial knowledge of local conditions – the constructor just has to hope that they are not going to meet any unstable formations which will fail to support their foundation or that the roof of their tunnels does not come crashing down. Some sites may be relatively uniform in their geology (one lithology with simple structure and a regular pattern of surface weathering). McCully, (1996), mention in their particular study, that in three quarters of 49 projects assessed by the World Bank study of hydropower in 1990, construction costs have been found to have experienced unexpected geological conditions. The study has concluded that for hydrodams " the absence of geological problems should be treated as the exception rather than the norm."

The selection of dam sites is based primarily on the topography, which has been determined by geology (fig. 4.6-1 and 4.6-2). Ideally, one seeks a narrow gorge, hoping for minimum quantities in the dam, and a valley opening out upstream from the dam to provide the required storage (Thomas, 1976). He states that the topographical and geological regional maps are the first data providers, while satellite imagery adds another dimension to photographic/visual interpretation, especially when the photographs are printed with colour differentiation (Thomas, 1976). The range of the geological features revealed in the vast coverage of these photographs can often improve on interpretations made from the examination of geological exposures or from normal aerial photography. Thomas, (1976), states that the assistance that this new facility can provide in identifying major fault systems and landslides on a regional scale is of special significance.

The river basin is divided into regions for development. There may be an alternative site along a length of the river within a region. According to Thomas, (1976), since going further downstream implies a greater catchment area and hence a greater flow. The value of this additional water and the greater head for the same full supply level must be balanced against the increased cost for a higher dam as well as a larger environmental impact area (Thomas, 1976).

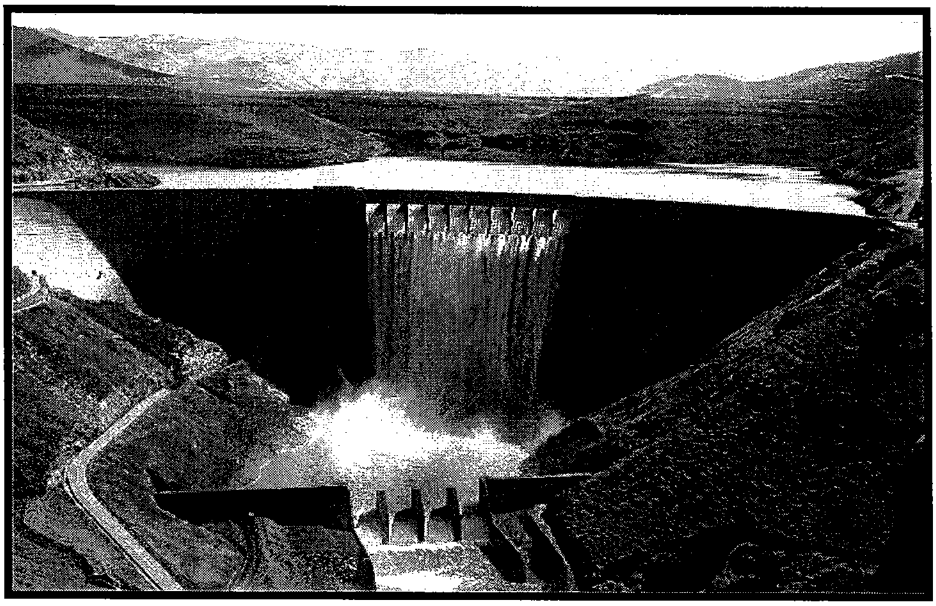


Figure 4.6-1: Katse Dam. The Photograph was taken facing North from the Operation Centre.

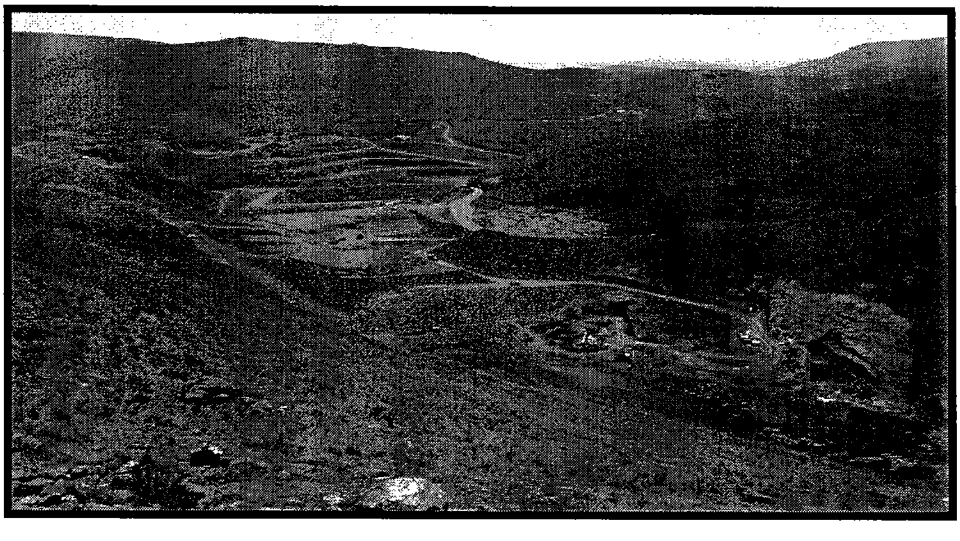


Figure 4.6-2: Mohale Gorge: Construction of the Dam at its early Stage. Diversion Tunnels. Dam Embankment. Further Upstream is Quarry for Rockfill Material. Photograph was taken looking Upstream from Engineers Office.

4.7 Site Investigation

Bell, (1980), Thomas, (1976) and Wahlstrom, (1974), state that site investigation involves exploration of the ground conditions on and below the surface. They take this as the prerequisite for the successful and economical design of engineering structures and earthworks. Insufficient or inadequate information with respect to the character of the ground can lead to the production of an unsatisfactory design, which may subsequently result in serious damage or failure of the structure concerned (Bell, 1980). He is of the opinion that there should be no attempt made on saving funds at this stage because this might lead to additional cost at later stages if unfavourable conditions, previously undiscovered, are found during the construction stage. Focus should also be, at this stage, on the environmental impact studies, no pushing aside of the study for later stage because the investigation is coupled with the environmental impact. It is thus clear that every little step of the project has a bearing on the environment. Economic considerations should be regarded as a secondary matter as far as safety is concerned (Bell, 1980). Safety should always be on top of the investigations agenda at all times.

Bell, (1980), considers the general objectives of site investigations as follows:

- Suitability assessment of a site for the proposed structure.
- Attempt to foresee and provide against difficulties that may arise during construction due to ground and/or other local conditions.
- Investigations should continue throughout the construction stage as well as environmental monitoring.
- It is essential that the prediction of ground conditions which constitute the basic design assumption, are checked as construction proceeds and designs should be modified accordingly if conditions are revealed which differ from those predicted.

Features that should be sought include old and potential landslides, geological faults and major joints striking parallel with the valley; they are considered the result of stress relief during erosion of the valley. The resultant joints may be open or infilled with the products of weathering and in the case of Mohale Dam zeolites, palaeosols and clay. The products of weathering easily absorb water and form a weak link between the rock mass. In the transfer tunnels, LHWP Tunnels, a phenomenon termed crazing was observed and it was linked to basalt exposure to moist air. Consequently, they can either be a lubricant or washed away resulting in mass movement. They may present construction hazards or if intercepted by other joint systems, may provide leakage paths around the dam (Thomas, 1976). Thomas, (1976) and Wahlstrom, (1974) agrees that the examination made along the beds of rivers and tributary streams will indicate strikes and dip of rock formations; these are features particularly relevant to the stability of both the dam and abutments. They emphasizes, that, any spring or underground water should receive close attention in case they provide paths for leakage from the reservoir. For this reason grouting was performed for the foundation of the Mohale Dam.

In summary, site investigations require careful planning and a considerable investment of time and resources. In situ and field test-techniques is performed to supplement laboratory testing and to make a comparison so that a comprehensive conclusion can be drawn (fig. 3.2-1, 3.2-2 and 3.2-3). The interpretation of geological and geotechnical data needs the closest cooperation between the engineering geologist, the geotechnical specialist and the dam engineer as well as the environmental specialist. The team members must stay always in technical touch with each other.

4.7.1 Reservoir Sites

Bell, (1980), considers the following aspects listed below as very important, when investigating a potential reservoir site:

- The climate (The amount of rainfall, run-off and infiltration and evaporation and transpiration)
- Topographical conditions
- Geological conditions of the area (e.g. Geohydrology, seismicity)
- Environmental conditions (e.g. Vegetation, Settlements)

4.7.2 Leakage from Reservoirs

When the reservoir is leaking, the following are the indicators:

- The sudden increases in stream flow downstream of the dam site with boils in the river,
- The appearance of springs on the valley sides.

Investigations will quickly focus on the geology of the site, i.e. rock formation, structural elements and construction materials (fig. 5.6-1, 5.6-2, 5.6-3 and 5.10-1). In the case of Mohale Dam, two quarries were identified for the production of aggregate and rockfill material. It may be attributed to major defects in the geological structure such as solution channels, fault zones or buried channels, for instance, a buried palaeo channel was discovered during site investigations of Mohale Dam, through which large and essentially localized flows take place. Leakage is considered a key parameter that relates to the overall performance of the CFRD and it has to be monitored on the daily basis. Large leakage rates are usually considered as an indication that damage has occurred to the perimeter joint and/or that the concrete face has cracked to a considerable degree. Seepage through the foundation may also be a contributing factor to large leakage rates. The fundamental design concept of the CFRD is that the several embankment zones of the dam including the face support material, filters, transitions, underdrainage and the body of the dam must remain stable even if extremely large leakage rates were to occur. The ability of rockfill to accept and pass large flows is well known in the literature. Thus, if the embankment zones and the foundation treatment have been designed and constructed appropriately, the large leakage rates are not an indication that safety is a problem, but rather that remedial treatment may be required to reduce the leakage, (Bell, 1980).

4.7.3 Time and Money Availability

The amount of money and time required to investigate a dam site depends primarily on the site and the type of dam. The CFRD was chosen for Phase 1B amongst other due to low construction price and short time required to complete. The Mohale Dam Contractors promised to complete construction ahead of schedule and at a discount. Indeed this was accomplished.

According to Thomas, (1976), investigations will normally encompass the following considerations:

- National or international policies; even the type of dam may be influenced by such policies rather than by minimum cost

- The purpose of the dam and how it fits into the existence or future plans for water conservation and utilization in the region
- Ultimate safety of the present or future inhabitants of the valley
- Finance (its availability and constraints)
- Environmental impacts (lacustrine, riverine, estuary, socio-economic)
- Quality of water, chemical and biological
- Physical factors such as hydrology, geology, topography
- Availability of resources, both materials and skills

4.7.4 Basic Concepts

Adequate information to be provided to the team (section 4.7) so that a dam that is stable against overturning and sliding, both on or within the foundations; the rock must be competent to withstand the superimposed loads without crushing or undue yielding and the reservoir basin must be watertight (Duncan, 1969; Thomas, 1976).

4.7.5 Basic Data

World Class team of expert has been engaged in all stages of investigations of the LHWP. The LHDA, the engineer and the contractor have their own panels of experts in search to deliver a world-class product, which is the completion of Phase 1. Experts should be employed to handle the following subjects according to Thomas, (1976):

I. Topography: It is the prime element used for planning and designing infrastructures. The determination of; the excavation volumes to be carried out at various levels; the existence of any low saddles around the perimeter; the quantities of material to be excavated in the dam, for the layout of access roads, and for the setting out of the dam; and onto the completed dam to verify that it behaves in accordance with design.

II. Meteorology and hydrology: A characteristic of a river that involves the average quantity of water available; the minimum flow, both as the absolute minimum and the minimum average over a period of a month or a year; the maximum flow that has been recorded and estimates of what might occur in the future. Variation in flow determines the storage necessary and the height of the dam to full supply level. Flood flows determine the spillway arrangement and freeboard required for flood routing through the reservoir; and wind velocity determines the behaviour of the reservoir body and the additional freeboard required to prevent overtopping of the dam by setup, seiche or wave effects. Each of the hydrological factors has some influence on the height of the dam and hence the cost of the project. It is therefore most desirable that meteorological and hydrological stations be installed near the site at the earliest possible date.

III. Geology and Seismicity: The geologist familiarizes himself with the regional geology and seismicity of the site. Any sign of landslide (ancient or recent) is very important for him to note. It is important for him and the team that geological investigations continue, not only through the design phase of the project, but also into the construction phase. Thomas, (1976) agrees to this opinion. Careful logging of all exposures is considered essential. Road cutting, trenches, shafts, adits or drill holes to observe and record the exposed rocks are always used whenever available. Colour photographs of cores are recommended for permanent record. This should also be employed to record all excavated surfaces that will later be covered by the dam. Thomas, (1976), emphasise the need for seismic investigations even though the region may be regarded as free from earthquakes

Since modern theories are actually relating microseismicity to possible seismic activity, it is therefore imperative to install seismographs near a proposed reservoir site some years prior to construction of the dam. Arrangements must be made for reporting all man-made events for instance blasting of any kind within 100 km; once correlated, such events are easily recognized on the charts (Thomas, 1976). According to Thomas, (1976), it is very important to record and report any seismic activity, a recurrence of which might necessitate modification of the dam, or might induce landslides following the filling of the reservoir. The reason for such records is to form a database of seismic events for the site, which can be used in future to analyse events. This database is aimed to provide valuable background information should seismicity occur after the filling of the reservoir.

Finance availability: The availability plays a crucial role in any project's formulation and ultimate take off. The method of financing a project and the money available may influence the selection of the type of dam. This in many instances brings about budget cuts, which may sometimes be detrimental to the project implementation and subsequently leads to incomplete studies.

Environmental Implications: Techniques are developed and economic evaluation is of dominant public concern. Intangibles, whether positive or negative are omitted from the studies since no one is prepared to put a monetary value on them. Around 1960, a realization dawned that there is actually more to life than monetary benefits to cost ratios (Thomas, 1976). During this period, the effects of pollution and environmental degradation are realised. In fact, the 1970s are remembered in history as the decade when man rebelled against pollution and desecration of the environment (Thomas, 1976).

4.7.6 Geological Classification of Rocks

In order to draw a three-dimensional picture of the geological subsurface, it is necessary to know the sequence in which the rocks were originally deposited (Mandel and Shiftan, 1981). They argue further that the geological age has a rock unit and time interval representing not only intrinsic scientific interest but also great practical importance. From a groundwater point of view, Mandel and Shiftan, (1981), regard the pore space in rocks as the most important property of rocks. The pore space may be contemporaneous with the rock (primary porosity), or it may be due to subsequent processes, such as fracturing, solution, and weathering (secondary porosity). On the other hand, primary porosity may be fully or partly obliterated by subsequent processes such as mechanical compaction, secondary deposition of minerals, and filling with clay particles. Geologically, there is no clear-cut correlation between the mode of rock generation, the subsequent geological history of the strata and the pore space, which the formation contains. Another descriptive classification distinguishes between granular porosity characterized by evenly distributed voids, irregular porosity characterized by large irregular distributed voids, and fissured porosity, where more or less continuous voids are aligned in a certain direction.

Joints are some of the most common rock structures. They differ from faults because their magnitude of displacement is very small. Duncan, (1969), defines joints as fractures along which no appreciable displacement has occurred. Although some joints have a random orientation, most occur in sets. He states that many rocks are broken by two or even three sets of intersecting joints that slice the rock into numerous regularly shaped blocks. These joint sets often exert a strong influence on other geological processes. For instance, chemical weathering tends to be concentrated along joints, and in many areas, groundwater movement and the resulting solution can influence the direction in which the stream courses flow. "Geological classification is an important data source and definition process since it is the fundamental means of understanding all other data sources." Drs. Sharp and Stacey cited in LHDA (1991), special advisers to LHC. They emphasise that it is also important to recognize that the extent and detail of geological classification may be greater than that which is observable in general during routine mapping.

5 PROJECT PROFILE

About 80% of the area is situated in the Maloti Mountains of Lesotho, 15% in the lowlands and the rest in the Republic of South Africa. The area is easily accessible by road. All Phase 1 areas can be reached using the excellent mountain roads from Maseru and Leribe. Katse Dam has a big enough airstrip that can easily accommodate medium-sized aeroplanes. The area has hospitals, primary schools, and commercial centers and lodges constructed under the LHWP infrastructure programme.

The area comprises incised mountainous topography with steep slopes, deep elongated valleys and generally shallow soils. The narrow, v-shaped valleys tend to widen and even out slightly in the southern section of the project area. The Senqu River dominates together with its tributaries; the Malibamats'o and Senqunyane Rivers provide the drainage of the area.

The mean annual precipitation is 1200 mm in the high altitude areas in the northwest and 600 mm in the southeast. Most of the rainfall occurs in the summer months as thunderstorms. Snow falls in the winter months and winters are cold and dry. There are three vegetation zones, which dominate the vegetation cover. The zones are mainly differentiated by altitude. Water quality is high. The upper Senqunyane River provides an important habitat for the maloti minnow.

The total population of the Mohale catchment is estimated at 7435 people. The communities earn their living through repatriation of money from urban Lesotho and as migrant labourers in the mines and other industries in the Republic of South Africa. Agriculture is practised in the valley bottoms and on ridges where the soil is somehow suitable. They also keep livestock, which they sometimes sell to gain some sort of income.

The majority of data and information of chapter 5 were extracted from the Final Report of MCG on the Geotechnical Investigations for Mohale Dam.

5.1 The Lesotho Highlands Water Project (LHWP)

Nthako et al., (1997), and Bell (1997) have discussed the project. The Lesotho Highlands Water Project (LHWP) has developed from what has been initially named the Oxbow Scheme. The purpose has been to transfer water from the upper tributaries of the Orange River in Lesotho to the tributaries of the Vaal River in the Republic of South Africa (RSA). The demand of water has grown exponentially from 1974 in the Vaal Triangle to such a degree that the Oxbow Scheme had to be abandoned and considerations for a much bigger scheme had to be made.

The Lesotho Highlands Water Project (LHWP) is a multi-purpose project designed to progressively divert the water from the Senqu River System to the Vaal River System north of Lesotho by a series of dams, tunnels, pumping stations and hydroelectric works. The development phases are Phase 1 (1A and 1B), Phase II, Phase III and Phase IV. These Phases comprise 5 major dams, 200 km of tunnels and a 72 MW hydroelectricity station, which provides Lesotho with sufficient electricity.

Table 5.1-1: Phases of the Lesotho Highlands Water Project

Phase	Reservoir System	Completion Date
1A	Katse Reservoir	1996
1B	Matsoku Weir	2001
1B	Mohale Reservoir	2003
2	Mashai Reservoir	2007
3	Tsoelike Reservoir	2017
4	Ntoahae Reservoir	2020

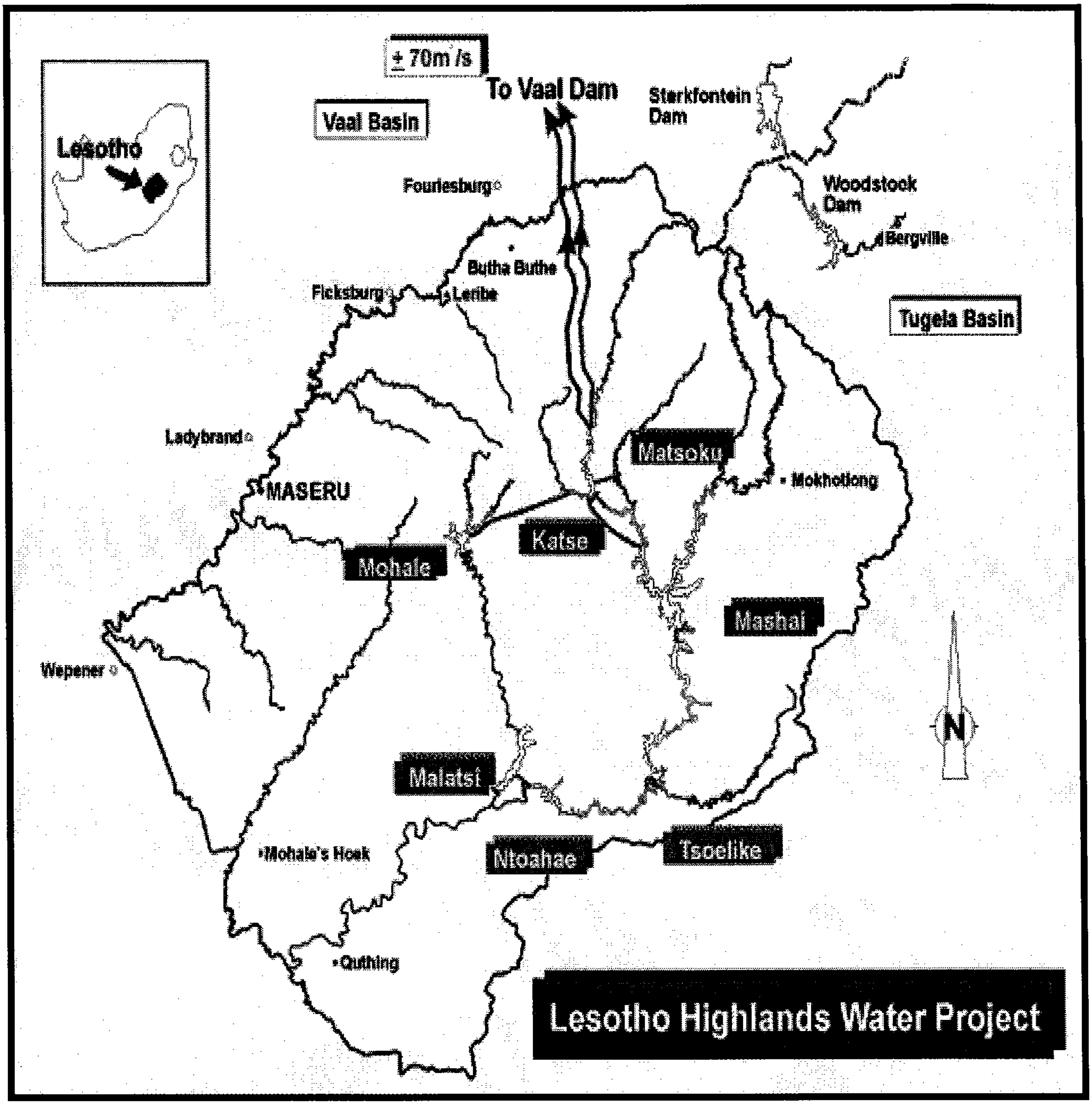


Figure 5.1-1: Map of the Lesotho Highlands Water Project (with no Scale)

The figure 5.1-1 is the map of the LHWP as is envisaged in the Treaty between the Kingdom of Lesotho and the Republic of South Africa.

The primary Objectives of the Project are:

- To change the direction of some of the southwesterly flowing waters of Lesotho to take a northerly direction to Gauteng in the Republic of South Africa.
- To generate hydroelectric power in Lesotho, in conjunction with water transfer.

- To provide a water supply, irrigation and regional development in Lesotho (to promote the general development of the remote and underdeveloped mountain regions of Lesotho and to ensure minimum environmental impact as well as to avoid negative socioeconomic impacts in the region).

Construction of Phase1 has commenced in 1990 and has been successfully completed in 2002. This phase consists of the construction of the following main structures:

- Access roads and bridges
- Power lines, telecommunications, housing offices and other commercial structures
- Katse Dam (a 185 m high double-curvature dam)
- The first transfer tunnel (a 45 km transfer tunnel from Katse Reservoir to Muela underground power station)
- Muela tailpond (a 55 m high concrete curved gravity tailpond)
- The first delivery tunnel (a 35 Km delivery tunnel from Muela Reservoir to the Ash River Valley in RSA)
- The terminal structures for transfer tunnel of Phases 1B and 2
- Mohale Dam (a 145 m high concrete faced rock-fill dam)
- Mohale Tunnel (a 32 Km long tunnel connecting Mohale and Katse Dams)
- Matsoku Weir and tunnel diversion (diverting Matsoku River water to Katse Reservoir)

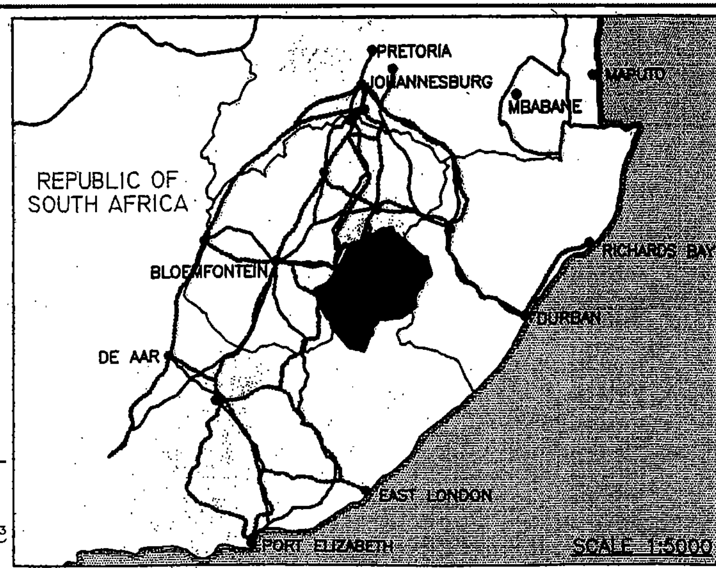
Phase 2 and 3 are still under discussion between the Government of the Kingdom of Lesotho and the Government of the Republic of South Africa (RSA). The project has been envisaged for completion in 2020. The planned water transfer amounts to 35 m^3/s in Phase 1. This has been expected to double at the completion of Phase 3. If the second and third Phases are completed as initially envisaged in 2019 or shortly after, the transfer requirement will increase to 70 m^3/s.

The total availability of water in the Senqu River was estimated to be approximately 150 m^3/s, and the conclusion is that up to 70 m^3/s can be abstracted without any resulting shortfalls in the water downstream of the Lesotho border, as far as the

Atlantic Ocean. The abstraction is therefore categorised as surplus water from the catchment of the Senqu River in Lesotho by the LHWP Environmental Action Plan.

The entire scheme has been budgeted at a cost of $US8 billion at the time of the signing of the treaty. Most of the money has been to be raised within South Africa with a large part emanating from a levy on existing water users. The rest has been raised from the international development bank and the World Bank acting as the main broker and monitoring the environmental and social impacts.

The immediate impact of the Project has been the need to relocate villages and to provide alternative food sources for a significant number of households. The villagers are being relocated to different parts of Lesotho in accordance with their choices. The project has naturally created both the negative and positive impacts in terms of environment, socio-economics and health problems. Some of these impacts have been experienced during the construction phase while others shall be encountered at a later stage of operation.



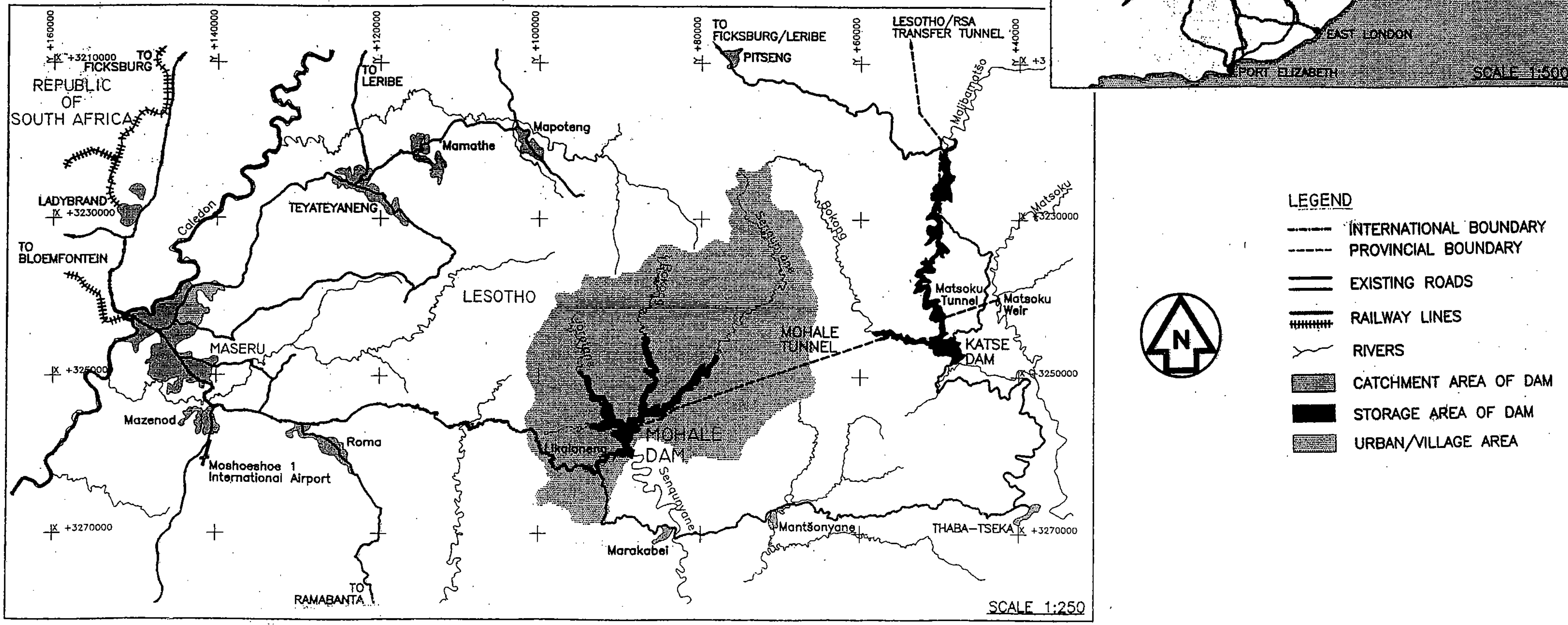


Figure 5.1-2: Phase 1B Catchment Area

5.1.1 Description of Phase 1A of LHWP

Phase IA, includes the construction of the Katse Dam and appurtenant structures, a water conveying system consisting of a 45 km long transfer tunnel and a 37 km long delivery tunnel and the Muela Hydroelectric development scheme, as well as a network of access roads and other infrastructure servicing the various construction sites. This phase is expected to satisfy the water delivery requirements to the Republic of South Africa (RSA) until the beginning of 2001; at which time the initial components of Phase IB (Matsoku Tunnel and Weir) should be operational. Different authors especially Bell and Haskins, (1997), have described this phase.

5.1.2 Description of Phase 1B of LHWP

This phase involves the construction of the Mohale Dam on the Senqunyane River and the Mohale Tunnel (to deliver 300 million m^3 of water into the Katse Reservoir) as well as the Matsoku Weir and Tunnel (to deliver 60 million m^3 of water to Katse). Phase 1B is the second and final stage in the construction of Phase 1 of the LHWP. Phase 1B, unlike Phase 1A, is situated in the Maloti Mountain Region. The basaltic lavas are layered to a near horizontal position and they form part of the Lesotho Formation.

The topography of the area has been described as an elevated and incised plateau (generally above 2 000 m.a.s.l) with high relief and steep-sided valleys and narrow rivers (fig. 5.2-1). The soils of the area are derived from the basalt, are generally thin at high elevations and on steep slopes and are deeper in the valley bottoms. The catchment of the Senqunyane River (fig.5.1-2) has been estimated at 929.9 square km. Jorotane, Bokong and Likalaneng are the main tributaries (fig. 5.3-2).

The Maloti Mountain Region of Lesotho is a place of high rainfall, temperate summers and long cold winters.

5.1.3 Mohale Dam and Infrastructure

The Mohale Dam Site has been identified in 1985-1986 during the feasibility study, and the type of dam has been identified in the planning study of 1995.

The Mohale Dam is a concrete-faced rockfill dam. The dam wall is located immediately on the confluence of the Senqunyane and Likalaneng Rivers. The embankment is 145 m high and contains 7.8 million cubic metres of rock material and concrete lined ungated spillway on the left abutment and an outlet facility. The construction of the dam involved a 27 m high Upstream Cofferdam and two diversion tunnels, tunnel 1 and tunnel 2, and a 7 m high downstream cofferdam. One of the tunnels is 6.7 m diameter and is 674 long, and is not lined. The other tunnel is 5.0 m in diameter, concrete lined, and 591 m long. The Cofferdam and the tunnels have been constructed to divert the Senqunyane River on the left bank, and comprise a stockpile area for tunnel spoil, two quarries (one to supply rockfill material and the other to supply material for concrete works), borrow areas, temporary access roads and bridges. The Mohale reservoir shall cover a surface area of 22 square kilometers and shall be able to transfer 857 million cubic metres out of the total storage of 946 cubic metres.

5.1.4 Mohale Tunnel and Infrastructure

The construction of the 32 km long and 4 m diameter Mohale Tunnel, involved the following:

- An area for tunnel spoil stockpile,
- Access roads and bridges,
- Borrow pits,
- Construction of a lower and upper intakes, construction of a gate shaft,
- Construction of an underground tippler on the outlet side, construction of tippler on the intake side,
- Precast factory for segments manufacturing,
- Construction of Workshops

The tunnel shall gravitate water into the Katse Reservoir on the Bokong River or into the Mohale when the Katse is full. Gates in the intake and outlet shafts will control the flow. At normal discharge, the velocity of water in the tunnel will be 0.7 m/s.

The area has a hospital, a primary school, and a commercial center and a lodge as well as good roads constructed under the LHWP infrastructure programme.

5.1.5 Matsoku Weir and Tunnel

The Matsoku Weir is a 10 m high and 180 m long concrete mass gravity weir constructed across the Matsoku River immediately upstream of the Tlopa Stream. It is connected to the Katse Reservoir by a 4 m diameter and 6 km long drill and blast tunnel. The tunnel is lined with concrete of approximately 400 mm. There is no storage capacity for the weir since it operates as a run off river diversion. Roads have been constructed to the outlet and to the intake.

5.1.6 Roads

Work on the first part of Phase1B involved the supply of access roads from Maseru to Mohale Dam and Tunnel Adit. It involved the construction of 59 km Mountain Road from St. Michael to Patiseng (reconstruction and rehabilitation of the Mountain Road between St. Michael and Patiseng Village). There is a newly constructed road, which leaves the Mountain Road at Patiseng, supplies access to the Mohale Dam via a 2 km spur road, and provides access to the Mohale Tunnel Adit at an approximate distance of 19 km.

5.2 Lesotho Highlands Geological Setting

The regional geology of the Lesotho Highlands as described in other geological literature and LHWP document is hereby given as follows: The regional geology of the Lesotho Highlands comprises horizontally layered tholeiitic basaltic lavas of the Lesotho Formation (Drakensberg Basalts). The tholeiitic basalt covers 80% of Lesotho and represents the remnants of a larger lava flood, the paleo-extent of which is unknown. This formation is the uppermost stratigraphic unit in the Karoo Sequence and has a total preserved thickness of approximately 1 450 m. The basalts were deposited in horizontal sequences of flow. These flow successions are typically 0.5 to 10m thick with occasional thicker flows of up to 40m.

The basalts are predominantly of olivine-poor tholeiitic composition. They are dark gray, reddish gray and occasionally maroon in colour and are fine to medium grained. Typically, each basalt flow is amygdaloidal toward the top, grades to non-amygdaloidal at the centre and displays characteristic tubular amygdales near the base. Amygdales are gas bubbles in the original lava flows, which were subsequently infilled by minerals such as clay, zeolite, quartz or calcite. At the centre of thicker flows, the basalt may grade to a rock with a doleritic appearance. Occasionally thin tuff interlayer occurs within the basalt flow sequence and the frequency of such interlayer is believed to increase with depth in the basalt succession. Narrow, intrusive dolerite dykes form prominent lineaments and there is evidence of dolerite sills either in outcrops or from fracture patterns in the basalts. The dolerite dykes are believed to be formed by the magmatic infilling of the original fractures which served to feed the surface basalt lava flows.

5.2.1 Mohale and Katse Catchment Area

The area is characterised by a dendritic drainage pattern focusing on the Senqunyane River. The area is generally located between 2500 and 3000 m.a.s.l and with a steep terrain. The geology of the area is similar to the geology given elsewhere in this work. The mountainous nature of the terrain determines that large areas are steep and rocky with no real agricultural potential except for some grazing. The parent material is so homogeneous that the soil properties differ by degree (i.e. texture, depth and drainage) rather than type. Soils are generally deep in the Senqunyane valley and their depth decreases with altitude and distance away from the valley bottoms, as it is the case in the Malibamatso valley. Winters are cold and dry with intermittent snowfalls. Maximum rainfall is in the summer months (December, January and February). Three dominant vegetation zones cover are also observed as described in the beginning of this chapter, Chapter 5. Water quality is high and the upper Senqunyane River provides an important habitat for the Maloti minnow.

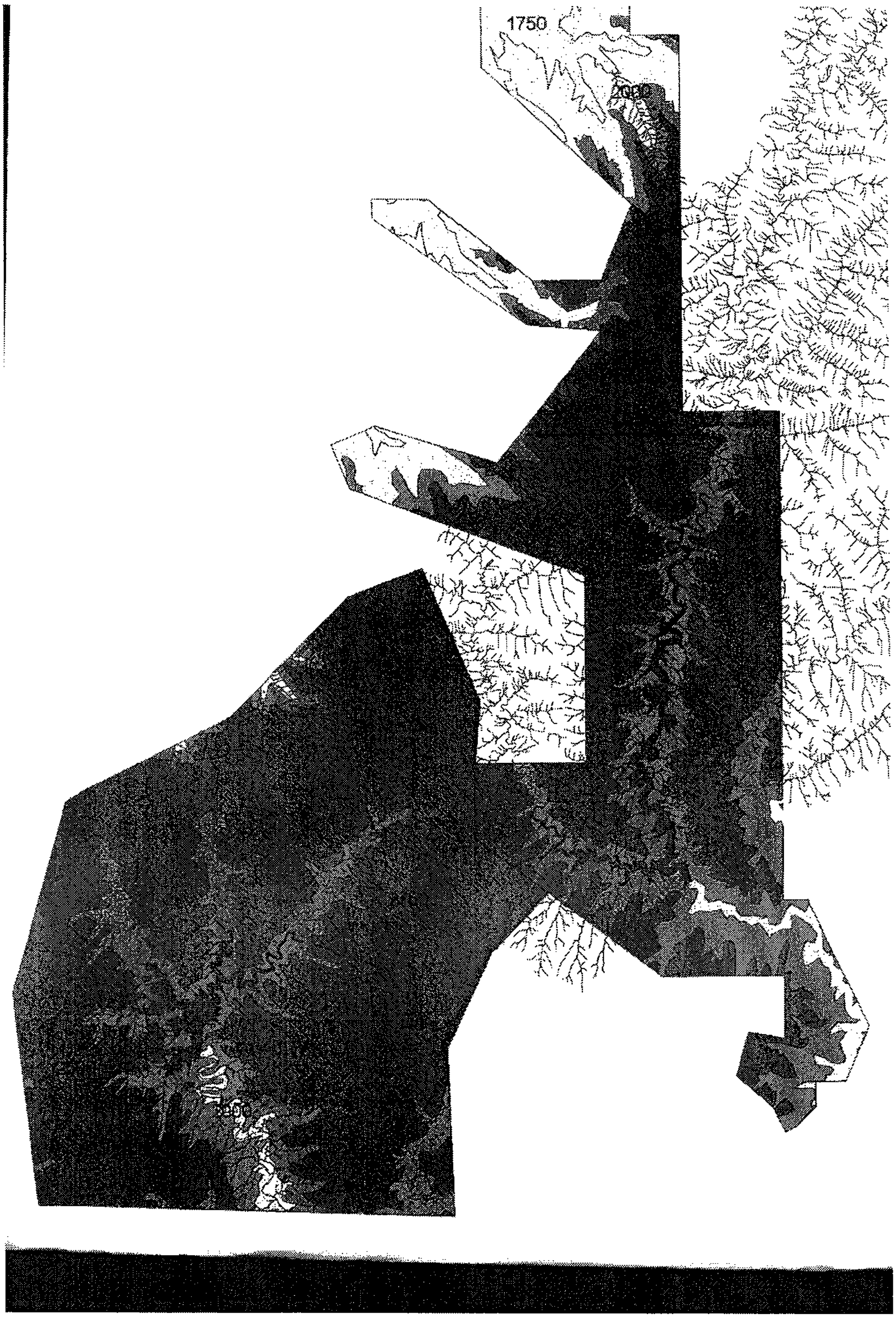


Figure 5.2-1: Drainage Patterns and Topography.

5.2.2 Climate

Frosts are severe in winter. The temperatures for Thaba-Tseka as indicated in Table 5.2-1 disguise mean minimum temperatures for May, June and July that are often below freezing, with an absolute minimum temperature typically around minus 10°C.

When combined with high winds, the wind chill factor can bring the effective temperature to minus 20°C. On average, nighttime frosts at Thaba-Tseka occur over a 6.5-month period from May to October, at Katse Dam, temperatures of minus 15°C have been recorded (Mohale Infrastructure Consultants, 1995).

The average annual precipitation varies considerably, depending on the influence of the topography. The deeply incised valleys tend to funnel prevailing winds, significantly altering the prevailing direction. In the Mohale area, a prevailing northerly wind direction predominates with the greater frequencies from the northwestern sector. Prevailing winds in the Highlands generally strengthen through the day from 08h00 to mid afternoon, after which wind speed gradually decreases. Nighttime is mostly still, particularly the period from midnight to dawn. Wind gusts up to approximately 120 km/h have been recorded, with prolonged gusts of 35 km/h lasting up to six hours (Lahmeyer, MacDonald Consortium, 1986).A climatic feature of the area is that its climate is less harsh than that experienced in the Katse/ Thaba-Tseka region with slightly less severe winters, a shorter period of frost risk and a higher and slightly more reliable rainfall, fig. 5.2-2 and 5.2-3.

Table 5.2-1: Climatic Statistics of Thaba Tseka (2160 m.a.s.l)

	Months											
Mean	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall (mm)	72	94	70	38	26	8	10	14	22	57	65	84
Temperature(°C)	16.4	16.2	12.6	11.5	7.8	5	5.8	8.2	9.1	10.8	15.6	15.1

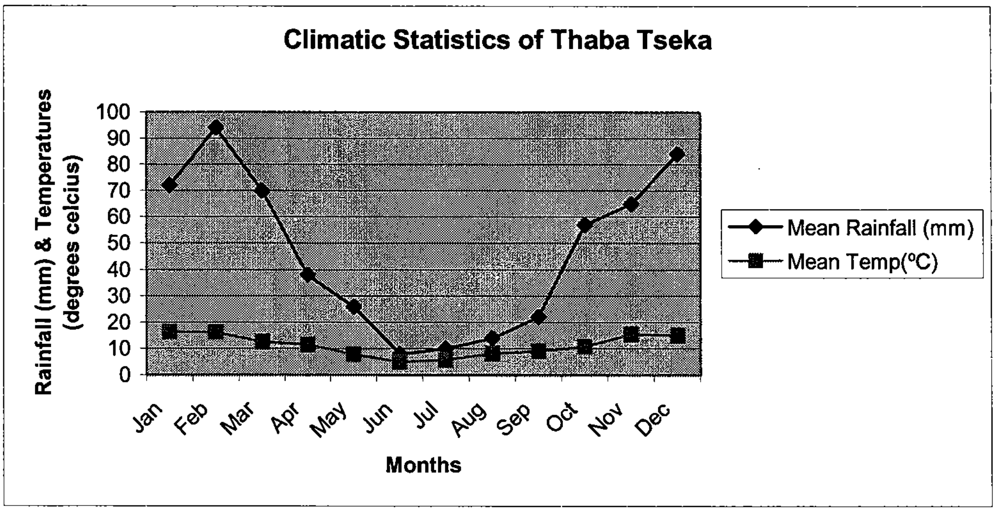


Figure 5.2-2: Climatic Statistics of Thaba Tseka

Table 5.2-2: Climatic Statistics of the Project Area

	Months										
Mean	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Nov	Dec
Rainfall (mm)	160	138	46	26	15	10	15	34	61	98	125
Temperature(°C)	17.6	17	14.2	13	9.5	6	6.5	7.5	10	16.8	17.4

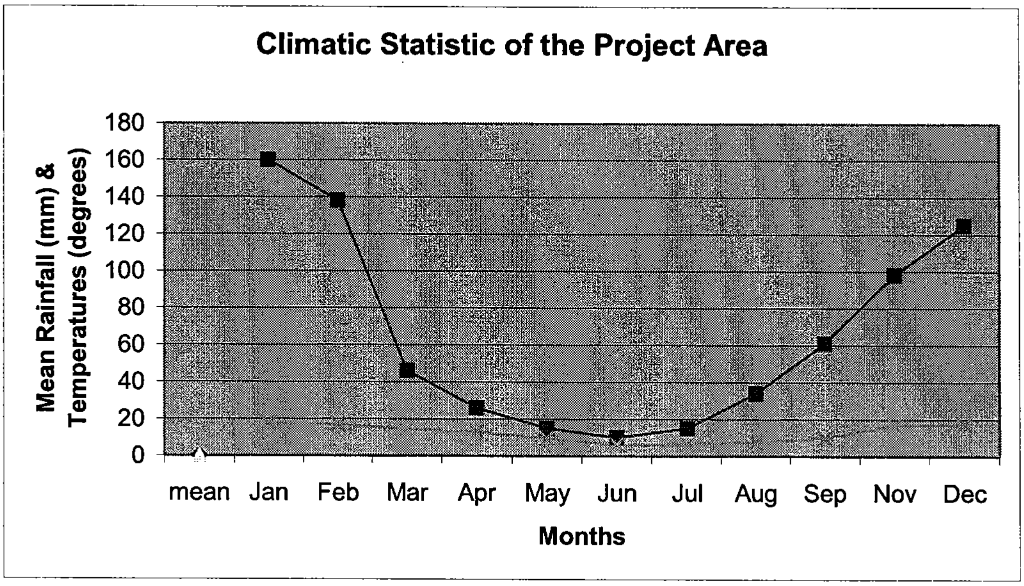


Figure 5.2-3: Climatic Statistics of the Project Area

5.2.3 Structural Geology

The Lesotho Highlands are an elevated area comprising resistant basalt rocks. The basalts are flat lying with dips of up to 1° regionally caused by a gentle synclinal structure. Steeper dips can be observed locally, which are probably related to the superimposition of laterally discontinuous flows. The base of the basalt flow sequence at the contact with the underlying sedimentary rocks is believed to be roughly basin shaped in the central part of Lesotho.

Linear discontinuities in the form of faults, fractures, shear and joints are imposed upon this broad basin shaped synclinal structure. The preferred orientation of these discontinuities is approximately E – W to ESE – WNW. Orientation of NNW – SSE and NE – SW are also common. Dolerite dykes along part of their length occasionally follow the major discontinuities. A good example is the relatively persistent Jorotane dyke lying 4.5 km north of the Mohale dam wall and passing some 200 m in front of the Mohale Tunnel intake structures (fig. 3.3-1; 3.3-2 and 3.3-3).

i.) Seismotectonics

It is believed that predominantly east-west compressive stresses in the basement rocks below the Karoo sediments cause deeper earthquakes. In general, earthquake activity cannot be related to surface geology.

Most earthquakes are believed to occur below the Karoo sediments. It is believed that these earthquakes possibly occur due to stress relief associated with isostatic rebound following the erosion of superficial formations. The primary source mechanism for these earthquakes may be the differential vertical movements.

ii.) Seismicity

There is no known volcanic activity in the area and the last known volcanic events are believed to be the emplacement of kimberlite pipes and dykes during the Cretaceous Period.

A total of about 100-recorded seismic events have occurred within a 150 km radius of the Mohale dam site. The largest event recorded within 150 km of the dam site was the M 5.5 Zastron earthquake, which occurred about 143 km south-west of the project area in 1957. Three recorded earthquake epicentres occurred within a 30 km radius of the Mohale dam site. These latter events are believed to have been mining induced. Nevertheless, they are indicative of crustal stresses and earthquake capability in the region. Earthquake focal depths vary throughout the region from zero to more than 60 km. The distribution of historical earthquake epicentres in Lesotho indicates that the majority of earthquakes occur in the southwest half of the country, including the Mohale site area.

5.2.4 Hydrology of Phase 1B

The hydrology of both the Mohale and Bokong areas is characterised by dendritic drainage patterns with high yields due to rapid runoff from steep slopes. The

Lahmeyer, MacDonald Consortium, study, (1986), showed that the catchment areas that had the highest mean annual precipitation also had the highest mean annual runoff.

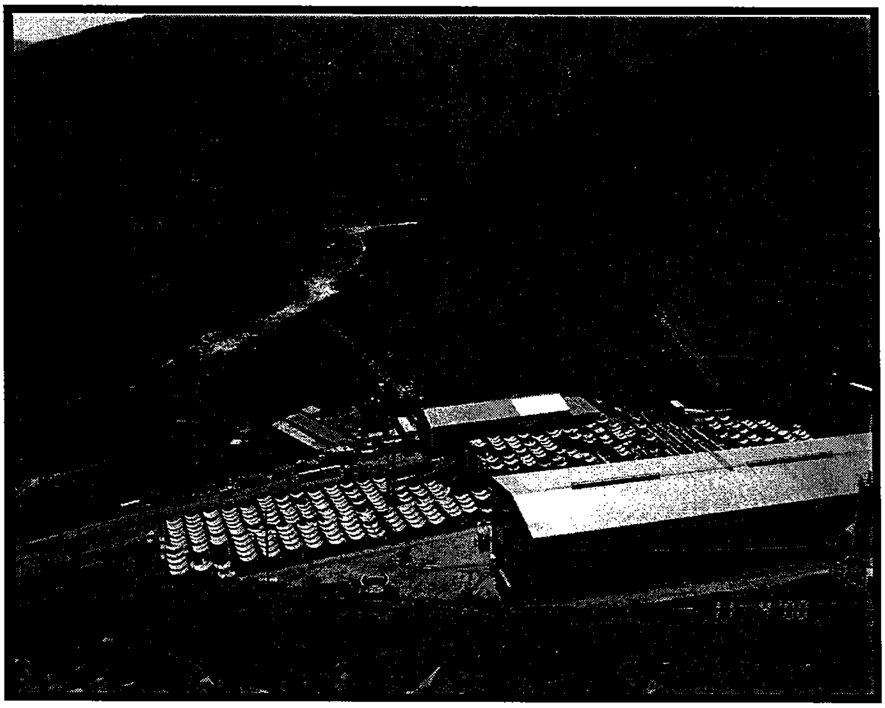


Figure 5.2-4: Medium flowing Senqunyane River. The Precast Factory of the Mohale Tunnel is clearly shown. Photograph was taken facing north from the contractors offices.

Rainfall occurs predominantly in the form of thunderstorms and is of high intensity and short duration. The nature of the rainfall, the rapid movement of water off the steep slopes and thin soils, results in a quick drainage reaction time in relation to surface runoff (fig. 5.2-4). With the Matsoku River, highly variable, wet, transitional and dry seasons are identifiable from the hydrological record. The wet/rainy season extends from December to March, while the dry season usually extends from June to September. Runoff occurring during the dry or transitional months is often the result of snowmelt. One of the characteristic features of the river system in the area is the highly variable flow regime. The historical hydrological records show approximately three elevated flow events per month during the wet season. A gauging weir (G42) is located on the Matsoku River at Ha Seshote approximately 15 km upstream from the confluence of the Matsoku and Malibamatso rivers. The record is regarded as reliable up to flow depths of 0.9 m which approximates 50 m^3/s. Comprehensive flow records are available from this station and these are summarized in Table 5.2-3.

Table 5.2-3: Hydrology at G42 (MDP Yield Report, 1995)

MAP	MAR	Area	1:1 Year Flood	1:2 Year Flood	1:5 Year Flood
759mm	110.9x10	679 km^2	25hrs peaking at $92m^3/s$	38 hrs peaking at $137m^3/s$	38 hrs peaking at $206m^3/s$

The catchment area is characterised by steep slopes, thin soils and minimal agriculture. By contrast, areas of low mean annual precipitation occurred over areas with better soil cover and a higher proportion of agricultural production.

Sudden flash floods may occur. The Senqunyane and Bokong rivers run through deeply incised gorges, with a number of small falls interspersed with deep pools. There is evidence of heavy scouring discharge during rainy periods. Small sandbanks are evident on the low-water riverbank. During the dry season, the river is slow flowing and travels from a deep to a relatively still pool by way of shallow riffles, larger rapids and some small falls. Lateral drainage across the valley floor is unimpeded, and occurs mainly by surface flows or along natural channels between adjacent rock masses.

The existence of water tables is not always evident, although shallow temporary phreatic aquifers may develop during the rainy season. A number of springs are located in the villages, which supplies water for livestock and domestic use.

Hydrology of the Catchment Area

5.3.1 General

The Senqunyane River is located in the Maloti Mountains in central Lesotho. The Mohale Dam site is located in the uppermost catchment area of the Senqunyane River, about 65 km due east of Maseru. The Mohale Dam will regulate runoff from a single catchment with an area of 938 km^2.

The climate is generally sub-humid with about 80% of the precipitation assigned to the summer months, October to March. Thaba Tseka is located 40 km east of the dam

GEOLOGICAL CONDITIONS AND ENVIRONMENTAL IMPACT OF THE MOHALE DAM, LESOTHO HIGHLANDS WATER PROJECT

By

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Abstract

A study of the roles played by geological forces in shaping man's interaction with the earth!

The Mohale Dam forms the second major dam to be built in Lesotho and is the main element in the Phase 1B of the Lesotho Highlands Project (LHWP). This 145-m high concrete faced rockfill dam (CFRD) is situated on the Senqunyane River immediately downstream of the confluence with the Likalaneng River. The Mohale Dam basin is connected to the Katse Dam Basin by the 32-km long Mohale Tunnel, which has been constructed under separate contract. The geotechnical investigations were undertaken in order to decide on whether the Mohale Dam should be a concrete faced rockfill dam instead of a roller compacted concrete (RCCD) or concrete arch dam (CAD).

Some of the reasons for selecting a CFRD have been the fact that such dams are very stable during earthquakes, as these are always anticipated for large dams; the availability of the construction materials, in this case the basalt rock sourced from the 2 quarries, located within 2 km of the embankment; the CFRD is less prone to construction time –delay risks than other types of dams, such as adverse weather conditions (The time schedule for construction for Mohale is five years) and the location of haul roads and quarries which minimized the environmental impact of the construction on the surrounding area. The availability and proximity of large quantities of rock played a pivotal role in the successful completion of the Mohale Dam. A broad ridge of basalt at the confluence of the Senqunyane and Likalaneng Rivers has been used as the main quarry. A broad colluvium filled oxbow in the quarry area made it unique because the bulk rock and the impervious clay materials have been mined from the same area and the same haul roads have been used for transportation. The impervious clay has been used for construction of both temporary and permanent cofferdams. It has also been used for a blanket purpose at the upstream toe of the dam. A 70-m thick coarse-crystalline jointed doleritic olivine basalt formed a secondary quarry on the steep slopes of the Likalaneng River valley. The rock has been mined for the purpose of producing a concrete aggregate for the dam and other concrete structures and for filter layers for the Mohale Dam. Another advantage of the CFRD is the fact that deep foundation excavations can be avoided but not compromised. Many outcrops in the areas have led to the plinth structure, which forms a contact between the rock foundation and the concrete face, being founded at shallow depths and hence avoiding high valley bottom stresses. The plinth structure has been placed on jointed competent doleritic basalt in the river. Consequently, there has been no deep excavation. Intrusive structures of mixed non-amygdaloidal basalt underlie the entire left abutment. A zone of breccia separates the basalt from the doleritic basalt. A typical sequence of basalt underlies the right abutment. There are some lineaments crossing the dam. These

lineaments have been treated with reverse filters and in some cases; joints have been performed in the plinth.

Two tunnels have been constructed on the left abutment for diverting the river. The provision of these tunnels is needed to allow construction to proceed in dry conditions. The concrete lined spillway is located on the right abutment. The purpose of this spillway is to allow anticipated floodwater to flow safely downstream when the reservoir is full. Topography has played an important role in keeping the spillway excavation very shallow.

The dam has required 7.8 x 10^6 $m^3$of material. The embankment of the Mohale Dam is a zoned rockfill dam. The rock has been mined from the two quarries mentioned earlier. The bulk of rock has been transported utilizing a fleet of 50-ton caterpillar 733 off-road dump trucks along access roads built to provide the most efficient trips from quarries to the dam. The development of the quarries has been organised in such a way that it has been the most economical in terms of transport costs and otherwise. The construction of roads has been ongoing as to the construction of the dam progresses. The bulk rock has been placed in 1 m layers on the upstream side of the embankment and in 2 m layers on the downstream. The graded filter layers below the face slab have been placed in 400 mm-thick layers, extending variably up to 4 m wide. The concrete face slab varies in thickness from 720 mm at the base of the dam to 300 mm near the crest. The upstream slope is built at a gradient of 1v: 1h, while the downstream slope is built at an overall slope of 1v: 1.45h, which includes the permanent access road down the face.

The Mohale Tunnel is located approximately 5 km upstream of the Mohale Dam wall. It comprises two intake structures: the lower and the upper intakes with operating levels at 2005 and 2050 m above sea level respectively. The tunnel has been excavated utilizing two tunnel-boring machines (TBMs): one from the intake side while the other advances from the outlet side.

The approach that has been adopted towards the environment aims at minimizing the adverse impacts on the Senqunyane River System while capitalizing simultaneously on the positive ones with the incorporation of the best compensation policies for the affected persons of the area. The adverse impacts put into perspective are such as, seismic activities possible during impoundment of any reservoir, possible breaching of the dam that can cause huge loss of human life and environmental damage; loss of land with its valuables due to the reservoir; permanent hazard posed by the reservoir; change in ground water behavior; displacement of communities and introduction of diseases. The positive impacts are introduction of all sorts of infrastructure (roads, clinics, shops, schools; etc.), provision of much needed jobs for poverty alleviation; increase in standard of education; and tourism boom. The environmental policies have been constructed in line with the world's experiences from other similar projects of the LHWP magnitude. While on the other hand taking cognizance that no two projects are the same, each is unique in its own right, the projects are governed by geological environment and socio-political situations of each country, specifically of the site of construction. The LHWP has tried very hard to always engage the civil society from all walks of life, in order to make it possible to achieve the LHWP goal, which has been to build dams and tunnels. However, the main challenge is to preserve the state of the environment during and after construction and impoundment respectively. The hydrological condition of the Mohale Tunnel during construction is appended in this work to highlight the hydrogeology of the basalt with a view to mapping out the possible impact of the Mohale Dam on to the groundwater regime. Measurements have been taken along the tunnel whenever water ingress has been encountered. The geology has been recorded and correlated with the surface. Spring measurements have also been appended, since they were carried out along the tunnel alignment, i.e. those located within 100 m on both sides of the alignment. Along the Katse shoreline, observations and monitoring of the groundwater behaviour have been undertaken and this has involved interviews with the local communities.

Below is the summary of the activities that this work has tried to cover with a view to realizing the aim and objective of this work:

- Description of work undertaken and terms of reference
- Sources of information (Literature information gathering and discussions with relevant people)
- Description of the site and surrounding area based on an inspection and walkover survey of the site
- Regional and site geology and any feature of the geology and known material properties which will is considered to impact on the project
- Groundwater conditions and hydrogeology/hydrology and climatological conditions
- Records of previous investigations undertaken at or close to the site, if any
- Site history and past uses of the site and adjacent sites (land-use in general.)
- Site characteristics and locality maps (topography/morphology)
- Records of quarrying, landfill, any impacts etc.
- Records of service and underground structures
- Field reconnaissance of the general area of the project noting particularly:
    1. Evidence of groundwater
    2. Location of surface waters (springs/streams/rivers) and evidence of flooding
    3. Behaviour of any existing structures on the site or neighbouring structures
    4. Exposures in quarries and borrow pits
    5. Area of instability (slope stability in rock/soil)
    6. History of the site
    7. Geology of the site (structural, lithological, etc.)
    8. Information from available aerial photographs
    9. Archaeological evidences
    10. Seismological history of the area. Type of field equipment used

- Clear conclusion regarding the conceptual geological and environmental impacts

- Overall strategy and objectives for subsequent intrusive investigation
- Purpose and scope of the investigation
- Names of all consultants and sub-contractors used
- Dates between which field and laboratory work was conducted, whenever possible
- A factual account of all field and laboratory work
- Exploratory hole records (boreholes, trial pits, window sample holes), including grid co-ordinates and ground elevation
- In-situ test results (site laboratory tests, etc.)
- Laboratory test results including any contamination test results
- Results of groundwater level monitoring and any geo-environmental monitoring
- Specialist sub-contract test results, (static or dynamic cone penetration test, geophysics, etc.)
- Site plan showing locations of exploratory holes (drillings, pits, quarries etc.)
- The provision of data in electronic format
- A review of the field and laboratory work
- Detailed description of all formation including their geological context, physical properties and their deformation and strength characteristics
- Comments on irregularities such as contact zones, podsols, depressions, cavities and boulders
- Sub-surface profiles showing the differentiation of the various formations
- Identification of geological, geotechnical or other hazards
- Depth of the groundwater table and its seasonal fluctuations
- The range and grouping of any derived values of the geotechnical data for each stratum
- Summary tables for chemical contamination data with listings of selected assessment criteria, wherever possible
- Tabulation, photographic and graphical presentation of the results of the field and laboratory work in relation to the requirements of the project and if deemed necessary histograms illustrating the range of values of the most relevant data and their distribution
- A review and summary of the derived values of geotechnical parameters
- Any proposals for further field and laboratory work, with comments justifying the need for this extra work and a detailed programme for the extra investigations to be done
- The assumptions, data and method of verifying the safety (environmental impacts) and serviceability of the geotechnical construction (e.g. method of calculation), including:
    1. A description of the site and its surroundings
    2. A description of the ground conditions
    3. A description of the proposed construction, including anticipated loading and any imposed deformations
    4. Site selection criteria
    5. Design values of soil and rock properties, including any necessary explanation for their selection
    6. Statements on any codes or standards used
    7. Statements on the suitability of the site for the proposed construction and the level of risk assumed in the assessment
    8. Geotechnical design calculations, photos and drawings
    9. Design recommendations
    10. A note of any items to be checked during construction or required maintenance or monitoring
    11. A statement of sequence of construction operations envisaged in the design. Alternatively, the design report may state that the sequence of construction is to be decided by the contractor

12. A plan of supervision and monitoring as appropriate for the type of project, stating acceptable limits for the results to be obtained by the supervision and specifying the type, quality and frequency of supervision

- Where appropriate records of preliminary prototype scale testing
- Records of ground conditions encountered
- Test records relating to additional geotechnical testing, materials testing and proof load testing, in particular piles and anchors
- Any non compliance records or similar raised during course of the project
- Monitoring records on the effect of the works on adjacent properties etc. (Environmental Issues)
- Where the observational method was used this shall be fully recorded
- Records of temporary works, particularly where these left in-situ or may affect future developments
- Detail of inspection regime including scope and frequency
- Proposed measures to be taken during the lifetime of the facility for maintenance of drainage, corrosion protection, etc.
- Long-term settlement records, water levels, piezometers, etc.
- Identification of critical elements of the structural system that may pose thread to the safety of the dam.

An appendix on information important structures in the dam and tunnels on the water ingress (groundwater condition of the site as derived from the inflows and springs)

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1 INTRODUCTION

Dams have been constructed for centuries for various reasons ranging from flood management, irrigation, and hydropower production to domestic and industrial water supply. Arguably, half the world's large dams have been built exclusively or primarily for irrigation and the dams have indeed made a significant contribution to human development and the benefits derived have been quite considerable. However, their geotechnical impact on the environment has raised a loud outcry from different sectors of the world population without concrete evaluation or exploration of individual projects. Concerns have recently been raised and have now let to global anti-dam campaigns to the point where future dam construction is in doubt. It is a reality that a dam built across a river indeed brings about certain environmental changes in ecosystems. These changes will be obvious in the river valley upstream of the dam, which will be flooded as the dam impounds a new reservoir behind it. A less obvious feature is that the river downstream will also be significantly changed. Large dam projects such as the Lesotho Highlands Water Project (LHWP) are highly unique in their design, geological setting and the construction materials used to build them. Their environmental impacts are also site specific. Some large dams in the world have created lakes hundreds of kilometres long in areas, which had large local populations. These projects bring about major impacts on the human, animal and plant population of the catchment area. These impacts have been well documented but it would be wrong to make an assumption that all dam projects have similar major impacts on the environment. The environmental impacts are heavily dependent on the geology of the site. This is clearly demonstrated by the huge budget made to buy expertise from all over the world to carry out the geotechnical investigations. From sufficient knowledge of local site conditions, gained from the investigation, the design can follow. These investigative procedures cover those important geotechnical investigations carried out at the LHWP sites and assessments of the geological conditions with respect to the environmental impacts. Specifically, this work deals with the environmental conditions of Mohale Dam and to a lesser extent with those of the Mohale Tunnel. However a detailed feature statistic analysis is herewith attached as appendix, to give the overview of the water ingress behaviour during tunnelling. The statistics cover the tunnel from the portal to kilometre 16. From the Katse Dam impoundment experience, seismic activities have played a major role in environmental impacts and this has led to a whole village (Mapeleng Village) being relocated and to a reassessment of the influence of impoundment on the water regime.

There is also the very important issue of the geology of the dam site including the foundation for the dam itself and the sites for other structures such as the spillway, diversion tunnels and outlet works. The investigations are based on the following concerns:

- Whether the dam foundation has sufficient strength and durability to support the type of dam proposed
- Whether the foundation is watertight,
- And if not, how much grouting is required
- Whether the spillway needs concrete lining.

Another very important issue relates to the geology of the area and includes the following very important points:

- Whether the basin to be impounded is watertight or whether there are areas of cavernous limestone, palaeosols, contacts between flows, fissures and banding in lavas present, this may lead to the dam not retaining water.
- Whether the slopes of the basin to be impounded are stable and, whether there is a possibility of a landslide into the reservoir that may cause a wave of water to be pushed over the top of the dam, as it was the case in a dam in Italy.

Another important consideration is the exploration of the sources of construction materials necessary to build the dam. Extensive site investigations are being carried out to give answers to the above questions. No two dam sites are identical as far as geology is concerned, hence each new dam construction project must be investigated individually. Some dam sites may appear to be uniform in their geology but each has its own site-specific geology. It implies that although the geology may be very similar for all sites, each site is explored individually. To achieve these site-specific characteristics the following are employed:

- Geological mapping of the surface rock outcrops and bedrock

- Topography of the Project Area
- Excavation of test trenches, and trial pits
- Geophysical surveys, for example, seismic refraction, which was used in the case of the Mohale Dam
- Diamond core drilling and core logging

The site-specific geological investigations are carried out to make it possible for the engineering geologist to construct a geological model of the site. The dam designers use this model as a basis on which they can design a safe and economic dam structure appropriate to the geology of that particular site.

Experience in engineering geology has shown that it is very important to realise that even the most comprehensive site investigation programme cannot hope to reveal all the significant geological features of the site. It is therefore of critical importance that the actual geological conditions revealed during construction be on the continual bases compared with the geological model of the site derived from the investigations. Quite commonly unexpected geological conditions are revealed during construction, which results in amendments or changes, which need to be made to the original design. A record of the site geology "as found" during construction is also very important. The Mohale Dam is also passing through the phases of geological/geotechnical investigations, design, and construction. The next phase, which follows is operation and maintenance, in which the geological records are of cardinal importance in these phases.

The Mohale Dam is built a few metres downstream from the confluence of Senqunyane and Likalaneng Rivers. The two quarry sites are located within the dam site. The river is diverted through two tunnels so that work can take place on the foundation. The following activities take place on the foundation before commencement of dam construction:

- Stripping of overburden along the plinth line
- Preparation of the dam foundation
- Treatment of the two linear features on the left bank
- Grouting of the foundation

The construction material has then been transported to commence with the building of Mohale Dam. The hauling distance is significantly short ca. 2 km to and fro. Basalt rock is dumped and compacted. Compaction is carried out to ensure that the fill, as placed, displays maximum possible density.

The Mohale Dam, when completed, will supply Katse Dam with water to meet the export demand and it will store water whenever Katse Dam is full. In this case the water will flow in either direction.

Assessing and managing the social and environmental impacts of a project of the magnitude of the LHWP is of utmost importance in order to ensure that the project is beneficial to the nation as a whole as well as to neighbouring countries, such as the Republic of South Africa (RSA).

1.1 Aim of the Study

The study takes the Mohale Dam, including its ancillary elements, as a case study to evaluate its geological and geotechnical conditions and impacts on the environment. The study focuses primarily on:

- Site geology
- Geotechnical site investigations
- Quarrying and quarries
- Construction and construction methods
- Environmental impact and proposed mitigation measures

1.2 Objectives of the Study

The objective of the study is to highlight the significance of geology and geotechnical operations and how both dictate site selection and hence the environmental impact. The study started as part of the LHWP during the researcher's geological engagement in the LHWP (1992-2003). Special attention has been given to the rehabilitation works as well, as to how the geological conditions will impact on the environmental decision making prior to, during and after construction of the tunnels and dams of the LHWP. Some fundamental principles of geology and current geological engineering are outlined in this study, which proceeds to examine the interaction of geology with the environment.

It is a well-known fact that every activity of construction either above or underground involves a total environmental change. Anderson and Trigg, (1976), describe the total environment to include the ecology of the site, of which man himself is a part, and its visual appearance, but from the point of view of safety, and largely of cost, the geological factors in the environment are the most important site considerations in civil engineering. In their words, the most significant of these geological factors are those affecting stability within the planned structure. They proceed to say that the basic task of the geologist is to collect and evaluate the data relating to the stability of the geological environment, taking into account the fact that this environment is itself changed by construction activities and the operation of the structure thereafter.

The presence of man-made structures in the environment may vary from negligible to critical depending on the size of the structure that is being constructed. On the geological time-scale, every site is unstable, that is, it is liable to be affected by geological processes. On the historical time-scale, some sites are geologically stable, that is, short-term processes affect them.

Geologists are invariably involved in environmental impact investigation. This study thus focuses on the investigation and evaluation of the consequences to the environment of Mohale Dam and Tunnel engineering works. This involves a description of the LHWP and its impact on the environment, particularly noting any positive and adverse effects. The author considers the involvement of geology as constituted in the control or reduction of the effects of geological processes or hazards.

The aim of this study is to demonstrate the fundamental role of geology in identifying, understanding and solving multidisciplinary, long term, environmental challenges brought about by the construction of large dams and tunnels, with special reference to the Mohale Dam and Tunnel and specifically slope stability and hydrogeology. This study provides examples of how geological surveys based on environmental issues can form a basis for employing geological criteria in the future planning for the LHWP.

1.3 Previous Works

The Government of the Kingdom of Lesotho appointed Lahmeyer MacDonald Consortium while the Government of the Republic of South Africa appointed Olivier Shand Consortium jointly to undertake the feasibility study. The study took place from 1983 to 1986. The Lahmeyer MacDonald Consortium had the task of undertaking the study for the Katse Dam and Transfer tunnels whereas Olivier Shand was responsible for the Mohale Dam.

Generally, the geotechnical and construction studies focused on the description of the nature and the engineering properties of the basalt of the Lesotho formations and the identification of possible construction sites.

The geotechnical studies at Mohale Dam concentrated on the geological mapping of the dam site and adjacent potential rock quarry and soil-borrow areas. The geotechnical investigation, along the Mohale Tunnel route and at potential quarry areas, was carried out to provide geotechnical information for the design and the construction of the tunnel. The geotechnical investigation for the Mohale Dam and Tunnel were carried out in three stages these included a feasibility study, planning study and tender design. The feasibility and planning investigations were undertaken effectively, and the technical feasibility of the selected site for the tunnel and dam types confirmed, respectively. The mapping procedures were a combination of airphoto interpretation as well as detailed field mapping of rock exposures. The steep-sided valleys revealed good outcrops.

The subsurface investigations were carried out by means of nine boreholes drilled at the likely dam and spillway sites, three boreholes drilled in potential quarry sites and one hole drilled at the portal site for the intake to the Mohale-Katse tunnel.

Fourty-nine test pits were dug to supplement the geological mapping of soil-borrow areas.

Three main types of soil deposits were identified in the Mohale dam basin. They were:

i. Relatively small and discontinuous deposits close to river level, containing predominantly alluvium and colluvial slope wash materials.

ii. Isolated bodies of variable size, generally occurring above river level, containing predominantly residual material with subordinate alluvial and colluvial materials.

iii. Abandoned raised oxbows, again of variable size, containing residual, colluvial and alluvial materials.

In all cases, it was found that the weathered rock beneath the surface material represents a significant proportion of the deposit. The alluvial and colluvial material have often weathered since deposition making it difficult to determine their origin and to differentiate between them and the underlying weathered rocks. A fourth soil deposit type of minor extent occurs as bars in the present river channels. These deposits consist of mixed alluvial sands and gravel.

In conclusion, the necessary investigations were carried out to a satisfactory level and the environmental protection procedures were put in place. The researcher has observed the broad geology of the area and has mapped out the link between geology and the environment, basing his findings on the geological investigations that were undertaken by LHWP and his fieldwork on the project.

The Vaal Augmentation Planning Study was undertaken in 1995 with a view to finding the alternative to the LHWP. A number of alternative schemes were considered:

- The Caledon Cascades Scheme to redirect water at the Lesotho border into the Caledon River and through a series of 19 dams transfers this water to the Vaal River System.
- The Canal Scheme with three canal options (Aloedal, Goedemoed and Upper Caledon) to transfer water of the Orange River into the Vaal River System.
- Water transfer from the Zambezi River
- Water transfer from the Tugela River
- Sea water desalination
- Reducing the amount of water used for irrigation etc.

The LHWP remained the most viable of them all. Another scheme that looked viable but smaller was the Oxbow Scheme. The scheme was envisaged to consist of five reservoirs with approximately 102 km of tunnels to transfer water via a hydropower station on the Hololo River at Tlhaka to the Kroonspruit River in the RSA. The scheme was later abandoned due to the exponential growth of water demand in the 1980s in RSA.

A similar study was carried out by Olivier, (1976). Although, he touched on "Some Engineering Geological Aspects of Tunnel Construction in Karoo Strata with special Reference to the Orange-Fish Tunnel", his thesis focused more on the engineering side whereas this thesis on "The Geological Conditions and Environmental Impacts of the Mohale Dam" emphasises on geology and environmental awareness.

The report titled the Landslide Loss Reduction: a Guide for the Kingston Metropolitan Area: Jamaica that was prepared by Ahmad et al., (1999); discuss the landslides and environmental impacts in Jamaica. This work is intended to promote an understanding of the potentially dangerous consequences of development and urbanization in geologically sensitive areas. Another similar paper is one that was prepared by the American Geological Institute, (1999) under the auspices of the Environmental Geosciences Advisory Committee, titled "The Role of the Earth Sciences in the National Institute for Environment".

Workshop 1, which the researcher attended at the Lesotho Sun Cabanas, Maseru, on the 25 and 29 May 1998 focused on the topic: "Review of the Current State of Knowledge of the Seismo Tectonic Setting of Lesotho and it's Significance in Predicting Seismic Design Parameters for the Katse and Mohale Dams and Further Phases of the LHWP". Fieldtrip inspections en route to Katse Dam and Mapeleng Village on the 26 May 1998 and en route to Thaba-Tseka, Mashai, Marakabei and Likalaneng on the 27 May 1998 completed the workshop.

Field inspections of alleged volcanic eruption sites, Mohale Dam, and environs were undertaken on the 28 May 1998. On the 29 May 1998, workshop 2 was held at the same venue as before. The researcher attended both workshops and all field trip sessions. The following papers were presented:

- A Review of Critical Aspects of the Structural Geological Setting of the Kingdom of Lesotho, Prof. I. W. Hälbich
- From Karoo Dolerite to Drakensberg Basalts: Structure, Volcanics and Seismicity, Dr L. Chevallier of the Council for Geosciences, Bellville
- Of Diamonds, Dinosaurs and Diastrophism: 150 million Years of Landscape Evolution in Southern Africa, Prof. T. C. Partridge of WITS
- A Review of Earthquake History and Interpretation of Seismotectonic of the Kingdom of Lesotho, Prof. J.H. Hartnady of UCT

- The Lesotho Volcanic Event of February 1983, R.R. Maud of University of Natal, T.C. Partridge of WITS, J.N. Dunlevey of University of Durban West-Ville

- Seismic Parameters used in the Design of Large Dams, and the Role of Reservoir-Triggered Earthquakes, Dr Clarence Allen of the Seismological Laboratory, California Institute of Technology, Pasadena, California

- Review of the Seismic Hazard Analysis and Selection of Seismic Design Parameters for Mohale Dam, R.J. Anderson of Harza Engineering Company Chicago, Illinois, USA

- A Summary Review of the Reservoir Induced Seismicity at the Katse Dam, Kingdom of Lesotho, Dr G. Graham of the Seismology Section of the Council for Geosciences, Bellville

- Brief Report on the Volcanic Activity under the Step-up Transformer of the Thaba Tseka Line, near Pony Farm of Molimo Nthuse, Dr G. Prasad, NUL

- The Seismic Response of the Katse Dam and its Appurtenant Works, Mr. D. Develay, Chief Katse Dam Designer of LHC

All the above papers are specialized. The researcher finds them to be very interesting and they address some of the issues that this thesis focuses on. As a result, they form part of this work due to their invaluable information. The workshop has also been very interesting while listening to geologists, engineers and seismic experts exchanging views on very important issues that may affect our environment with probable devastating results. The stops during the field trips have been made at 24 locations in total, from the 26 to the 28 May 1998.

The environmental studies undertaken by various groups on the LHWP and other similar projects elsewhere have been also studied. The researcher likes specifically to take into account the Phase 1B Environmental Impact Assessment (EIA) of 1996. The study has been prepared during the planning period and it is based on a number of studies specific to Phase 1B. It takes into account lessons learned from the implementation experience of Phase 1A. It is concurrently being prepared with the early detailed design stages of the various engineering components.

1.4 Methodology

The researcher uses the geological/geotechnical information and data gathered from excavation of quarries, trenches, test pits, adits, dam foundations, piezometric measurements, seismic refraction surveys, geological mapping of the dam site and along the Mohale tunnel route, tunnels and laboratory investigations. The field geological mapping is carried out to define the geological conditions of the Mohale Catchment and Dam Site and to confirm some of the structures seen on the maps. A substantial amount of information is also being collected along the Katse shoreline villages concerning the changes in the water regime of the project area. The researcher has been involved in the construction of the Katse telemetric seismic stations and their monitoring. He has also attended relevant workshops and field excursions provided by the Lesotho Highlands Development Authority (LHDA) in conjunction with the consultants employed. He has also been involved in the geological mapping of the Katse Dam foundation and access galleries. He has been employed as quarry geologist and he has been LHDA's supervisor of the geotechnical investigations of the Mohale Dam.

The following data and information has been used to address the main objectives:

i. Monitoring of water springs along Mohale tunnel route to establish the impact of the tunnel excavation on the springs (carried out by the environmental officer)

ii. Face mapping in the Mohale tunnel and feature analysis, core logging and tunnel-spoil logging to establish the encountered geology of the tunnel for payment purposes (carried out by the researcher and the engineering geologist)

iii. Monitoring of the LHDA telemetric seismic stations and analyzing seismic events. (Carried out by the researcher and seismic technician)

iv. Mapping of linear features at Mapeleng village during relocation of the whole village following the collapse of poorly constructed houses in the wake of the first reservoir-induced seismic activities (carried out by the researcher)

v. Performed point-load tests as control measures on the remainders of the cores sent for testing at the Council of Science and Industrial Research (CSIR) and the University of the Witwatersrand (Wits), (carried out by the researcher)

vi. Field trips along the Katse Reservoir Shoreline and around the Mohale Reservoir Site in order to make field observations on slopes accompanied by interviewing villagers along the traverse about any change in their environment (carried out by the researcher)

vii. Compiled seismic inventory map (carried out by the researcher)

viii. Photographing areas in and along the valley of the Mohale Reservoir (carried out by different engineers personnel)

ix. Reading and extracting information from relevant text books of engineering geology

This information and data were combined with environmental and geological engineering aspects.

1.4.1 Scope

- General geology of the basalts
- Main characteristics of Lesotho Basalt formation
- Type of hydraulic structures built and being built in Lesotho
- Conductivity of Lesotho Basalts formations in general
- Rock mass properties of Lesotho Basalt in general

Dams with Reference to Mohale Dam

- The general location merits and disadvantages
- Factors that influenced the choice of the location design method of construction and efficiencies.
- Hydraulic factors studied
- Studied rock mechanics parameters.
- Influence of basalt conductivity and siltation effects on design and structural life expectancy.
- Design justification of the spillway and efficiency.
- Environmental Impact

Geology

The study shall detail the geological aspects of all sites as well as the engineering geology aspects. The geological study shall include details of the physical effects on the mineral and of the rock mass formations.

Hydrogeology and Hydrology

The study shall detail all hydraulic features of the Lesotho basalt and shall go into more detail for site elaborating on the Mohale site particularities. The conductivity of basalt shall be addressed in detail especially when evaluating Mohale dam sites.

Sediment movement shall be discussed showing silting period and measures taken in design for cleaning the reservoir base and the lower outlet structure. Such an approach can be adopted to evaluate sediment transportation for the tunnel inlets. The catchment's basin shall be investigated for silt collection and sedimentation.

Rock Properties

The study shall describe the general properties of Lesotho basalt. Emphasis shall be on rock mass mechanics properties and the method of evaluation of such properties showing its merits and the uncertainty of testing and evaluation methods. The study shall also focus on measures taken for treating the foundations and materials used in the construction to ensure the safety of the Mohale dam.

The study shall show the monitoring-scheme adopted for the Mohale dam structure in terms of instrumentation. A comparison between the investigation-stage findings and the findings during construction shall be tabulated and a recommendation for future investigation plans shall be devised and explained.

Environmental Management

The study shall look into the mitigation measures in place during construction and after. The study shall look into the post-construction rescue measures adopted in case of any disaster.

The Domain of Comparison and Evaluation shall include:

- Historical records
- Maps, aerial photographs and aerial surveying
- Seismic surveying
- Preliminary investigation schemes
- Detailed investigation scheme
- Quality control measures applied during construction and any particular investigations by either contractor or consultants and the reason for such investigations.
- Methods of boring and diameter of boreholes
- In-situ testing
- Laboratory testing
- Post-construction monitoring schemes

The achievement of the purpose of the research shall be judged according to the degree of accuracy with which the study managed to construct the general view of the Lesotho basalt formations properties, the details of the Mohale Dam particularities and the conclusions relating to the geological conditions on the environmental impact.

1.4.2 Data Source

The study focuses mainly on the Mohale Dam. The observation/data gained from the Mohale Tunnel and Katse Dam as well as the catchment areas are also used. In all these areas of interest, the Lesotho formation dictates the type of techniques to be employed in excavations. The structures of the formation can be easily mapped due to little or no soil cover in the project areas of interest. The researcher extracts some of his information/data from the following reports:

I. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 1. General Project Description and Survey Data. Rodio (RSA), March 1997.

II. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 2. Water acceptance Tests. Rodio (RSA), March 1997

III. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 3. Test Pit Logs. Rodio (RSA), March 1997

IV. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 4. Laboratory Tests. Rodio (RSA), March 1997

V. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 5. Borehole Logs, Part A and B. Rodio (RSA), March 1997

VI. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 6.Seismic Refraction Survey Report. Rodio (RSA), March 1997

VII. The Lesotho Highlands Water Project, Mohale Dam Geotechnical Investigations Contract LHDA 2015, Volume 7. Borehole Core Photographs. Rodio (RSA), March 1997.

VIII. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Data File B7 and B9 Volume 1. Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

IX. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

X. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Annex B7. Geotechnical and Construction Materials Studies-Mohale Dam. LMC and OSC, April 1986.

XI. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Annex B11. Geotechnical and Construction Materials Studies-Petrology and Durability of Igneous Rocks. LMC and OSC, April 1986.

XII. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Annex B7, Drawings. Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

XIII. The Lesotho Highlands Water Project, Feasibility Study, Supporting Report B, Data File B7 and B9 Volume 1. Geotechnical and Construction Materials Studies. LMC and OSC, April 1986.

XIV. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume A Text. Geotechnical Investigation Report. LHDA, June 1995.

XV. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume B Appendices. Geotechnical Investigation Report. LHDA, June 1995.

XVI. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume 1. Dam Type Selection. Geotechnical Investigation Report. LHDA, June 1995.

XVII. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume 3. Drawings. Geotechnical Investigation Report. LHDA, June 1995.

XVIII. The Lesotho Highlands Water Project, Phase 1B, Planning Study, Volume A Text. Geotechnical Investigation Report. LHDA, June 1995.

XIX. The Lesotho Highlands Water Project, Contract 123 and 124 Tender Documents: Data for Tenderers for Katse Dam Volume 5, Geotechnical Report, Data for Tenderers for Transfer Tunnel South and Associated Files.

1.4.3 Field Evidence

Du Toit's extensive writings, culminating in the third edition of The Geology of South Africa, which appeared six years after his death in 1954, make it clear that he regards good field observation as the most powerful tool of the geologist (Partridge, 1997). Partridge emphasizes the importance of field observation in his address to a workshop held at Maseru Cabanas in 1997. He states that geology should be based primarily on observable facts, and models – so much the present fad in almost every branch of science, which is, in his opinion, useful in geology only insofar as they can satisfactorily replicate or explain such observations, or predict from them.

Pre-field work is the first step towards successful site-specific geological investigations. This involves going out to relevant institutions to find out if there is any relevant existing information regarding the study. Geological maps, aerial photos, and the LHWP reports have been examined as tabulated in 1.4.1. Ahmad et al., (1999, state that other pre-field work involves the gathering of the proper equipment needed for the investigation and this may be as simple as getting a notebook and arranging for transportation or as complicated as getting seismic and survey instruments and arranging for several days in the field.

The researcher has undertaken field traversing to check and verify the presence of geological features seen on geological maps and lineaments observed from aerial photographs. The soil cover on the slopes springs and vegetation has also been checked as well as the river bed-floor for water tightness. Villages along the tunnel route and on the shoreline of the dams have been visited to create public awareness on the environmental impacts of some geological features. Reservoir construction presents a unique means for inducing landslides (Ahmad et al., 1999). In their analysis, it is stated that the filling of the reservoir increases the pore-water pressure in the material on the reservoir slopes and possibly causes slope failure. They claim further that rapid drawdown of the reservoir due to pumped storage use or dam failure can cause landslides at the reservoir's edge as lateral support is removed faster than pore-water pressures can decrease in the soil or rock. On that basis the Katse Dam, shoreline has been observed with the view to collect information that has been used as a model for the Mohale Dam. Material excavated from the Mohale Tunnel and Dam has been geologically logged. Logging of cores, mapping and face mapping have been continually undertaken as the dam and tunnel construction advanced. Environmental monitoring has been also in place in conjunction with the environmental monitors.

1.5 Laboratory Techniques

Laboratories on site have a capacity to carry out laboratory tests such as determining durability tests, the potential concrete aggregate test, water sample tests and petrographic analysis, while geochemical analysis and strength and deformation tests, are performed at the laboratories of Wits University and CSIR. The number of cores are normally being divided equally, for example if, five cores are sampled, two are sent to Wits and two to CSIR and one is kept for reference purposes. Core logging was performed on site to characterise the geology. Logging of tunnel spoil, with a 10 m spacing was carried out to confirm the geology.

2 REGIONAL GEOLOGY

The geology of Lesotho is given in the literature of the data sources above. In addition, a short account is presented in this thesis.

2.1 General

The Mohale dam site and basin is situated in an area underlain by basalt of the Lesotho Formation (Drakensberg Formation). The geology of the area is described in all the information sources of this thesis and in other LHWP documents.

2.2 Geological History and Stratigraphy

The project area is situated within the Great Karoo Basin, defined as a shallow depression in the earth's crust covering a large area in Southern Africa. This depression is said to have developed in the Paleozoic period. It was subsequently filled with a sequence of predominantly terrestrial and lacustrine sediments up to 7 km thick and capped by volcanic outpourings of basalts. It took more than 100 million years for the Karoo Sequence to be deposited, that is, between the Late Carboniferous Era about 290 million years ago (Dwyka Formation with tillites) and the Jurassic Era about 160 million years ago (basalt of the Lesotho/Drakensberg Formation).

It has been recognized that the depositional environment in the Karoo basin changed from polar in Dwyka times to arid at the time of deposition of the Clarens Formation and the Lesotho basalts. The sediments are mainly detrital, ranging from fine-grained argillaceous sedimentary rocks to sandstones and gritstones, and have been deposited in extensive alluvial flood plains, river channels and ephemeral lakes. The uppermost formations (Molteno, Elliot and Clarens) indicate increasing aridity of the depositional environment, and much of the Clarens sandstone is interpreted as a wind-blown sediment deposited in the form of sand dunes in a desert environment.

Different authors are of the opinion that the outpourings of the Karoo basalts appear to have started in Upper Triassic times and cut short the deposition of Clarens sandstone. The dolerite dykes are believed to be feeders of the basalt flows. The numerous dykes criss-cross the Karoo basin, cut through the sedimentary rocks and die out at various levels within the basalt flows. Basalt has been extruded flow upon flow to a thickness of at least 1 450 m, and originally covered a much larger area in the central part of the Karoo basin than today as observed by Dingle et al., 1983. They refer to the flood basalts as the Lesotho Formation instead of the Drakensberg Formation used by the South African Committee on Stratigraphy (S.A.C.S.), 1980. The huge aerial extent of individual flows shows that the flood basalts were highly mobile.

The basalt plateau of the Lesotho Highlands as well as the surrounding mesas of the Clarens, Elliot and Molteno Formations in the project area, are relics that have resulted from 150 to 160 million years of erosional process. The erosion of the pile has resulted in the formation of peneplain, dissected by a wandering steep-sided river valley.

- The South African Committee on Stratigraphy (1981) recommends the use of the term "Drakensberg Formation" for the same rocks that the Lesotho Department of Mining and Geology name "Lesotho Formation" on the 1981 geological maps of Lesotho. The two names are therefore used synonymously in this study.

2.3 Regional Geological Structures

The regional structure of northern Lesotho and surrounding areas is an elevated plateau of nearly horizontal lava flows and sedimentary strata, which undulates slightly. Locally, dips of up to 10° have been recorded. The base of the strata is thought to be broadly basin shaped in the central part of Lesotho (Dusar, 1980 and Burley et al., 1982 cited in the LHWP Reports).

Imposed upon this overall structure are sets of faults, shears and joints. These features are often selectively followed by dolerite dykes, which are contemporaneous with the lava flows. The preferred orientations for these features are east-southeast and northeast. There is also a set of joints with an approximate east-west orientation.

Other major structural features that occur in the region are diatremes, which are roughly circular feeder vents to the lavas, and diamond-bearing kimberlite pipes. The latter features are vertical pipes that postdate the lavas and thus transect all the strata.

2.3.1 Lithology

The overall lithostratigraphic column for the project area is summarized in Table 2.3-1. This table indicates that the sedimentary and igneous rocks of the Karoo sequence underlie the project area. This sequence of rocks was formed during the period from about 290 to 160 million years ago. The Karoo rocks were deposited in a basin, which is deepest in the eastern Cape Province. The project area is located towards the northern margin of the main Karoo basin, where formations are considerably thinner than in the middle.

Table 2.3-1: Summary Lithostratigraphy of the Project Area

Geological name	Lithology	Maximum thickness
Recent alluvium and coluvium	Gravel, sands and silts	N/A
Lesotho formation	Dolerite dykes, sills, plugs Basaltic lavas	N/A 1 600 m
Clarens Formation	Aeolian sandstone	165 m
Elliot Formation	Siltstones and mudstones with subsidiary interbedded sandstones	120 m
Molteno Formation	Sandstones with subsidiary mudstones and siltstones	50 m
Beaufort Group	Mudstones with sandstones	1 000 m

Karoo Sedimentary Rocks

The basal geological formation of the project area is the Beaufort Group, of which only the Delivery Tunnel traverses the uppermost strata. The Beaufort Group formed about 230 million years ago (Dingle et al., 1983 cited in LHWP reports). No outcrop of Beaufort Group was found in the area of the tunnel, but was penetrated to a maximum of 90 m in boreholes drilled during the feasibility study. Mudstones with subsidiary sandstone horizons were intersected in the boreholes.

Overlying the Beaufort Group on a disconformable erosion surface is the Molteno Formation. Sandstone is the dominant rock type in this formation, although subsidiary mudstone and siltstone beds are also encountered. The formation has a maximum thickness of 50 m in the project area and thins to less than 20 m in places,

The Elliot Formation lies conformably above the Molteno Formation and consists of siltstones and mudstones interbedded with subsidiary, though extensive, sandstone horizons. Individual beds range in thickness from less than 1m to more than 40m, and aggregate to about 110 m thicknesses. The finer grained rocks of this formation, and the mudstones of the Beaufort Group, exhibit a tendency to disintegrate rapidly when exposed to the atmosphere.

Conformably overlying the Elliot Formation are the mainly Aeolian i.e. windblown sandstones and siltstones of the Clarens Formation. The sands and silts were deposited in desert conditions as dunes with an undulating topography.

Karoo Basalt

The stratigraphic boundary between Clarens Formation sandstones and the basalt in the project area is sharp, generally flat lying and structurally continuous. Variations occur due to an undulating surface of the sands at the time of the first basalt outpourings, a post-depositional consolidation of the Lesotho Highlands area, and minor post-depositional faulting.

The basalts consist of a sequence of horizontally bedded flows, which have an accumulated thickness of about 1 500 m (range is 1 400 m to 1 600 m). Volcanic clastics and tuffs with volcanic breccias occur quite often within the lowermost basalt flows. Another frequent phenomenon in the lowermost parts of the basalts is intercalation of lenticular sandstones and siltstones, which represent the final phases of deposition of the Clarens sandstone facies. Intercalated sedimentary horizons have not been observed in the basalts higher than some 200 m above the general boundary of the Clarens and Lesotho Formations, and are not considered likely to occur at the higher elevations.

The thickness of basalt flows normally varies between 0.5 m and 20 m with an average of about 5 m. Exceptional flows up to 100 m thick occur, especially in the higher part of the whole sequence. Flow boundaries do not represent major discontinuities and are generally tight. Palaeosols have also been observed between flows high above full supply level of the reservoir as discontinuous layers, which form lenses in places.

Five types of basalt have been distinguished in the field during geotechnical investigations. The field descriptions that follow have been amplified from petrographic studies:

(i) Dense, compact microcrystalline basalt, which have a doleritic character. These basalts only occur in the centre of some flows that are usually greater than 5 m thick. They contain very minor or no amygdales. This type contains rare zeolites and minor quantities of expandable clays derived from deuterically-altered glass. Such basalts usually have a high strength

(ii) Dense basalt with disseminated soft mineral spots. This type usually occurs in the centre of flows and contains a varying but considerable amount of expandable clay minerals derived from deuterically-altered glass that is concentrated in spots. These basalts are not durable in ethylene glycol. Amygdales and zeolites may occur in minor quantities and the latter sometimes cause breaking of the rock material

(iii) Slightly amygdaloidal basalt (less than 1% of cut surface area).

(iv) Moderately amygdaloidal basalt (1 – 10% of cut surface area).

(v) Highly amygdaloidal basalt (more than 10% of cut surface).

The latter three types occur either at the base and top of flows, or within thin flows. These three types contain significant amounts of expandable clay minerals derived from deuterically altered volcanic glass and thus tend to react with ethylene glycol. The durability of all the basalts depends on the proportion and accessibility of free water to the altered glass. Zeolites are always in variable amounts, and cause some of these basalts to be crazed.

The five types of basalt occur in the following general proportions:

-	Dense and compact doleritic basalt	6%
-	Dense basalt with disseminated soft mineral spots	20%
-	Slightly amygdaloidal basalt	38%
-	Moderately amygdaloidal basalt	24%
-	Highly amygdaloidal basalt	12%

A full analysis and description of the petrography and durability of the basalts is given in most of the LHWP geotechnical documents.

Intrusions

Subsequent to the deposition of the sedimentary rocks and contemporaneous with the outpouring of the lavas, the strata were injected with feeder channels of lava which have solidified to form dolerite plugs, dykes, sills and inclined bodies of various sizes and shapes.

The dolerites are generally fine or medium grained, dense, very strong rocks of great durability. Occasionally the dykes coincide with faults (based on reported observations by several authors, who state that faulting is only present in the lower flows of the basalts and underlying formations). The dolerite rock material in the faults is sheared and slickensided. Details of dolerite petrology and durability are given in Feasibility Study Supporting Report B. The project area has also been intruded since Karoo times by diamond-bearing kimberlite pipes. Weathering is usually noticeable along contacts and where it penetrates deeper into the basalts.

2.4 Post-Karoo Deposits

The project area has undergone several phases of erosion since the outpouring of the lavas about 160 million years ago and the consequent tilting of the strata. The eroded material has largely been transported away from the project area by rivers, while colluvial soils have developed on the hill slopes and terrace surfaces. Localized alluvial deposits occur in the major rivers and in some minor stream valleys.

Seismotectonic and Seismicity

i.) Seismotectonics

The seismotectonics model for the area is based on limited source mechanism studies. It is believed that predominantly east-west compressive stresses in the basement rocks below the Karoo sediments cause deeper earthquakes. These can cause strike-slip movement along Precambrian 'megashear' zones in the basement rocks. In view of the tectonic stability of the area, the compressive stress field is considered to be relatively low. There are only a few major fault systems evident in Lesotho.

The ones that do exist are of the same age or older than the volcanic strata and are believed to be inactive. There is no known volcanic activity in the area and the last known volcanic events were the emplacement of kimberlite pipes and dykes during the Cretaceous Period. In general, earthquake activity cannot be related to surface geology.

Most earthquakes are believed to occur below the Karoo sediments. This believe is attributed to the possibility that these earthquakes occur due to stress relief associated with isostatic rebound following the erosion of superficial formations. Hence, for these earthquakes, differential vertical movements may be the primary source mechanism.

ii.) Seismicity

Lesotho and the surrounding areas are said to have experienced a relatively low level of seismicity. While the distribution of earthquakes in general appears random, some patterns are evident. There is no direct relationship with known tectonic features, which are exposed at the surface, has yet been determined for any recorded earthquakes. A total of about 100-recorded earthquakes have occurred within a 150 km radius of the Mohale dam site. One of these was a Magnitude (M) 5.5 event; five events were M 4.5 to M 5.0 and the remainder ranged from M 2.0 to M 4.5. The largest earthquake recorded within 150 km of the dam site was the M 5.5 Zastron earthquake, which occurred about 143 km south-west of the project area in 1957. Large events of

M 6.0 and M 5.9 occurred further west at Koffiefontein in 1912 and 1976 respectively.

The mining activities are thought to have induced the events at Koffiefontein in 1912 and 1976. Nevertheless, those events are indicative of crustal stresses and earthquake capability in the region. Three recorded earthquake epicentres occurred within a 30 km radius of the Mohale dam site. These events occurred at the following times and sites:

- M 3.8 on February 10, 1958; 23 km NE of the site
- M 4.2 on February 5, 1971; 13 km S of the site
- M 3.6 on May 3, 1976; 24 km from the site

Earthquake focal depths vary throughout the region from 0 km to more than 60 km. The majority of focal depths are, however, less than 33 km in depth. These are termed shallow earthquakes whilst focal depths greater than 33 km are referred to as deep earthquakes. The distribution of historical earthquake epicentres in Lesotho indicates that the majority of earthquakes occur in the southwest half of the country, including the Mohale site area. Ongoing studies will attempt to delineate precise seismic zoning. To date, however, no clear pattern of earthquakes has been proven in the Mohale area.

2.5 Physiography and Morphology

Climatic records in or near the Mohale area are limited to rainfall records at Marakabei and St. John the Baptist Mission, both situated south of the area, and a two-year data record from Blue Mountain Pass which lies west of the area. With a relative close correlation existing between temperature and altitude in the highlands, extrapolation from temperature data at Thaba Tseka (2160 m.a.s.l) and Butha Buthe (1680 m.a.s.l) forms a basis for a reasonable estimate of mean monthly temperatures in the study area.

The climate of the Mohale area is generally sub-humid, with about 85% of the precipitation occurs in the summer season from October to March when 10 to 12 rain days per month may be projected. The winters are normally dry. Hail occurs frequently and snow is common and may fall in any month of the year. Sudden weather changes are common, with temperatures falling rapidly within a few hours.

Summers are warm and winters are cold, with wide diurnal variations (Lahmeyer, MacDonald Consortium, 1986).

2.6 Hydrology and Hydrogeology of Lesotho

Lesotho Formation (Lower-Middle Jurassic) consists of massive amygdaloidal tholeiitic basalt with maximum thickness of 1600 m in the eastern part of the country with moderate to low groundwater potential. Well productivity and spring discharge from 0.1 l/s to 3.0 l/s. Perched aquifers are localized both in the uppermost and weathered zones of the formation. Indicative maximum depth of groundwater occurrence is 100 m. Intrusive rock represented by dolerite dykes and sills (lower Jurassic), including kimberlite (Cretaceous), with moderate ground water potential. Well productivity and spring discharge ranges from 0.3 l/s to 4 l/s. Aquifers are localised within the intrusion and often along the contact zones with the surrounding country rock. This is observable along the mountain slopes especially after rain.

Water is the most valuable natural resource of Lesotho. The headwaters of the major drainage system in Southern Africa, namely the Senqu/Orange River system, are located in Lesotho (see Map of Southern Africa). The Senqu River provides about half of the total flow of the Orange River. The Senqu River has a mean annual discharge of approximately 107 m^3 /s at Seaka Bridge. The country (30 648 km^2) comprises two major morphological units, referred to as the Lowlands and the Highlands which have altitudes between 1 400 to 1 750 m and 1 750 to 3 500 m respectively. The Highlands cover more than 75% of the country and include the Senqu River and its main tributaries, mainly the Malibamats'o, Senqunyane and Makhaleng Rivers, flowing mainly on the basaltic rocks of the Lesotho Formation. The Mohokare River (Caledon River) and its principal tributaries, the Hololo, Hlotse, North and South Phuthiatsana Rivers, flowing mainly on the sedimentary rocks of the Karoo Super Group, incise the Lowlands.

Base flow, the groundwater contribution during dry seasons, also differs between the Highlands and the Lowlands. In 1985 (May-October), one of the driest years on record for the past 20 years, most of the Rivers and Streams in the Lowlands were dry, while those in the Highlands were still flowing, albeit at a very low discharge rate. Various factors explain this persistence of Highland's base flow:

- The runoff/rainfall coefficient varies essentially with rainfall and slope, with higher values in the highlands than in the Lowlands. It has been estimated that in the mountain catchment areas, runoff is approximately 22% of rainfall, whilst in the Mohokare catchment is 12%;

- The basins in the Highlands have a larger catchment area than those in the Lowlands.
- Occurrence of snow in the Highlands, particularly between the months of June and September, encourages groundwater infiltration;
- Basaltic rocks generally weathered to a depth of a few metres and covered by the bush vegetation overlie the Highlands, providing a temporary storage for groundwater. The sedimentary rocks of the Lowlands are less permeable than the volcanic rocks, encouraging proportionally more evapotranspiration and surface runoff at the expense of infiltration.

Table 2.6-1: Estimates for Lesotho Water Resources are as follows:

(Source: Dept. of Groundwater Studies)

Catchment	Area Km^2	Precipitation P (mm/y)	Total Runoff (mm/y)	%R	Flow (cub.mm/y)	Deficit (mm/y)	%P
Lesotho	30,648	775	158	20	$4.73*10^9$	621	80
Senqu	20,485	749	167	22	$3.42*10^9$	582	78
Makhaleng	2,911	844	199	24	$0.58*10^9$	645	76
Maphutseng	363	737	166	23	$0.06*10^9$	567	77
Mohokare	6,890	824	97	11	$0.67*10^9$	727	82

3 GEOLOGY OF BASALT

The information below is extracted from the works of Veevers et al. (Southern Africa: Karoo Basin and Cape Fold Belt, 1994) and other authors.

3.1 Lesotho Formation

This formation is synonymous with the Drakensberg Formation as referred to by (Dingle et al., (1983), Eales et al., (1984) and Marsh et al., (1997)) cited in several LHWP Reports, Visser (1984), Olivier (1976), Chevallier et al., (1997), and Duncan, (1997).

The tholeiitic basalts cover 80 % of Lesotho and represent the remnants of a large lava flood, the paleo-extent of which is unknown. It is speculated that the basalts covered an area similar to the one occupied by the dolerite dyke and sill outcrops of the Main Karoo Basin. On the other hand, according to Chevallier, (1997), geochemical (mantle plume), structural (the proximity of an inferred triple junction), stratigraphic (thicker sills beneath Lesotho) would argue towards a restricted extension of the basalt surface and eruptivity.

Veevers et al., (1994) and Stockley, (1946), describe the Lesotho Formation as a monotonous succession of amygdaloidal basalt lava flows. The thickness of individual flows differs slightly in the opinion of different authors. The range is 0.5 m to 70 m and the aggregate thickness to the exposed top is 1 400 m (range is 1 400 m to 1 600 m).

The basalt is tholeiitic. Rare silicic lavas in the andesite-dacite range are encountered in the lower part of the Drakensberg Group. Eales et al., (1984) cited in LHWP, write that in the northeast Cape therefore a feature of the overall development of the Karoo igneous events is that the earliest eruptions were characterized by diversity in style and composition of the erupted products. This later evolved into widespread and regular effusion of compositionally monotonous lavas, which built the bulk of volcanic pile.

The subvolcanic complex of dolerite dykes, sills, so called "bell jar intrusions" and irregular bodies that intruded the Karoo sediments represents the feeder channels for the overlying lavas. Within the limits of the dolerite line that skirts basement and the southern edge of the Karoo Basin, the greatest concentration of the dolerite (dolerite: sediment ratio of >0.5) lies in an elliptical zone that trends across northern Lesotho; the dolerite sheets, according to Winter and Venter, (1970) cited in LHWP, seem to terminate at a critical distance of a few thousand feet below the basalts. Dingle et al. (1983) cited in LHWP explain this observation and the stratigraphic distribution of the sills in the Beaufort and Molteno, dolerite plugs in the Molteno, tuff in the Elliot, agglomerate and lava in the Clarens, and pyroclastics and lava at the base of the Drakensberg Group and lava above, by the following sequence of events:

- Earliest upward movement of magma in the late Triassic to produce explosive activity
- During late Triassic-Early Jurassic Elliot and Clarens deposition, an increased scale of upward movement of the magma in sills 1 to 2 km beneath the surface, degassing of the magma, and contact with ground water produced explosive action and created diatremes
- Main eruption flood basalt by dyke injection in the Early Jurassic (193 ± 5 Ma), followed by a second major injection in the Middle Jurassic (178 ± 5 Ma), as dated by Fitch and Miller, 1984. Recent dating (Chevallier, 1997 citing Duncan, 1997) shows that the Drakensberg Lavas were extruded in a very short period of time 180 Ma.

The effect of zeolite mineral on the durability of Lesotho basalts:

Basalts intersected by tunnels of the LHWP have shown variable response with respect to durability and this has led to the decision to provide the full tunnel length with concrete lining. Rapid deterioration of the tunnel profile during construction could be ascribed to either overstressing or chemical deterioration, when in many cases a combination of these factors could be identified. Future construction works in Lesotho will be similarly affected and a better understanding of the deterioration may result in cost saving by selecting alternative alignments/sites and construction methods. It has been suggested that the most vulnerable mineral is the zeolite laumontite, and that the mineral is concentrated along specific horizons within the basalt succession.

3.2 Basalt Classification

Bieniawski, (1974), emphasises the classification of rock masses to be of cardinal importance in terms of the design of tunnel routes and cross-sections of a tunnel, estimation of construction duration and in terms of selecting the appropriate tunneling method. This opinion stands in the context of dam construction. Geological classification is an important data source and definition process since it is the fundamental means of understanding more or less all other sources (Drs. Sharp and Stacey special advisers to Lesotho Highlands Consultants (LHC, 1992)). They emphasise that it is also important to recognize that the extent and detail to which geological classification can be taken may be greater than what is observable in general during routine mapping.

The main characteristic of a lava flow is the presence of pipe amygdales at the base, non-amygdaloidal basalt in the central part, and amygdaloidal basalt towards the top of the flow. The amygdales are usually filled with secondary minerals (zeolites, calcite, clay minerals, rarely chlorite, quartz). Thinner flows are commonly amygdaloidal, while the basalt of thicker flows sometimes is doleritic at the centre with characteristic medium-crystalline texture and random joint orientation.

Table 3.2-1: Field classification of basalt

Description	Amygdales (% by area)	Clay (% by area)	Comment
Doleritic basalt	0	<2	Some clay may be visible in this rock type
Non amygdaloidal basalt with dark soft spots (DSS)	0-1	>2	
Moderately amygdaloidal basalt	1-10	Variable	
Highly amygdaloidal basalt	>10	Variable	

Doleritic Basalt (DB)

Doleritic basalt is similar to medium-crystalline doleritic intrusions and is generally only found in thicker flows. It contains very small amounts of smectite clays.

Non-Amygdaloidal Basalt (NAB)

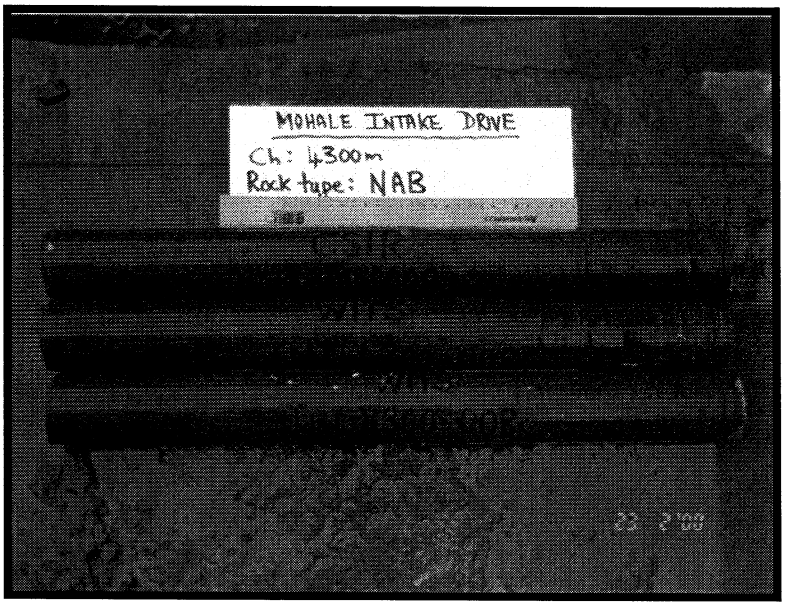


Figure 3.2-1: Cores of NAB. They are selected for WITS and CSIR for Laboratory. Tests (UCS and TS)

NAB is gray to dark gray; (fig. 3.2-1), often-speckled red or green, medium crystalline and usually occurs in the centre of the flows, which are more than two metres thick. It contains smectite clays (DSS), which occur in varying concentrations, (generally 2% to 25%) and range in size from 1 mm to 7 mm in diameter. Isolated amygdales do occur and occupy about 1% of the total cross section. The olivine rich basalt is a slightly coarser crystalline NAB and is found in thicker persistent flows.

Moderately Amygdaloidal Basalt (MAB)

MAB is usually grey to dark grey, fine to medium crystalline, and contains between 1 and 10% amygdales, (fig. 3.2-2). The upper contact, with the highly amygdaloidal basalt, is gradational. It contains less smectite clays than the NAB. MAB is commonly about 2 m thick.

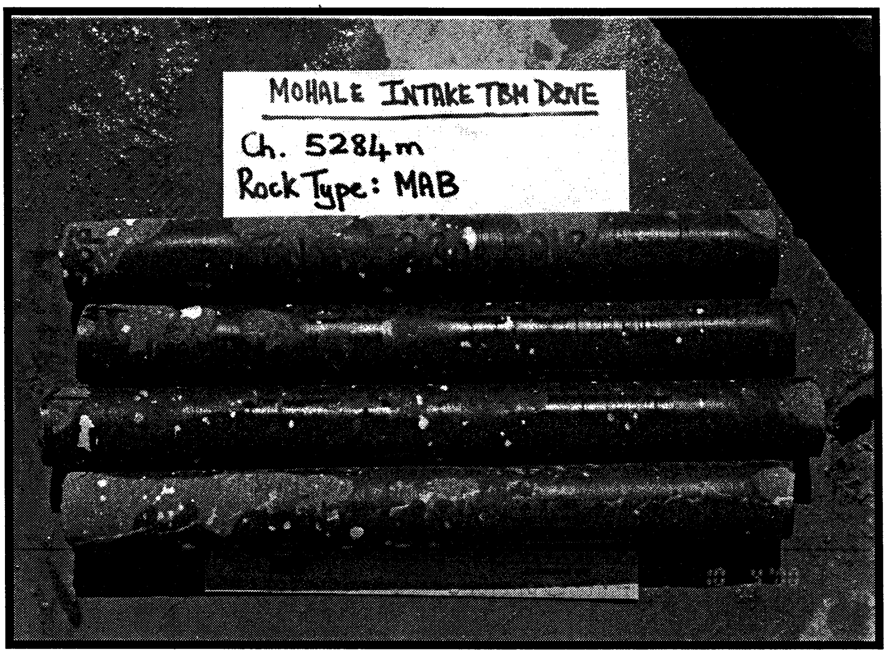


Figure 3.2-2: Cores of MAB. They are selected for WITS and CSIR for Laboratory Tests (UCS and TS)

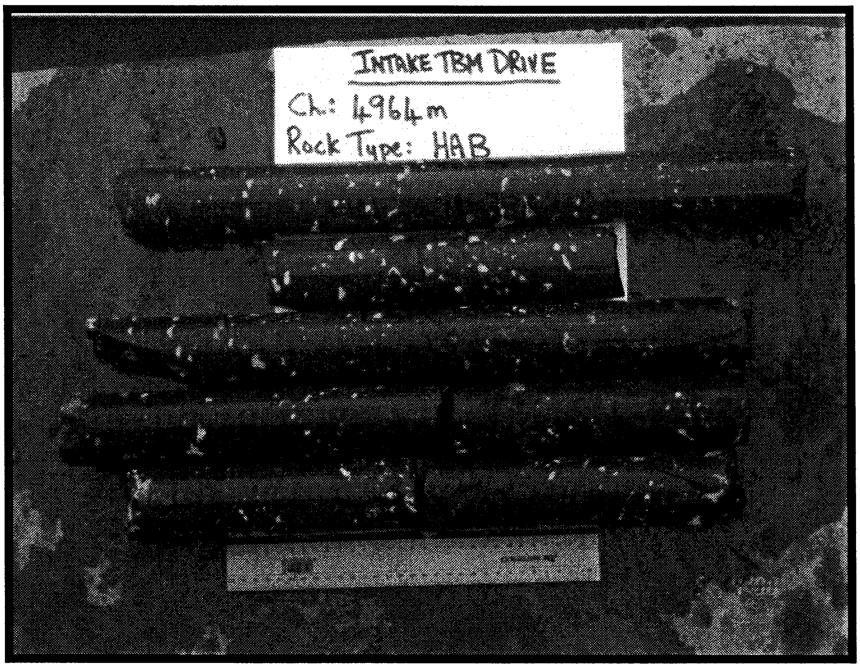


Figure 3.2-3: Cores of HAB. They are selected for WITS and CSIR for Laboratory Tests (UCS and TS)

Highly Amygdaloidal Basalt (HAB)

HAB varies in colour from red, reddish grey to greenish grey, is fine to very fine crystalline, figure 3.2-3. It occupies the upper zones of flows or areas near the base of flows in the pipe amygdale zone. HAB contains more than 10% (50% in the uppermost part of the zones just below the flow contact) amygdales.

3.3 Intrusive Rocks

Dolerite dykes as described in several LHWP-documents usually form intrusive rocks, which generally occur as near-vertical structures bisecting pre-existing strata and in most cases can be followed for several kilometres in plan. Several authors take dykes to be feeder channels for the basalt outpourings and consequently extend upwards through successive lava flows as deposition occurred. However, dykes die out at various levels of the lava pile. This implication is that a greater number of dykes will be encountered in the lower levels of the lava pile without outcropping. This has been seen in studies elsewhere which indicate a greater frequency of dykes outcropping at lower elevations. This kind of behaviour was often experienced during driving of the LHWP tunnels. This phenomenon was termed unforeseen ground conditions in tunnels. Dolerite dykes tend to be following the predominant lineament trends, which strike west northwest to east-southeast or northeast to southwest. Few dolerite dykes have been identified along the Mohale tunnel route. Seven dykes that cross the tunnel axis have been identified in the field, two of which are associated with joint zones. These dykes range in width from 1,0 m to 12,5 m. Dolerite sills are infrequently observed in outcrops in the Lesotho basalts. These linear features have also been identified across the Mohale Reservoir area (Fig. 5.4-1).

Regional maps and visual observations indicate a relatively high density of dykes to the east of the Malibamats'o River. This may be indicative of the presence of an impermeable barrier at depth (such as a sill) near the Mohale Tunnel, which has prevented dykes penetrating up into the overlying sequence (LHWP). It is thus expected that the tunnel may possibly intersect one or more dykes or sills. It should be noted that the drilling investigation programmed for the Mohale Tunnel has not been extensive enough to predict the presence of dolerite sills at tunnel elevation. Boreholes have been spaced at intervals of between approximately 2 km and 22,5 km. Rock cover above the tunnel ranges between approximately 400 m and 700 m over a substantial length of the Mohale Tunnel. Therefore drilling closely spaced boreholes to determine the presence of sills would have been impractical. The excavation of the Mohale tunnel however has produced valuable information relating to the above discussion.

Dolerite is very strong, ranges from fine to medium crystalline and is substantially harder than the basalt host rock (fig. 3.3-1). Dyke contacts are generally very fine crystalline and field observations have indicated a range of conditions varying from tight and welded to open with weathered rock. The encountered dykes in the Transfer Tunnel (deliver water from Katse to Muela) indicate a range of joint spacing varying from very close to medium spaced joints to those dykes with medium to very wide joint spacing. There are usually two jointing sets either observed, of a random attitude or comprising three sub-perpendicular joint sets. Instability may result where a regular joint system implies blocky conditions. These blocky conditions were one of the time delays courses during the excavation of the Mohale tunnel (Appendix B). Joints range from tight to those containing infill, which is common in dykes and may be emplaced as joint infill and/or as anastomosing veins. Dykes are often associated with faulting and slickensiding can be observed on some dykes, as it was the case with the dolerite dykes intersected by the Mohale and Transfer Tunnels. Figures 3.3-1, 3.3-2 and 3.3-3 below, show this particular type of dyke, which has been examined.

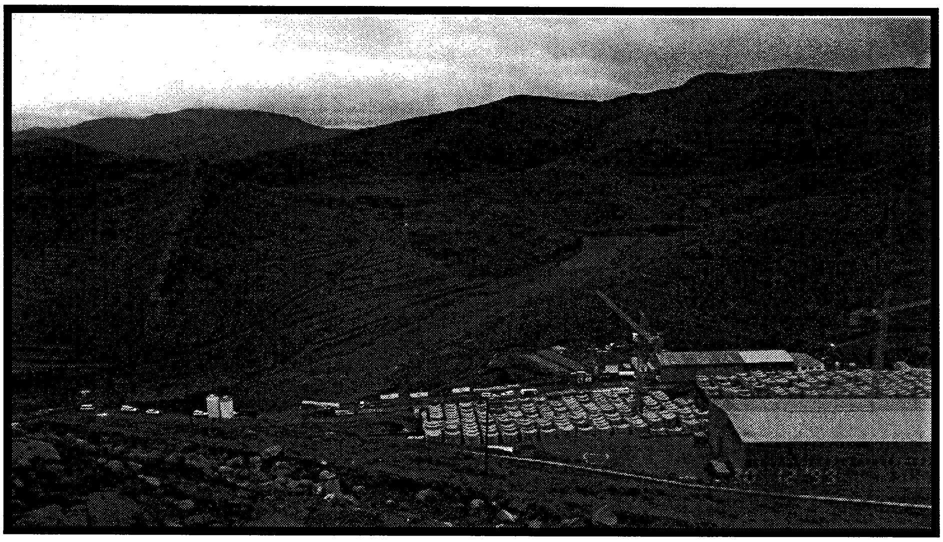


Figure 3.3-1: Dolerite Dyke. It is about 100 m from the Mohale Intake Structures. This Photograph was taken looking to the north over the Senqunyane River. In the Background on east Bank is the Precast Yard.

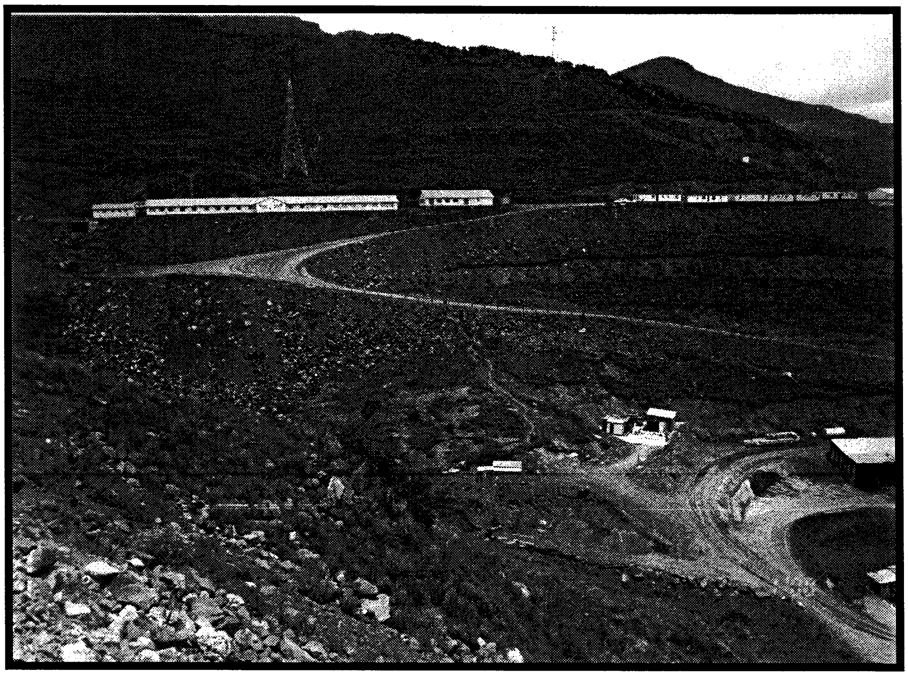


Figure 3.3-2: Dolerite Dyke. It is on the eastern Side of the River. Photograph was taken looking southeast from the Intake Structure. Engineers and contractors offices and access roads can be seen.

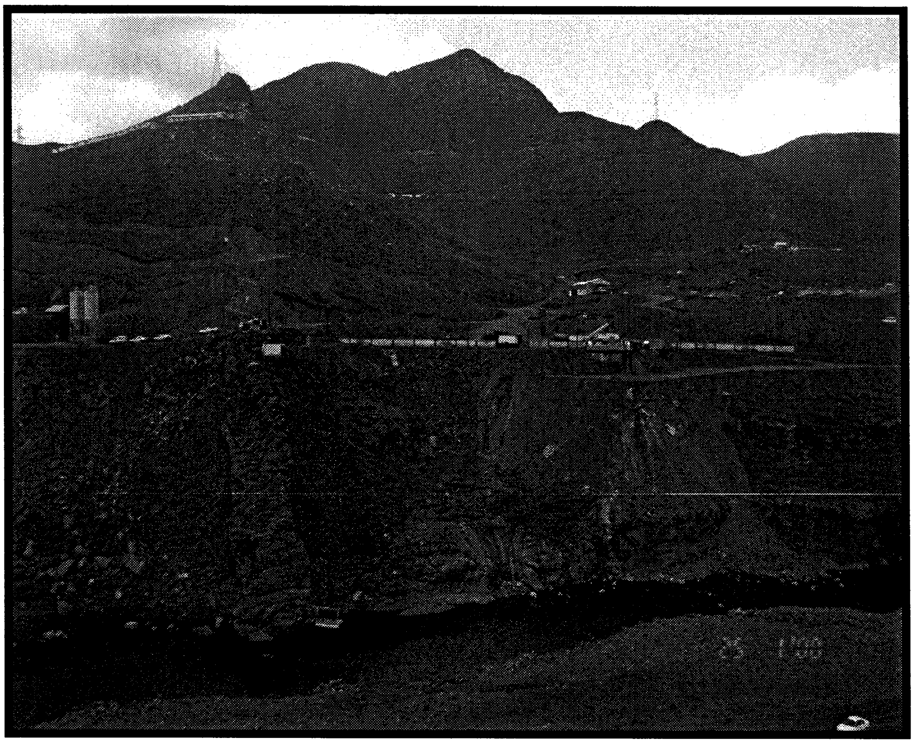


Figure 3.3-3: Dolerite Dyke. It is on the eastern Side on the River Bank. Photograph was taken looking east from the west bank.

4 SITE ASSESSMENTS AND SELECTION OF TYPE OF DAM

4.1 General Site Appraisal

The following points are considered very important for a general site appraisal

- A satisfactory site for a reservoir must fulfill certain functions and technical requirements.
- Functional suitability of a site governed by the balance between its natural physical characteristics and the purpose of the reservoir.
- Catchment hydrology, available head and strong volume etc. must be matched to operational parameters set by the nature and scale of the project served.
- The presence of a site (or sites) for a dam, the availability of materials suitable for dam construction and the integrity of the reservoir basin with respect to leakage (all these sum up to what is termed technical suitability dictation).
- The geological and hydrogeological or geotechnical characteristics of the catchment area and site are the primary determinants establishing the technical suitability of a reservoir site. To these an assessment of the anticipated environmental impacts of construction and operation of the dam or tunnel is added.

Investigation of the reservoir shoreline is conducted as required to confirm the stability of the potentially vulnerable areas in the vicinity of the intended dam. The construction material availability, especially suitable fills and aggregate sources are investigated and evaluated thoroughly. The hydrological studies are continual to confirm the initial investigation extent and results.

4.2 Type of Dams

Types of dams are summarized as follows:

- Rockfill Dam: rock foundation preferable; can accept variable quality and limited weathering. Cut-off to sound horizons is required. It is suited for all weather placing. Requires material for core, filters etc.

- Gravity Dam: suited to wide valleys, if excavation is less than ca. 5 m. check discontinuities (jointing/pattern of joints) in rock mass with regard to sliding. Moderate contact stress. Requires imported cement.
- Buttress Dam: similar to gravity dam, but higher contact stresses require sound rock. Concrete saving relative to gravity dam is from 30 – 60%.
- Arch and Cupola Dams: suited to narrow gorges, subject to uniform sound rock of high strength and limited deformability in foundation and most particularly in abutment loading. Concrete saving relative to gravity dam is 50 – 80%.

4.3 Design Features of Dams

Below follows a list of the design features of dams, which can have major implications with regard to programming and costs as well as to the environmental impact:

- Cut-off
- Spillway systems, including channels and stilling basins
- Internal drainage systems
- Internal culverts, galleries
- Foundation preparation, including excavation and grouting
- Construction details, e.g. transitions or filters in embankments or construction joints details in concrete dams.

4.4 Selection of Type of Dam

The optimum type of dam for a specific site is determined by estimates of cost and construction programmes for all design solutions, which are technically valid. Where site conditions are such that viable alternatives exist it is desirable that options are kept open, until a preferred solution is apparent. It may also be necessary to take into account the less tangible sociopolitical and environmental considerations in the determining of that solution (site selection).

Four important considerations are listed below:

- Hydraulic gradient: the nominal value of hydraulic gradient ? for seepage under, around or through a dam varies by at least one order of magnitude according to dam type.
- Foundation stress: nominal stresses transmitted to the foundation vary greatly with the dam type.
- Foundation deformation: certain types of dams are better able to accommodate significant foundation deformation and/or settlement with serious damage.
- Foundation excavation: economic considerations dictate that the excavation volume and foundation preparation should be minimized.

In general, foundation investigations are about foundation competence of the dam that is assessed in terms of stability, loading capacity, compressibility (soils) or deformability (rocks), and effective mass permeability. This means that the foundation should be competent enough to support the weight of the dam for safety, thereby reducing any chances of dam failure. The investigative techniques to be adopted depend upon the geomorphology and geology of the specific site. Nowadays, environmental issues play a pivotal role in the decision making process. There should be a full comprehensive study before the approval of any kind. The study encompasses the physical and social environment.

4.5 Dam Site and Geology

More than any other form of civil engineering, the construction, and adequate geological knowledge of dam sites conditions is an essential consideration (Duncan, 1969). Bell, (1980) and Wahlstrom, (1974) shares this point of view. They are of the opinion that if at some future date there is any failure of the dam, its foundations or its reservoir banks, the forces, which will be unleashed, are enormous and that geology is the first area to be investigated. Wahlstrom, (1974): "No matter how much preliminary investigations may have been addressed to the problem, it is never certain what geological features will be discovered when a dam site is excavated and even years later, unforeseen and unpredictable weaknesses may appear. Most uncertainties have been related to the geology of the site rather than to engineering design and workmanship". For instance, Wahlstrom cites Gruner, (1962) as listing the causes of dam failures as follows:

• Foundation failure	40%
• Inadequate spillway	23%
• Poor construction	12%
• Uneven settlement	10%
• High pore pressure	5%
• Acts of war	3%
• Embankment slip	2%
• Defective materials	2%
• Incorrect operation	2%
• Earthquakes	1%

The vast sums of money spent upon construction of dams make it necessary to ensure that they will function efficiently for long periods. Comprehensive studies are necessary, therefore, and these fall into three groups (Duncan, 1969):

- Studies at the dam site
- Studies of the reservoir area
- Studies of the catchment area.

The possible environmental impact can be elucidated timeously by impact studies. In this case, the environmental impact on the dam site and the catchment area create the opportunity for timeously remedial procedures.

4.6 Site selection

One of the major tasks of geological engineering is site investigation (Duncan, (1969); Thomas, (1976); Wahlstrom, (1974); Bell, (1980)). It is applied to gain information about the subsurface by employing both traditional methods such as classical geological mapping and drilling, and geophysical surveying methods. Other tasks, which must be undertaken, are the assessment of the stability of foundations, dams, slopes, underground structures (e.g. tunnel construction), the prediction of natural hazards (e.g. seismic activities), geohydrological analysis (e.g. watertightness of a reservoir), and environmental impacts.

Every reservoir has unique geological characteristics. Gaining a thorough understanding of these characteristics is very expensive and time-consuming according to (McCully, (1996); Duncan, (1969); Bell, (1980); Thomas, (1976)). They claim that millions of Rands may have to be spent on a geological survey before it determines whether a site is suitable for the construction of a dam. Hence, they agree that it is normal for the dams to be designed with a partial knowledge of local conditions – the constructor just has to hope that they are not going to meet any unstable formations which will fail to support their foundation or that the roof of their tunnels does not come crashing down. Some sites may be relatively uniform in their geology (one lithology with simple structure and a regular pattern of surface weathering). McCully, (1996), mention in their particular study, that in three quarters of 49 projects assessed by the World Bank study of hydropower in 1990, construction costs have been found to have experienced unexpected geological conditions. The study has concluded that for hydrodams " the absence of geological problems should be treated as the exception rather than the norm."

The selection of dam sites is based primarily on the topography, which has been determined by geology (fig. 4.6-1 and 4.6-2). Ideally, one seeks a narrow gorge, hoping for minimum quantities in the dam, and a valley opening out upstream from the dam to provide the required storage (Thomas, 1976). He states that the topographical and geological regional maps are the first data providers, while satellite imagery adds another dimension to photographic/visual interpretation, especially when the photographs are printed with colour differentiation (Thomas, 1976). The range of the geological features revealed in the vast coverage of these photographs can often improve on interpretations made from the examination of geological exposures or from normal aerial photography. Thomas, (1976), states that the assistance that this new facility can provide in identifying major fault systems and landslides on a regional scale is of special significance.

The river basin is divided into regions for development. There may be an alternative site along a length of the river within a region. According to Thomas, (1976), since going further downstream implies a greater catchment area and hence a greater flow. The value of this additional water and the greater head for the same full supply level must be balanced against the increased cost for a higher dam as well as a larger environmental impact area (Thomas, 1976).

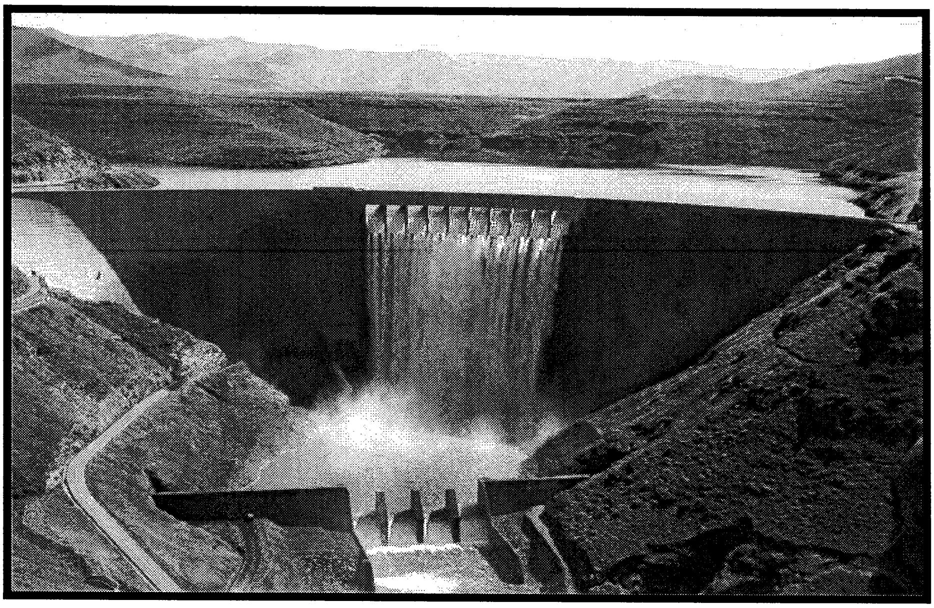


Figure 4.6-1: Katse Dam. The Photograph was taken facing North from the Operation Centre.

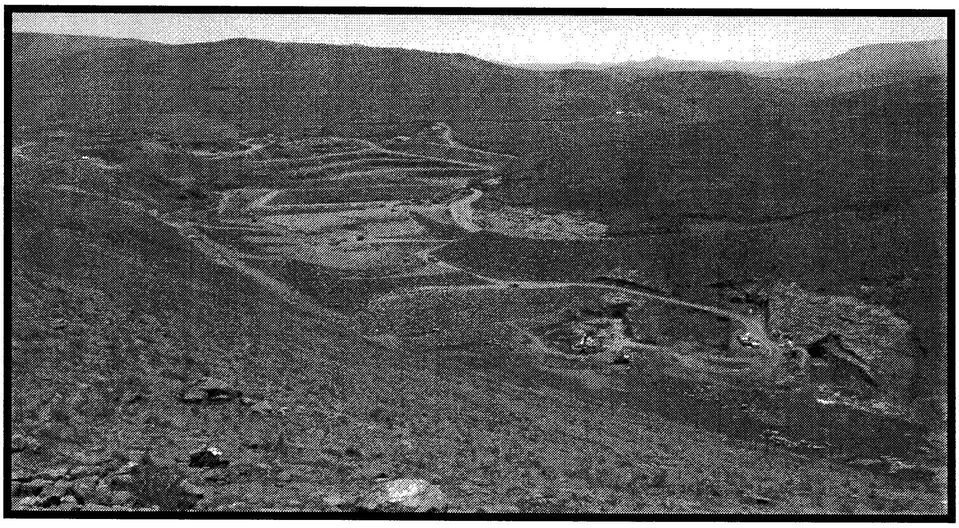


Figure 4.6-2: Mohale Gorge: Construction of the Dam at its early Stage. Diversion Tunnels. Dam Embankment. Further Upstream is Quarry for Rockfill Material. Photograph was taken looking Upstream from Engineers Office.

4.7 Site Investigation

Bell, (1980), Thomas, (1976) and Wahlstrom, (1974), state that site investigation involves exploration of the ground conditions on and below the surface. They take this as the prerequisite for the successful and economical design of engineering structures and earthworks. Insufficient or inadequate information with respect to the character of the ground can lead to the production of an unsatisfactory design, which may subsequently result in serious damage or failure of the structure concerned (Bell, 1980). He is of the opinion that there should be no attempt made on saving funds at this stage because this might lead to additional cost at later stages if unfavourable conditions, previously undiscovered, are found during the construction stage. Focus should also be, at this stage, on the environmental impact studies, no pushing aside of the study for later stage because the investigation is coupled with the environmental impact. It is thus clear that every little step of the project has a bearing on the environment. Economic considerations should be regarded as a secondary matter as far as safety is concerned (Bell, 1980). Safety should always be on top of the investigations agenda at all times.

Bell, (1980), considers the general objectives of site investigations as follows:

- Suitability assessment of a site for the proposed structure.
- Attempt to foresee and provide against difficulties that may arise during construction due to ground and/or other local conditions.
- Investigations should continue throughout the construction stage as well as environmental monitoring.
- It is essential that the prediction of ground conditions which constitute the basic design assumption, are checked as construction proceeds and designs should be modified accordingly if conditions are revealed which differ from those predicted.

Features that should be sought include old and potential landslides, geological faults and major joints striking parallel with the valley; they are considered the result of stress relief during erosion of the valley. The resultant joints may be open or infilled with the products of weathering and in the case of Mohale Dam zeolites, palaeosols and clay. The products of weathering easily absorb water and form a weak link between the rock mass. In the transfer tunnels, LHWP Tunnels, a phenomenon termed crazing was observed and it was linked to basalt exposure to moist air. Consequently, they can either be a lubricant or washed away resulting in mass movement. They may present construction hazards or if intercepted by other joint systems, may provide leakage paths around the dam (Thomas, 1976). Thomas, (1976) and Wahlstrom, (1974) agrees that the examination made along the beds of rivers and tributary streams will indicate strikes and dip of rock formations; these are features particularly relevant to the stability of both the dam and abutments. They emphasizes, that, any spring or underground water should receive close attention in case they provide paths for leakage from the reservoir. For this reason grouting was performed for the foundation of the Mohale Dam.

In summary, site investigations require careful planning and a considerable investment of time and resources. In situ and field test-techniques is performed to supplement laboratory testing and to make a comparison so that a comprehensive conclusion can be drawn (fig. 3.2-1, 3.2-2 and 3.2-3). The interpretation of geological and geotechnical data needs the closest cooperation between the engineering geologist, the geotechnical specialist and the dam engineer as well as the environmental specialist. The team members must stay always in technical touch with each other.

4.7.1 Reservoir Sites

Bell, (1980), considers the following aspects listed below as very important, when investigating a potential reservoir site:

- The climate (The amount of rainfall, run-off and infiltration and evaporation and transpiration)
- Topographical conditions
- Geological conditions of the area (e.g. Geohydrology, seismicity)
- Environmental conditions (e.g. Vegetation, Settlements)

4.7.2 Leakage from Reservoirs

When the reservoir is leaking, the following are the indicators:

- The sudden increases in stream flow downstream of the dam site with boils in the river,
- The appearance of springs on the valley sides.

Investigations will quickly focus on the geology of the site, i.e. rock formation, structural elements and construction materials (fig. 5.6-1, 5.6-2, 5.6-3 and 5.10-1). In the case of Mohale Dam, two quarries were identified for the production of aggregate and rockfill material. It may be attributed to major defects in the geological structure such as solution channels, fault zones or buried channels, for instance, a buried palaeo channel was discovered during site investigations of Mohale Dam, through which large and essentially localized flows take place. Leakage is considered a key parameter that relates to the overall performance of the CFRD and it has to be monitored on the daily basis. Large leakage rates are usually considered as an indication that damage has occurred to the perimeter joint and/or that the concrete face has cracked to a considerable degree. Seepage through the foundation may also be a contributing factor to large leakage rates. The fundamental design concept of the CFRD is that the several embankment zones of the dam including the face support material, filters, transitions, underdrainage and the body of the dam must remain stable even if extremely large leakage rates were to occur. The ability of rockfill to accept and pass large flows is well known in the literature. Thus, if the embankment zones and the foundation treatment have been designed and constructed appropriately, the large leakage rates are not an indication that safety is a problem, but rather that remedial treatment may be required to reduce the leakage, (Bell, 1980).

4.7.3 Time and Money Availability

The amount of money and time required to investigate a dam site depends primarily on the site and the type of dam. The CFRD was chosen for Phase 1B amongst other due to low construction price and short time required to complete. The Mohale Dam Contractors promised to complete construction ahead of schedule and at a discount. Indeed this was accomplished.

According to Thomas, (1976), investigations will normally encompass the following considerations:

- National or international policies; even the type of dam may be influenced by such policies rather than by minimum cost

- The purpose of the dam and how it fits into the existence or future plans for water conservation and utilization in the region
- Ultimate safety of the present or future inhabitants of the valley
- Finance (its availability and constraints)
- Environmental impacts (lacustrine, riverine, estuary, socio-economic)
- Quality of water, chemical and biological
- Physical factors such as hydrology, geology, topography
- Availability of resources, both materials and skills

4.7.4 Basic Concepts

Adequate information to be provided to the team (section 4.7) so that a dam that is stable against overturning and sliding, both on or within the foundations; the rock must be competent to withstand the superimposed loads without crushing or undue yielding and the reservoir basin must be watertight (Duncan, 1969; Thomas, 1976).

4.7.5 Basic Data

World Class team of expert has been engaged in all stages of investigations of the LHWP. The LHDA, the engineer and the contractor have their own panels of experts in search to deliver a world-class product, which is the completion of Phase 1. Experts should be employed to handle the following subjects according to Thomas, (1976):

I. Topography: It is the prime element used for planning and designing infrastructures. The determination of; the excavation volumes to be carried out at various levels; the existence of any low saddles around the perimeter; the quantities of material to be excavated in the dam, for the layout of access roads, and for the setting out of the dam; and onto the completed dam to verify that it behaves in accordance with design.

II. Meteorology and hydrology: A characteristic of a river that involves the average quantity of water available; the minimum flow, both as the absolute minimum and the minimum average over a period of a month or a year; the maximum flow that has been recorded and estimates of what might occur in the future. Variation in flow determines the storage necessary and the height of the dam to full supply level. Flood flows determine the spillway arrangement and freeboard required for flood routing through the reservoir; and wind velocity determines the behaviour of the reservoir body and the additional freeboard required to prevent overtopping of the dam by setup, seiche or wave effects. Each of the hydrological factors has some influence on the height of the dam and hence the cost of the project. It is therefore most desirable that meteorological and hydrological stations be installed near the site at the earliest possible date.

III. Geology and Seismicity: The geologist familiarizes himself with the regional geology and seismicity of the site. Any sign of landslide (ancient or recent) is very important for him to note. It is important for him and the team that geological investigations continue, not only through the design phase of the project, but also into the construction phase. Thomas, (1976) agrees to this opinion. Careful logging of all exposures is considered essential. Road cutting, trenches, shafts, adits or drill holes to observe and record the exposed rocks are always used whenever available. Colour photographs of cores are recommended for permanent record. This should also be employed to record all excavated surfaces that will later be covered by the dam. Thomas, (1976), emphasise the need for seismic investigations even though the region may be regarded as free from earthquakes

Since modern theories are actually relating microseismicity to possible seismic activity, it is therefore imperative to install seismographs near a proposed reservoir site some years prior to construction of the dam. Arrangements must be made for reporting all man-made events for instance blasting of any kind within 100 km; once correlated, such events are easily recognized on the charts (Thomas, 1976). According to Thomas, (1976), it is very important to record and report any seismic activity, a recurrence of which might necessitate modification of the dam, or might induce landslides following the filling of the reservoir. The reason for such records is to form a database of seismic events for the site, which can be used in future to analyse events. This database is aimed to provide valuable background information should seismicity occur after the filling of the reservoir.

Finance availability: The availability plays a crucial role in any project's formulation and ultimate take off. The method of financing a project and the money available may influence the selection of the type of dam. This in many instances brings about budget cuts, which may sometimes be detrimental to the project implementation and subsequently leads to incomplete studies.

Environmental Implications: Techniques are developed and economic evaluation is of dominant public concern. Intangibles, whether positive or negative are omitted from the studies since no one is prepared to put a monetary value on them. Around 1960, a realization dawned that there is actually more to life than monetary benefits to cost ratios (Thomas, 1976). During this period, the effects of pollution and environmental degradation are realised. In fact, the 1970s are remembered in history as the decade when man rebelled against pollution and desecration of the environment (Thomas, 1976).

4.7.6 Geological Classification of Rocks

In order to draw a three-dimensional picture of the geological subsurface, it is necessary to know the sequence in which the rocks were originally deposited (Mandel and Shiftan, 1981). They argue further that the geological age has a rock unit and time interval representing not only intrinsic scientific interest but also great practical importance. From a groundwater point of view, Mandel and Shiftan, (1981), regard the pore space in rocks as the most important property of rocks. The pore space may be contemporaneous with the rock (primary porosity), or it may be due to subsequent processes, such as fracturing, solution, and weathering (secondary porosity). On the other hand, primary porosity may be fully or partly obliterated by subsequent processes such as mechanical compaction, secondary deposition of minerals, and filling with clay particles. Geologically, there is no clear-cut correlation between the mode of rock generation, the subsequent geological history of the strata and the pore space, which the formation contains. Another descriptive classification distinguishes between granular porosity characterized by evenly distributed voids, irregular porosity characterized by large irregular distributed voids, and fissured porosity, where more or less continuous voids are aligned in a certain direction.

Joints are some of the most common rock structures. They differ from faults because their magnitude of displacement is very small. Duncan, (1969), defines joints as fractures along which no appreciable displacement has occurred. Although some joints have a random orientation, most occur in sets. He states that many rocks are broken by two or even three sets of intersecting joints that slice the rock into numerous regularly shaped blocks. These joint sets often exert a strong influence on other geological processes. For instance, chemical weathering tends to be concentrated along joints, and in many areas, groundwater movement and the resulting solution can influence the direction in which the stream courses flow. "Geological classification is an important data source and definition process since it is the fundamental means of understanding all other data sources." Drs. Sharp and Stacey cited in LHDA (1991), special advisers to LHC. They emphasise that it is also important to recognize that the extent and detail of geological classification may be greater than that which is observable in general during routine mapping.

5 PROJECT PROFILE

About 80% of the area is situated in the Maloti Mountains of Lesotho, 15% in the lowlands and the rest in the Republic of South Africa. The area is easily accessible by road. All Phase 1 areas can be reached using the excellent mountain roads from Maseru and Leribe. Katse Dam has a big enough airstrip that can easily accommodate medium-sized aeroplanes. The area has hospitals, primary schools, and commercial centers and lodges constructed under the LHWP infrastructure programme.

The area comprises incised mountainous topography with steep slopes, deep elongated valleys and generally shallow soils. The narrow, v-shaped valleys tend to widen and even out slightly in the southern section of the project area. The Senqu River dominates together with its tributaries; the Malibamats'o and Senqunyane Rivers provide the drainage of the area.

The mean annual precipitation is 1200 mm in the high altitude areas in the northwest and 600 mm in the southeast. Most of the rainfall occurs in the summer months as thunderstorms. Snow falls in the winter months and winters are cold and dry. There are three vegetation zones, which dominate the vegetation cover. The zones are mainly differentiated by altitude. Water quality is high. The upper Senqunyane River provides an important habitat for the maloti minnow.

The total population of the Mohale catchment is estimated at 7435 people. The communities earn their living through repatriation of money from urban Lesotho and as migrant labourers in the mines and other industries in the Republic of South Africa. Agriculture is practised in the valley bottoms and on ridges where the soil is somehow suitable. They also keep livestock, which they sometimes sell to gain some sort of income.

The majority of data and information of chapter 5 were extracted from the Final Report of MCG on the Geotechnical Investigations for Mohale Dam.

5.1 The Lesotho Highlands Water Project (LHWP)

Nthako et al., (1997), and Bell (1997) have discussed the project. The Lesotho Highlands Water Project (LHWP) has developed from what has been initially named the Oxbow Scheme. The purpose has been to transfer water from the upper tributaries of the Orange River in Lesotho to the tributaries of the Vaal River in the Republic of South Africa (RSA). The demand of water has grown exponentially from 1974 in the Vaal Triangle to such a degree that the Oxbow Scheme had to be abandoned and considerations for a much bigger scheme had to be made.

The Lesotho Highlands Water Project (LHWP) is a multi-purpose project designed to progressively divert the water from the Senqu River System to the Vaal River System north of Lesotho by a series of dams, tunnels, pumping stations and hydroelectric works. The development phases are Phase 1 (1A and 1B), Phase II, Phase III and Phase IV. These Phases comprise 5 major dams, 200 km of tunnels and a 72 MW hydroelectricity station, which provides Lesotho with sufficient electricity.

Table 5.1-1: Phases of the Lesotho Highlands Water Project

Phase	Reservoir System	Completion Date
1A	Katse Reservoir	1996
1B	Matsoku Weir	2001
1B	Mohale Reservoir	2003
2	Mashai Reservoir	2007
3	Tsoelike Reservoir	2017
4	Ntoahae Reservoir	2020

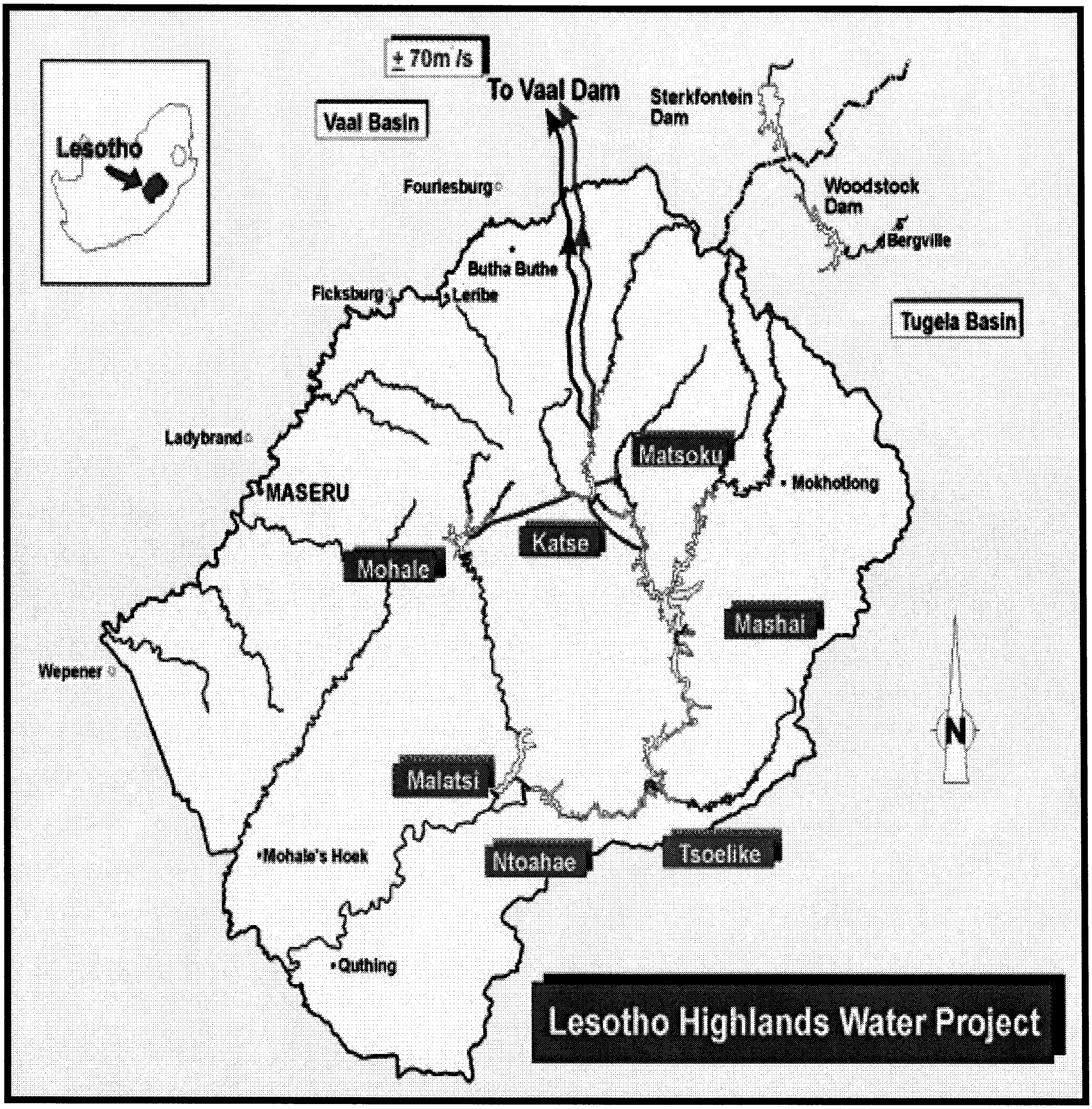


Figure 5.1-1: Map of the Lesotho Highlands Water Project (with no Scale)

The figure 5.1-1 is the map of the LHWP as is envisaged in the Treaty between the Kingdom of Lesotho and the Republic of South Africa.

The primary Objectives of the Project are:

- To change the direction of some of the southwesterly flowing waters of Lesotho to take a northerly direction to Gauteng in the Republic of South Africa.
- To generate hydroelectric power in Lesotho, in conjunction with water transfer.

- To provide a water supply, irrigation and regional development in Lesotho (to promote the general development of the remote and underdeveloped mountain regions of Lesotho and to ensure minimum environmental impact as well as to avoid negative socioeconomic impacts in the region).

Construction of Phase1 has commenced in 1990 and has been successfully completed in 2002. This phase consists of the construction of the following main structures:

- Access roads and bridges
- Power lines, telecommunications, housing offices and other commercial structures
- Katse Dam (a 185 m high double-curvature dam)
- The first transfer tunnel (a 45 km transfer tunnel from Katse Reservoir to Muela underground power station)
- Muela tailpond (a 55 m high concrete curved gravity tailpond)
- The first delivery tunnel (a 35 Km delivery tunnel from Muela Reservoir to the Ash River Valley in RSA)
- The terminal structures for transfer tunnel of Phases 1B and 2
- Mohale Dam (a 145 m high concrete faced rock-fill dam)
- Mohale Tunnel (a 32 Km long tunnel connecting Mohale and Katse Dams)
- Matsoku Weir and tunnel diversion (diverting Matsoku River water to Katse Reservoir)

Phase 2 and 3 are still under discussion between the Government of the Kingdom of Lesotho and the Government of the Republic of South Africa (RSA). The project has been envisaged for completion in 2020. The planned water transfer amounts to 35 m^3/s in Phase 1. This has been expected to double at the completion of Phase 3. If the second and third Phases are completed as initially envisaged in 2019 or shortly after, the transfer requirement will increase to 70 m^3/s.

The total availability of water in the Senqu River was estimated to be approximately 150 m^3/s, and the conclusion is that up to 70 m^3/s can be abstracted without any resulting shortfalls in the water downstream of the Lesotho border, as far as the

Atlantic Ocean. The abstraction is therefore categorised as surplus water from the catchment of the Senqu River in Lesotho by the LHWP Environmental Action Plan.

The entire scheme has been budgeted at a cost of $US8 billion at the time of the signing of the treaty. Most of the money has been to be raised within South Africa with a large part emanating from a levy on existing water users. The rest has been raised from the international development bank and the World Bank acting as the main broker and monitoring the environmental and social impacts.

The immediate impact of the Project has been the need to relocate villages and to provide alternative food sources for a significant number of households. The villagers are being relocated to different parts of Lesotho in accordance with their choices. The project has naturally created both the negative and positive impacts in terms of environment, socio-economics and health problems. Some of these impacts have been experienced during the construction phase while others shall be encountered at a later stage of operation.

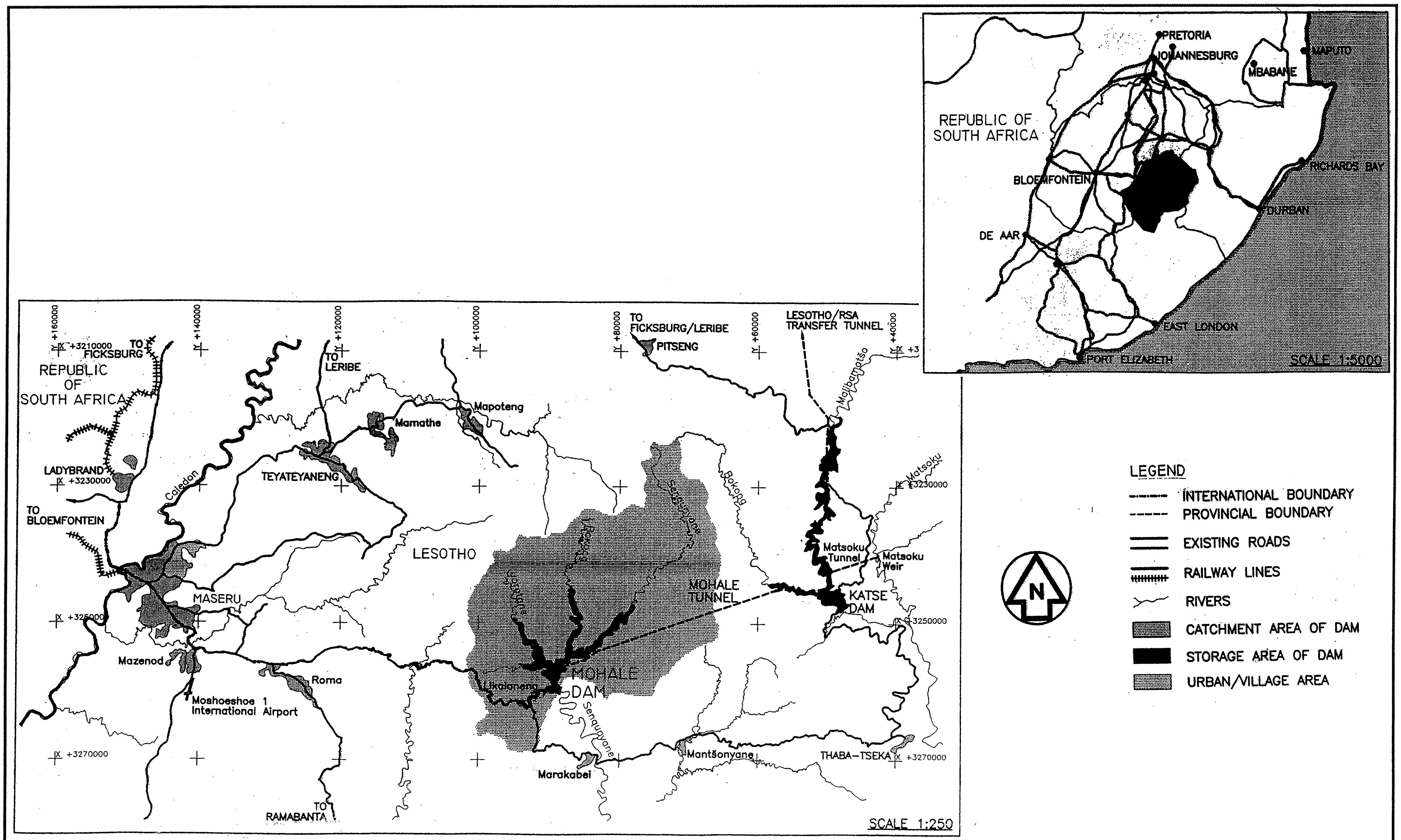


Figure 5.1-2: Phase 1B Catchment Area

5.1.1 Description of Phase 1A of LHWP

Phase IA, includes the construction of the Katse Dam and appurtenant structures, a water conveying system consisting of a 45 km long transfer tunnel and a 37 km long delivery tunnel and the Muela Hydroelectric development scheme, as well as a network of access roads and other infrastructure servicing the various construction sites. This phase is expected to satisfy the water delivery requirements to the Republic of South Africa (RSA) until the beginning of 2001; at which time the initial components of Phase IB (Matsoku Tunnel and Weir) should be operational. Different authors especially Bell and Haskins, (1997), have described this phase.

5.1.2 Description of Phase 1B of LHWP

This phase involves the construction of the Mohale Dam on the Senqunyane River and the Mohale Tunnel (to deliver 300 million m^3 of water into the Katse Reservoir) as well as the Matsoku Weir and Tunnel (to deliver 60 million m^3 of water to Katse). Phase 1B is the second and final stage in the construction of Phase 1 of the LHWP. Phase 1B, unlike Phase 1A, is situated in the Maloti Mountain Region. The basaltic lavas are layered to a near horizontal position and they form part of the Lesotho Formation.

The topography of the area has been described as an elevated and incised plateau (generally above 2 000 m.a.s.l) with high relief and steep-sided valleys and narrow rivers (fig. 5.2-1). The soils of the area are derived from the basalt, are generally thin at high elevations and on steep slopes and are deeper in the valley bottoms. The catchment of the Senqunyane River (fig.5.1-2) has been estimated at 929.9 square km. Jorotane, Bokong and Likalaneng are the main tributaries (fig. 5.3-2).

The Maloti Mountain Region of Lesotho is a place of high rainfall, temperate summers and long cold winters.

5.1.3 Mohale Dam and Infrastructure

The Mohale Dam Site has been identified in 1985-1986 during the feasibility study, and the type of dam has been identified in the planning study of 1995.

The Mohale Dam is a concrete-faced rockfill dam. The dam wall is located immediately on the confluence of the Senqunyane and Likalaneng Rivers. The embankment is 145 m high and contains 7.8 million cubic metres of rock material and concrete lined ungated spillway on the left abutment and an outlet facility. The construction of the dam involved a 27 m high Upstream Cofferdam and two diversion tunnels, tunnel 1 and tunnel 2, and a 7 m high downstream cofferdam. One of the tunnels is 6.7 m diameter and is 674 long, and is not lined. The other tunnel is 5.0 m in diameter, concrete lined, and 591 m long. The Cofferdam and the tunnels have been constructed to divert the Senqunyane River on the left bank, and comprise a stockpile area for tunnel spoil, two quarries (one to supply rockfill material and the other to supply material for concrete works), borrow areas, temporary access roads and bridges. The Mohale reservoir shall cover a surface area of 22 square kilometers and shall be able to transfer 857 million cubic metres out of the total storage of 946 cubic metres.

5.1.4 Mohale Tunnel and Infrastructure

The construction of the 32 km long and 4 m diameter Mohale Tunnel, involved the following:

- An area for tunnel spoil stockpile,
- Access roads and bridges,
- Borrow pits,
- Construction of a lower and upper intakes, construction of a gate shaft,
- Construction of an underground tippler on the outlet side, construction of tippler on the intake side,
- Precast factory for segments manufacturing,
- Construction of Workshops

The tunnel shall gravitate water into the Katse Reservoir on the Bokong River or into the Mohale when the Katse is full. Gates in the intake and outlet shafts will control the flow. At normal discharge, the velocity of water in the tunnel will be 0.7 m/s.

The area has a hospital, a primary school, and a commercial center and a lodge as well as good roads constructed under the LHWP infrastructure programme.

5.1.5 Matsoku Weir and Tunnel

The Matsoku Weir is a 10 m high and 180 m long concrete mass gravity weir constructed across the Matsoku River immediately upstream of the Tlopa Stream. It is connected to the Katse Reservoir by a 4 m diameter and 6 km long drill and blast tunnel. The tunnel is lined with concrete of approximately 400 mm. There is no storage capacity for the weir since it operates as a run off river diversion. Roads have been constructed to the outlet and to the intake.

5.1.6 Roads

Work on the first part of Phase1B involved the supply of access roads from Maseru to Mohale Dam and Tunnel Adit. It involved the construction of 59 km Mountain Road from St. Michael to Patiseng (reconstruction and rehabilitation of the Mountain Road between St. Michael and Patiseng Village). There is a newly constructed road, which leaves the Mountain Road at Patiseng, supplies access to the Mohale Dam via a 2 km spur road, and provides access to the Mohale Tunnel Adit at an approximate distance of 19 km.

5.2 Lesotho Highlands Geological Setting

The regional geology of the Lesotho Highlands as described in other geological literature and LHWP document is hereby given as follows: The regional geology of the Lesotho Highlands comprises horizontally layered tholeiitic basaltic lavas of the Lesotho Formation (Drakensberg Basalts). The tholeiitic basalt covers 80% of Lesotho and represents the remnants of a larger lava flood, the paleo-extent of which is unknown. This formation is the uppermost stratigraphic unit in the Karoo Sequence and has a total preserved thickness of approximately 1 450 m. The basalts were deposited in horizontal sequences of flow. These flow successions are typically 0.5 to 10m thick with occasional thicker flows of up to 40m.

The basalts are predominantly of olivine-poor tholeiitic composition. They are dark gray, reddish gray and occasionally maroon in colour and are fine to medium grained. Typically, each basalt flow is amygdaloidal toward the top, grades to non-amygdaloidal at the centre and displays characteristic tubular amygdales near the base. Amygdales are gas bubbles in the original lava flows, which were subsequently infilled by minerals such as clay, zeolite, quartz or calcite. At the centre of thicker flows, the basalt may grade to a rock with a doleritic appearance. Occasionally thin tuff interlayer occurs within the basalt flow sequence and the frequency of such interlayer is believed to increase with depth in the basalt succession. Narrow, intrusive dolerite dykes form prominent lineaments and there is evidence of dolerite sills either in outcrops or from fracture patterns in the basalts. The dolerite dykes are believed to be formed by the magmatic infilling of the original fractures which served to feed the surface basalt lava flows.

5.2.1 Mohale and Katse Catchment Area

The area is characterised by a dendritic drainage pattern focusing on the Senqunyane River. The area is generally located between 2500 and 3000 m.a.s.l and with a steep terrain. The geology of the area is similar to the geology given elsewhere in this work. The mountainous nature of the terrain determines that large areas are steep and rocky with no real agricultural potential except for some grazing. The parent material is so homogeneous that the soil properties differ by degree (i.e. texture, depth and drainage) rather than type. Soils are generally deep in the Senqunyane valley and their depth decreases with altitude and distance away from the valley bottoms, as it is the case in the Malibamatso valley. Winters are cold and dry with intermittent snowfalls. Maximum rainfall is in the summer months (December, January and February). Three dominant vegetation zones cover are also observed as described in the beginning of this chapter, Chapter 5. Water quality is high and the upper Senqunyane River provides an important habitat for the Maloti minnow.

Figure 5.2-1: Drainage Patterns and Topography.

5.2.2 Climate

Frosts are severe in winter. The temperatures for Thaba-Tseka as indicated in Table 5.2-1 disguise mean minimum temperatures for May, June and July that are often below freezing, with an absolute minimum temperature typically around minus 10°C.

When combined with high winds, the wind chill factor can bring the effective temperature to minus 20°C. On average, nighttime frosts at Thaba-Tseka occur over a 6.5-month period from May to October, at Katse Dam, temperatures of minus 15°C have been recorded (Mohale Infrastructure Consultants, 1995).

The average annual precipitation varies considerably, depending on the influence of the topography. The deeply incised valleys tend to funnel prevailing winds, significantly altering the prevailing direction. In the Mohale area, a prevailing northerly wind direction predominates with the greater frequencies from the northwestern sector. Prevailing winds in the Highlands generally strengthen through the day from 08h00 to mid afternoon, after which wind speed gradually decreases. Nighttime is mostly still, particularly the period from midnight to dawn. Wind gusts up to approximately 120 km/h have been recorded, with prolonged gusts of 35 km/h lasting up to six hours (Lahmeyer, MacDonald Consortium, 1986).A climatic feature of the area is that its climate is less harsh than that experienced in the Katse/ Thaba-Tseka region with slightly less severe winters, a shorter period of frost risk and a higher and slightly more reliable rainfall, fig. 5.2-2 and 5.2-3.

Table 5.2-1: Climatic Statistics of Thaba Tseka (2160 m.a.s.l)

	Months											
Mean	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Rainfall (mm)	72	94	70	38	26	8	10	14	22	57	65	84
Temperature(°C)	16.4	16.2	12.6	11.5	7.8	5	5.8	8.2	9.1	10.8	15.6	15.1

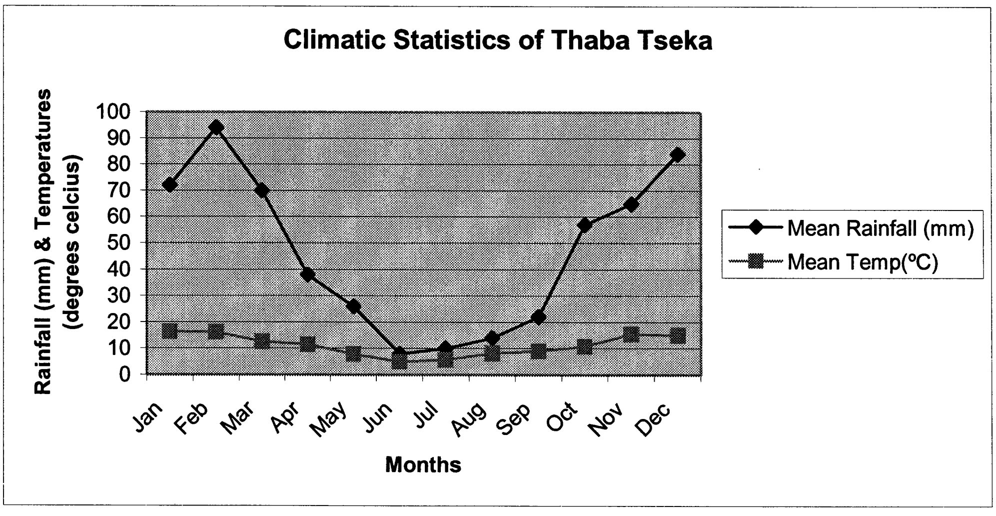


Figure 5.2-2: Climatic Statistics of Thaba Tseka

Table 5.2-2: Climatic Statistics of the Project Area

	Months										
Mean	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Nov	Dec
Rainfall (mm)	160	138	46	26	15	10	15	34	61	98	125
Temperature(°C)	17.6	17	14.2	13	9.5	6	6.5	7.5	10	16.8	17.4

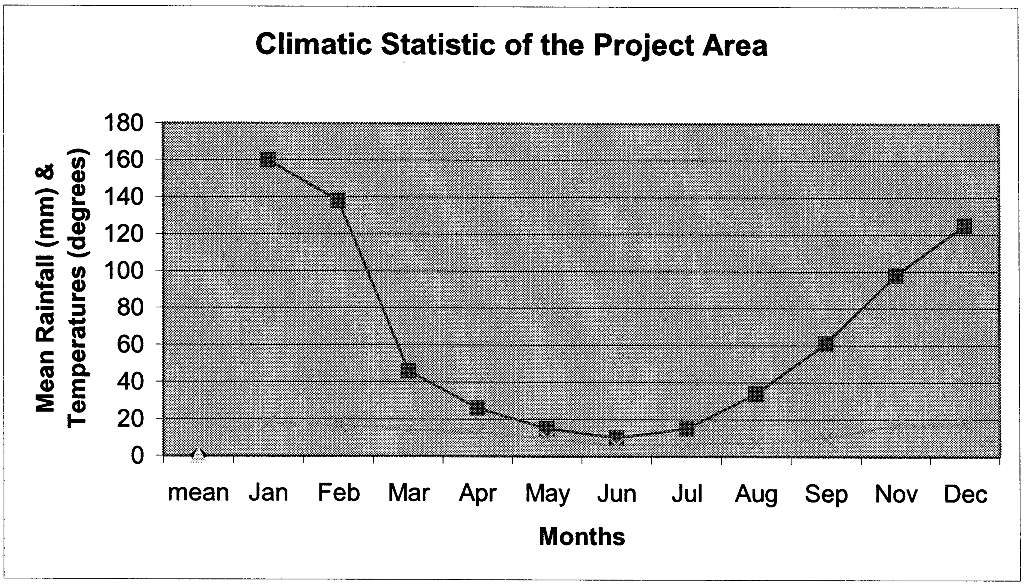


Figure 5.2-3: Climatic Statistics of the Project Area

5.2.3 Structural Geology

The Lesotho Highlands are an elevated area comprising resistant basalt rocks. The basalts are flat lying with dips of up to 1° regionally caused by a gentle synclinal structure. Steeper dips can be observed locally, which are probably related to the superimposition of laterally discontinuous flows. The base of the basalt flow sequence at the contact with the underlying sedimentary rocks is believed to be roughly basin shaped in the central part of Lesotho.

Linear discontinuities in the form of faults, fractures, shear and joints are imposed upon this broad basin shaped synclinal structure. The preferred orientation of these discontinuities is approximately E – W to ESE – WNW. Orientation of NNW – SSE and NE – SW are also common. Dolerite dykes along part of their length occasionally follow the major discontinuities. A good example is the relatively persistent Jorotane dyke lying 4.5 km north of the Mohale dam wall and passing some 200 m in front of the Mohale Tunnel intake structures (fig. 3.3-1; 3.3-2 and 3.3-3).

i.) Seismotectonics

It is believed that predominantly east-west compressive stresses in the basement rocks below the Karoo sediments cause deeper earthquakes. In general, earthquake activity cannot be related to surface geology.

Most earthquakes are believed to occur below the Karoo sediments. It is believed that these earthquakes possibly occur due to stress relief associated with isostatic rebound following the erosion of superficial formations. The primary source mechanism for these earthquakes may be the differential vertical movements.

ii.) Seismicity

There is no known volcanic activity in the area and the last known volcanic events are believed to be the emplacement of kimberlite pipes and dykes during the Cretaceous Period.

A total of about 100-recorded seismic events have occurred within a 150 km radius of the Mohale dam site. The largest event recorded within 150 km of the dam site was the M 5.5 Zastron earthquake, which occurred about 143 km south-west of the project area in 1957. Three recorded earthquake epicentres occurred within a 30 km radius of the Mohale dam site. These latter events are believed to have been mining induced. Nevertheless, they are indicative of crustal stresses and earthquake capability in the region. Earthquake focal depths vary throughout the region from zero to more than 60 km. The distribution of historical earthquake epicentres in Lesotho indicates that the majority of earthquakes occur in the southwest half of the country, including the Mohale site area.

5.2.4 Hydrology of Phase 1B

The hydrology of both the Mohale and Bokong areas is characterised by dendritic drainage patterns with high yields due to rapid runoff from steep slopes. The

Lahmeyer, MacDonald Consortium, study, (1986), showed that the catchment areas that had the highest mean annual precipitation also had the highest mean annual runoff.

Figure 5.2-4: Medium flowing Senqunyane River. The Precast Factory of the Mohale Tunnel is clearly shown. Photograph was taken facing north from the contractors offices.

Rainfall occurs predominantly in the form of thunderstorms and is of high intensity and short duration. The nature of the rainfall, the rapid movement of water off the steep slopes and thin soils, results in a quick drainage reaction time in relation to surface runoff (fig. 5.2-4). With the Matsoku River, highly variable, wet, transitional and dry seasons are identifiable from the hydrological record. The wet/rainy season extends from December to March, while the dry season usually extends from June to September. Runoff occurring during the dry or transitional months is often the result of snowmelt. One of the characteristic features of the river system in the area is the highly variable flow regime. The historical hydrological records show approximately three elevated flow events per month during the wet season. A gauging weir (G42) is located on the Matsoku River at Ha Seshote approximately 15 km upstream from the confluence of the Matsoku and Malibamatso rivers. The record is regarded as reliable up to flow depths of 0.9 m which approximates 50 m^3/s. Comprehensive flow records are available from this station and these are summarized in Table 5.2-3.

Table 5.2-3: Hydrology at G42 (MDP Yield Report, 1995)

MAP	MAR	Area	1:1 Year Flood	1:2 Year Flood	1:5 Year Flood
759mm	110.9x10	679 km^2	25hrs peaking at $92m^3/s$	38 hrs peaking at $137m^3/s$	38 hrs peaking at $206m^3/s$

The catchment area is characterised by steep slopes, thin soils and minimal agriculture. By contrast, areas of low mean annual precipitation occurred over areas with better soil cover and a higher proportion of agricultural production.

Sudden flash floods may occur. The Senqunyane and Bokong rivers run through deeply incised gorges, with a number of small falls interspersed with deep pools. There is evidence of heavy scouring discharge during rainy periods. Small sandbanks are evident on the low-water riverbank. During the dry season, the river is slow flowing and travels from a deep to a relatively still pool by way of shallow riffles, larger rapids and some small falls. Lateral drainage across the valley floor is unimpeded, and occurs mainly by surface flows or along natural channels between adjacent rock masses.

The existence of water tables is not always evident, although shallow temporary phreatic aquifers may develop during the rainy season. A number of springs are located in the villages, which supplies water for livestock and domestic use.

Hydrology of the Catchment Area

5.3.1 General

The Senqunyane River is located in the Maloti Mountains in central Lesotho. The Mohale Dam site is located in the uppermost catchment area of the Senqunyane River, about 65 km due east of Maseru. The Mohale Dam will regulate runoff from a single catchment with an area of 938 km^2.

The climate is generally sub-humid with about 80% of the precipitation assigned to the summer months, October to March. Thaba Tseka is located 40 km east of the dam site with maximum of 910 mm rain annually. Thaba Tsèka climatic data is shown on fig. 5.2-2.

Table5.3-1: April and September Marakabei Discharge Data

RIVER STAGE (M)	APRIL RATING FLOW (M^3/S)	SEPTEMBER RATING FLOW (M^3/S)	DIFFERENCE IN FLOW (%)
2	140	142	1.42%
2.5	245	245	0.00%
3	386	389	0.78%
3.5	564	583	3.37%
4	782	835	6.78%
4.5	1042	1153	10.65%
5	1346	1547	14.93%

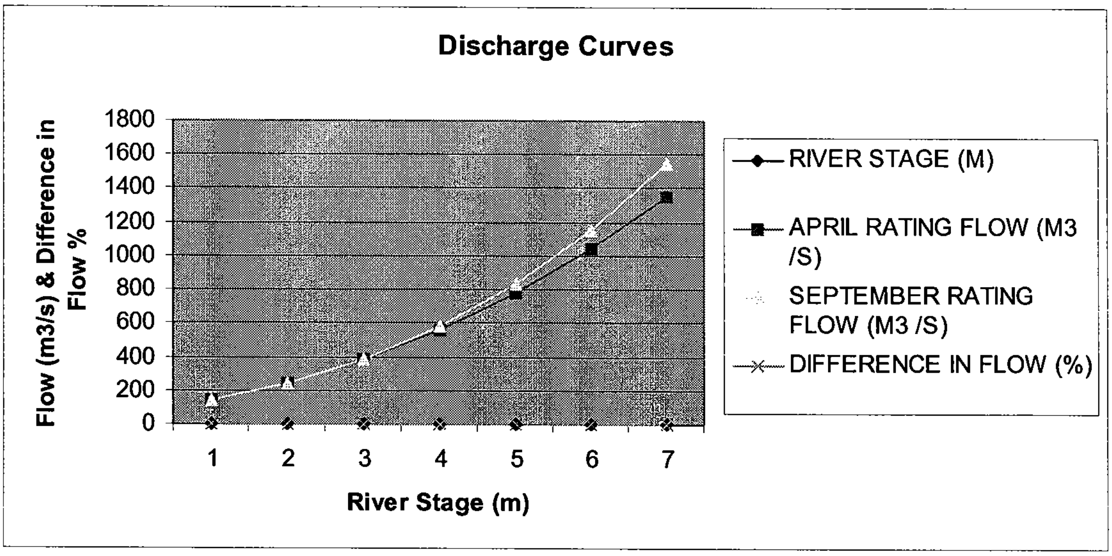


Figure 5.3-1: Marakabei Discharge Curve

Historic records used to determine the yield and floods were based on a revised stage - discharge curve for Marakabei (G17) as determined by the Institute of Hydrology, Wallingford, and advised by LHDA in late September 1996 (MCG, 1996). The April and September 1996 stage-discharge curves are shown in Figure 5.3-1.

5.3.2 Mohale Catchment Yield

5.3.2.1 General

The yield analysis of the Lesotho Highlands system is described in a separate document called "Mohale Development Yield Assessment Report using the design hydrology data 1930 - 1996". The Katse - Clarens Tunnel, connects the system to the Vaal River System. The following dams and tunnels form the system in Lesotho:

- The Katse Dam, with a Transfer Tunnel and Delivery Tunnel South tö the Vaal River System, including Muela Dam and Power Station.
- The Mohale Dam with a 32 km long delivery tunnel to Katse reservoir.
- The Matsoku Weir with a 10 km long tunnel to Katse reservoir.

5.3.2.2 Input Parameters

Table 5.3-2: Summary of Input Parameters used in Modelling of Phase 1 of LHWP

Item	Unit	Value
Annual Reliability of Supply using Sequential Analysis	%	98
Katse Reservoir:		
Full Supply Level	EL	2053
Storage at FSL	10	1950
Minimum Operating Level	EL	1989
Storage at MOL	$10^6 m^3$	431.4
Initial Storage	% of FSL	60%
Inflow Sequences	-	Up-dated Values (October 1996)
Mean Annual Inflow (1930-1996)	$10^6 m^3/yr$	571.8
Instream Flow Requirement	m^3/s	0.5
Mohale Reservoir:		
Full Supply Levels Considered	EL	2050.00 - 2085.00
Storage at the above FSLs	$10^6 m^3$	479 - 1085
Minimum Operating Level	EL	2005
Storage at MOL	$10^6 m^3$	89.81
Initial Storage	% of FSL	60%
Inflow Sequences	-	Up-dated Values (October 1996)
Mean Annual Inflow (1930-1996)	$10^6 m^3/yr$	317.5
Instream Flow Requirements	m^3/s	0.3
Matsoku Diversion:		
Full Supply Level	EL	2090.5
Mean Annual Diversion Flow	$10^6 m^3/yr$	84.4
Instream Flow Requirements	m^3/s	0.03 - 0.08
Mohale Tunnel:		
Length	km	32
Diameter	m	4
Flow equation computed based on storage levels at Katse	-	Equation given in Tunnel Design Report

5.3.2.3 Methodology and Results

The summary of yield analyses results for the Phase 1 projects using sequential simulation is given in Table 5.3-2. This evaluation has been given in the final report of MCG. In this evaluation, Katse's yield has been evaluated first, then Matsoku Diversion has been added to the system. Any increase in total yield resulting from adding Matsoku has been allocated to Matsoku and referred to in this work as the incremental yield. The same concept has been used in evaluating Mohale's incremental yield. The initial reservoir storage expressed as a percentage of active storage has been also given in Table 5.3-2.

Table 5.3-3: Summary of overall Yield Analyses at 98% reliability - Mohale FSL at EL 2075 (MCG, 1996)

Reservoir	Total System Yield (m^3/s)	Incremental Yield (m^3/s)	Reservoir Initial Storage (% of FSL)
Katse reservoir	16.8	16.8	60
Katse reservoir and Matsoku Diversion included	19.0	2.2	60
Katse and Mohale reservoirs and Matsoku Diversion	28.6	9.6	60

Note: Instream flow requirements (IFR) may be re-assessed in the future. A change will result in an equal reduction of the firm yield.

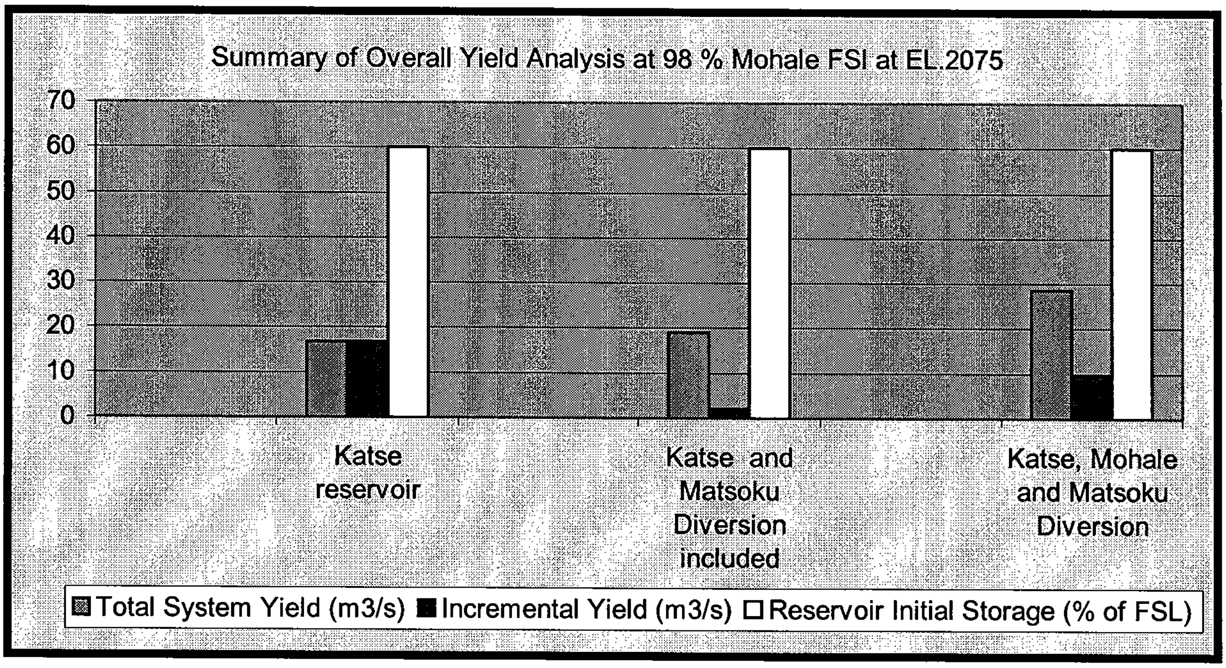


Figure 5.3-2: Summary of Overall Yield Analysis at 98% Mohale FSL at EL. 2075

5.3.3 Flood Hydrology

5.3.3.1 Probable Maximum Flood (PMF)

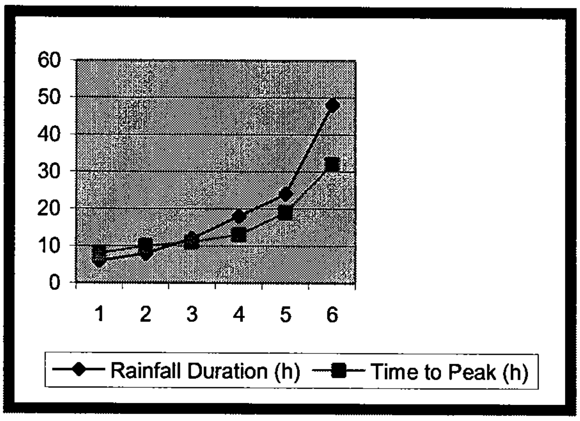
Herschfield empirical method has been employed in estimating the PMP for the Mohale Dam site (MCG). Storm profiles for hypothetical probable maximum storms have been examined, based on locally recorded maximum precipitation amounts and locally accepted practice. The various duration hydrographs have been derived using the procedure recommended in "Design Flood Determination in South Africa", HRU Report No. 1/72, University of Witwatersrand, 1972 (MCG). A summary of the PMF hydrographs determined by MCG is shown in Table 5.3-4.

Table 5.3-4: Summary of Recommended PMF Hydrographs (MCG, 1996)

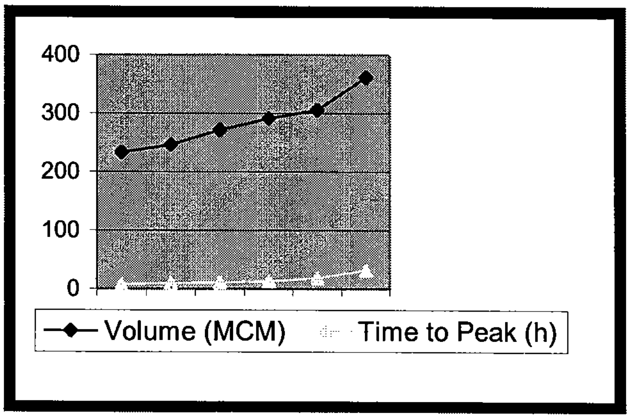
Rainfall Duration (h)	6	8	12	18	24	48
Qmax, (m^3/s)	10 519	9 998	8 581	7 333	5 873	3 672
Volume (MCM)	232.6	246.6	271.0	291.7	305.7	362.0
Time to Peak (h)	8	10	11	13	19	32

The hydrographs are shown in fig. 5.3-3 a, b, c and the numeric values in table 5.3-4.

a.)



b.)



c.)

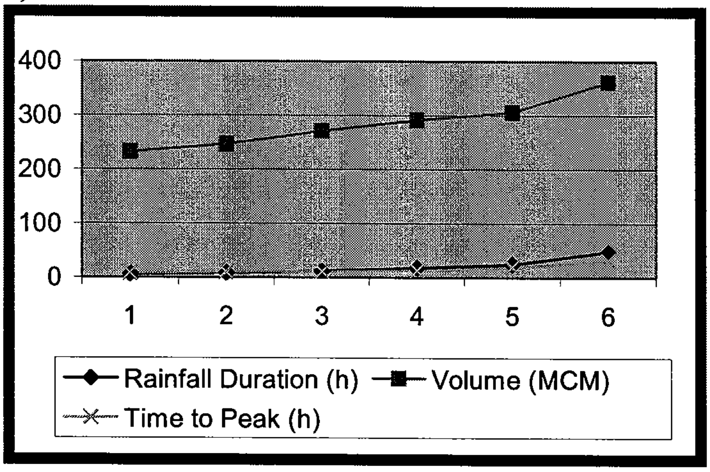


Figure 5.3-3 (a, b, c): Summary of Recommended PMF Hydrographs

5.3.3.2 Wet Period

Two site-specific methods have been applied. Flood peaks are shown in Table 5.3-5.

Table 5.3-5: Flood Peaks at the Mohale Dam Site (m^3/s) by MCG

Method	Recurrence Interval (year)					
	2	5	10	20	50	100
Statistical Analysis of Maximum Annual Flood Peaks	441	815	1 063	1 300	1 700	1 979
Design Hydrograph and Unitgraphs (24 hour Rainfall Duration)	410	660	850	1 000	1 210	1 390

Flood peaks using the Statistical Analysis of Maximum Annual Floods Peaks method (1st method in Table 5.3-5) have been derived from the half-hour compounded flow records (available years 1963-1995), fitting the data and extrapolating with the Type 1 External equation, while for the method in line 2 of table 5.3-5 several assumptions have to be made, including use of a numeric model developed for large catchments in the RSA, which may not be applicable for the Mohale catchment. Accordingly, principal weight is given to the values derived by the statistical analysis of flood volumes (Table 5.3-6) in sizing the diversion tunnels and the height of the Upstream

Cofferdam. Flood volumes for various recurrence intervals estimated from the historic records of measuring weir G17, and scaled for the Mohale catchment are shown with the design hyetograph method in Table 5.3-6.

Table 5.3-6: Mohale Flood Volumes (Million m^3) (by MCG)

Method	Recurrence Interval (year)					
	2	5	10	20	50	100
Statistical Analysis on Maximum						
• 1 day volumes	21.7	37.3	46.7	56.3	68.6	77.9
• 2 day volumes	32.2	53.9	67.8	76.2	92.9	111.5
• 3 day volumes	40.0	67.8	78.9	92.9	111.5	124.5
Design Hyetograph and Unitgraphs (24-hour) with maximum monthly base flow of 76 m^3/s	41.3	54.4	64.7	77.2	83.5	92.9

The final selected hydrographs prepared by using peaks and volumes derived from the statistical method and shaped in accordance with the 1976 historic hydrograph are shown in figures 5.3-2, 5.3-3 and 5.3-4.

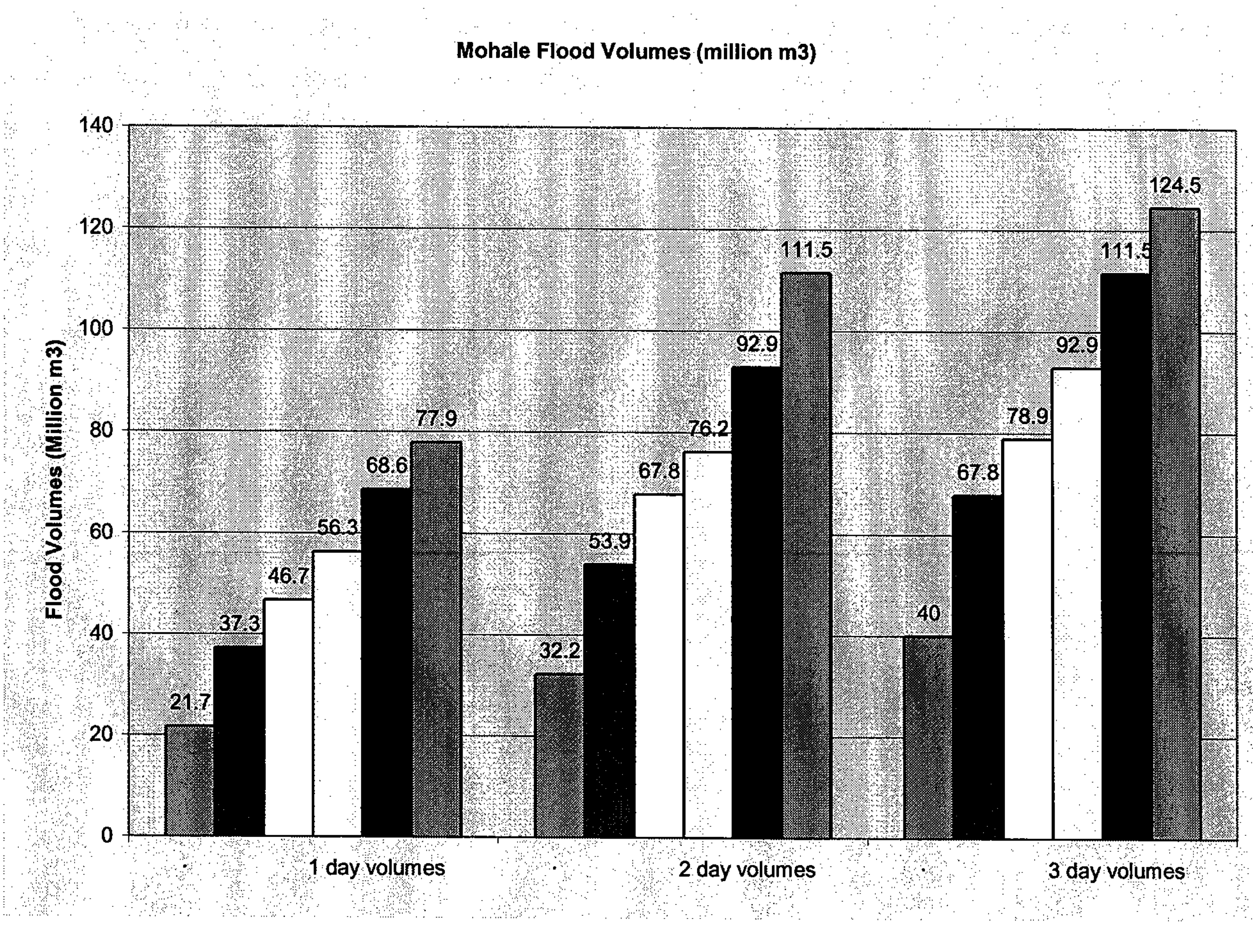


Figure 5.3-4: Mohale Flood Volumes

5.3.3.3 May to September Floods

The 1:10 and 1:20 year flood hydrographs for the months of May to September derived by statistical methods, used to assess water levels upstream of the Pre-Cofferdam and the partially constructed Upstream Cofferdam during the dry period of 1999 are shown in (fig.5.3-5).

A frequency analysis has been performed for assessing the 10-yr and 20-yr floods for the dry months (MCG, 1996). Maximum flood volumes of 1-, 2- and 3-days from May to September have been selected from 0.5 hr water level records measured at G17 (Marakabei) from the 1963 to 1995 records (MCG, 1996). The distributions used in the flood peak analysis have been adopted. Among five distributions tested, Type 1 External distribution fitted the results best. The results are shown in Table 5.3-7.

Table 5.3-7: Results of Flood Frequency Analysis for 1-Day, 2-Day and 3-Day Flood Volume for the Period May to September (MCM), by MCG.

Month	1-Day		2-Day		3-Day	
	10-yr	20-yr	10-yr	20-yr	10-yr	20-yr
May	11.71	15.27	16.06	20.80	18.61	24.00
June	6.47	8.56	9.71	12.78	11.79	15.52
July	3.37	4.48	5.45	7.21	7.05	9.31
August	7.34	9.74	12.33	16.29	15.60	20.59
September	21.16	28.05	34.19	45.31	40.11	53.09

(1). For the month of September, the statistical values fall on the high side due to random data.

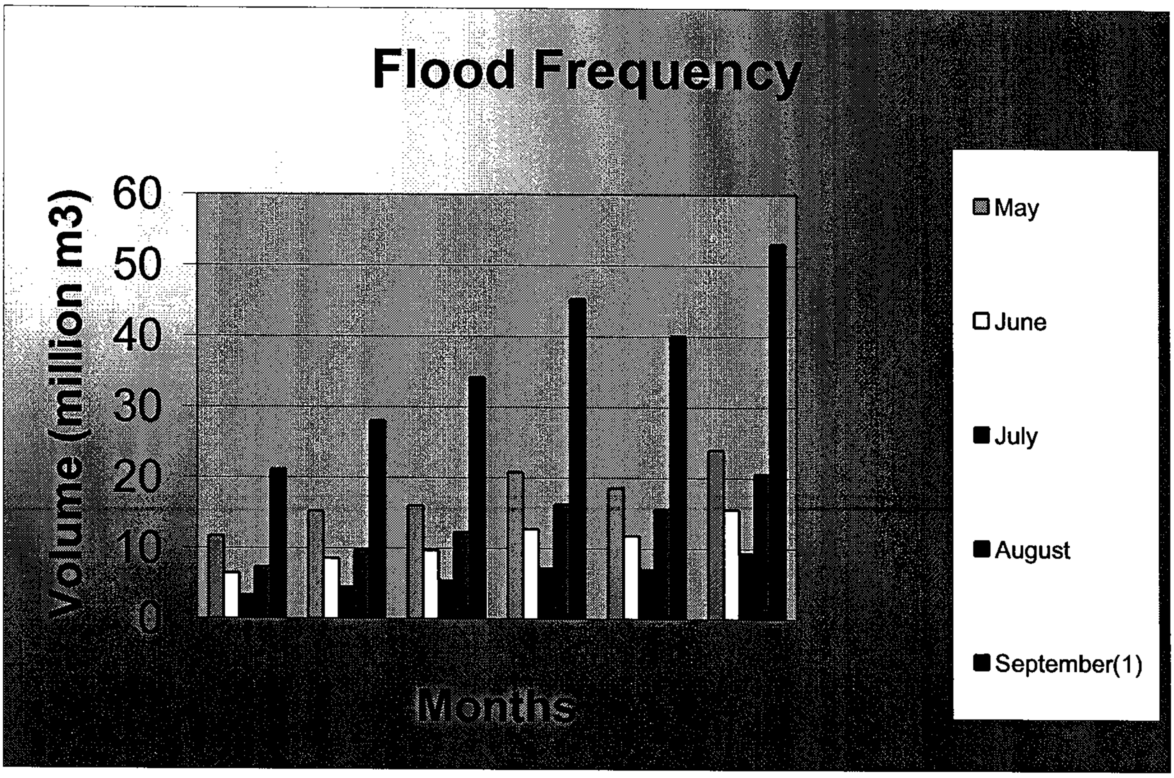


Figure 5.3-5: Results of Flood Frequency Analysis. They were performed for 1-Day, 2-Day and 3-Day Flood Volume for the Period May to September (MCM)

5.3.3.4 October to December Flood Analysis

Tunnel 2 has been not closed on 1 October 2001, as has been initially envisaged. This has been due to the late award of the Contract and other work-related delays, which occurred during the construction of the project. To ensure that Tunnel 2 could be closed under potentially higher flow conditions and high season flow could be impounded, the design criteria for Tunnel 2 stoplogs regarding the design flood for closure during higher base flow conditions related to October, November or December have been modified. To assess the situation, daily flow records for the months of October, November and December have been analysed using historic flows derived from Marakabei G17 weir for the period on record from 1965 to 1995. An assessment of the duration of flow exceeding 25 m^3/s and 50 m^3/s is shown in Table 5.3-8 and figure 5.3-6, demonstrating that flow exceeding 25 m^3/s for the month of October may last for 2 weeks, while flows exceeding 50 m^3/s last only a few days. Taking account of the cost of waiting time, but more so the value of water lost if closure could not take place, the stoplogs have been designed for a flow of 50 m^3/s and a differential head of up to 3 m.

Table 5.3-8: One-Day Flows Exceeding 25 m^3/s and 50 m^3/s (MCG, 1996)

YEAR	CONSECUTIVE DAYS WITH FLOWS					
	>25m^3/s			>50 m^3/s		
	October	November	December	October	November	December
1965	0	4	0	0	1.2	0
1967	0	36	0	0	12	0
1968	0	0	6	0	0	2
1972	0	0	0	0	0	0
1973	0	0	3	0	0	0
1974	3	?	?	0	?	?
1975	10	9	4	0	4	3
1976	0	10	0	7	9	0
1977	0	6	0	0	2	0
1978	7	0	7	0	0	6
1979	0	5	5	5	2	3
1980	0	2	6	0	0	3
1981	3	4	4	0	3	1.5
1982	0	11	0	2	8	0
1983	0	3	1	0	2	0
1984	0	7	3	0	4	1
1985	4	3	12	0	1	5
1986	4	9	0	3	4	0
1987	7	6	3	2	4	2
1988	0	3	5	5	1	4
1989	0	3	0	0	2	0
1990	7	0	1	0	0	0
1991	2	0	0	4	0	0
1992	7	5	0	1	3	0
1993	0	3	4	4	1	1
1994	3	4	3	0	3	1
1995	0	4	?	2	3	?

5.3.4 Reservoir Sedimentation

The discussions in this paragraph have been adopted from the MCG-final report. Sediment yield and distribution in the reservoir are discussed in the Sediment Report of July 1996. The study produced the following findings discussed in the next subparagraphs

5.3.4.1 Sediment Yield

Yield has been based on the suspended sediment data of station G17 at Marakabei, sampled by LHDA (1988-1993) and DWA-Lesotho (1979-1995) with consideration of G32 (Nkaus: 1971/72) data located further downstream (partially in the sandstone region).

To obtain the project mean sediment yield the following has been applied:

- The sediment rating to the continuous flow record for the period 1965 to 1994, giving a yield of 310 $t/km^2/a$.
- The inclusion of bedload of approximately 20% and for possible future land deterioration of about 5% increase, the figure to 400 $t/km^2/a$.

This yield value combined with economic design life of 100 years has been used in setting the low-level intake structure.

5.3.4.2 Reservoir Sedimentation Distribution

A one-dimensional laterally and depth-averaged mathematical modelling has been carried out with the river model "MIKE II" developed by the Danish Hydraulic Institute and modified as well as calibrated to reservoir sedimentation conditions in a study for the South African Water Research Commission.

The following has been taken into consideration while assessing turbulent suspended sediment transport:

- With a full reservoir, the widening in the Senqunyane River just downstream from the confluence with the Bokong River will cause reduced sediment transport and depositing can be expected.

- Due to the steep slopes in the Mohale Reservoir, sliding of sediment can be expected and for modelling, both horizontal and as a function of flow depth deposition (lateral) have been considered.
- Dominant inflows of 1:2 year and 1:5 year recurrence intervals have been modelled.

The scenarios considered are shown in Table 5.3-9.

Table 5.3-9: Mohale Reservoir Sedimentation Analysis (MCG): Scenarios Considered

Scenario *	Water Level			Discharge		Sediment Distribution	
	Mean = 2059m	% Time up	% Time down	Q_2=430m^3/s	Q_5=710m^3/s	f(H)	f(D)
5H - 1	Y				Y	Y	
5D		Y			Y	Y	
5E			Y		Y	Y	
5J			Y		Y		Y
6A-2	Y			Y		Y	
7*	Y				Y	Y	

Notes: f (H) = Sediment lateral distribution as horizontal build up

f (D) = Sediment lateral distribution as function of flow depth

% time up = Starting at 2005 m to 2075 m, as simulated in water yield analysis

% time down = Starting at 2075 m to 2005 m, as simulated in water yield analysis

Y = Analysed

* = Scenario 7A considered sedimentation modelling of Senqunyane, Bokong, Jorotane and Likalaneng River basins, while all other scenarios considered the Senqunyane River basin.

- The impact of density currents on sedimentation at the dam has been considered significant. Accordingly, the minimum elevation of the entrance of the Low Level Intake Structure has been conservatively set at EL 1977.
- It has been not expected that significant long-term sedimentation shall build up in the upper reaches of the reservoir due to the relative high velocities in the headwater river section at the lower lake level.
- The sediment volume, which has been expected to deposit in the reservoir over a 100-year design period assuming 100% trap efficiency, has been estimated at 30.6 million m^3. The expected total sediment volume is small in relation to the reservoir capacity and may have negligible impact on the reservoir yield.

5.4 Geomorphology of Mohale

The Mohale Dam site is situated on the Senqunyane River immediately downstream from the confluence with the Likalaneng River and 2 km downstream from the Jorotane River confluence. It is located at the upstream end of a broad east – west trending meander. This meander is part of a river drainage pattern, which, upstream from the dam site flows in a south -westerly direction and downstream from the dam site follows a southerly direction. The incised meandering course continues for about 48 km further downstream.

Immediately upstream from the confluence with the Likalaneng River, the Senqunyane River flows due southwards turning to the east at the confluence and the dam is constructed with an approximate north south alignment. The river valley at the dam site is asymmetrical with steep-sided slopes of about 25° to 40° from the horizontal at most locations. As is often the case in Lesotho, the cooler southward facing slope at the dam site (left abutment of the dam) is less steep and has thicker soil cover than the northward facing slope (right abutment). The apparent uniformity of the valley slopes is a result of the consistency of the basalt rock pile through Lesotho. The horizontally bedded successions of basalt flows are generally exposed along the slopes. A step - like topography has resulted from the differential erosion of basalt flow layers. The many outcrops and cliff faces occurring on the slopes are a reflection of the variability in resistance to erosion of the rock strata and a consequence of few structural features in the basalts.

The topography in the project area ranges from approximately EL 1941 along the riverbed at the dam site to as high as EL 2180 on the ridges to the south and north of the dam. The highest peaks in the dam area are the EL 2570 Khalo La Tenesolo Range 3.5 km to the northeast and the EL 2625 Sehlaba Sa Likalaneng Range 3 km to the west of the site. Intermediate erosional levels, primarily between EL 2010 and EL 2060, occur in a number of relic oxbows in the reservoir area, notably the Ha Mohlabane, Ha Piti and Ha Tsapane oxbows 0.7 km northwest, 1.2 km west and 4.5 km north of the dam site, respectively. Steep sided tributaries, including the Likalaneng, Jorotane and Bokong Rivers and various other minor gullies, streams and tributaries, cut across the terrain at numerous localities (fig 5.2-1). Many prominent topographical features and secondary streams and tributaries in the area are aligned along an east-west axis, which corresponds to the overall structural fabric in Lesotho.

The slope of the upper left abutment area is convex and the general thickness of the soil and weathered rock overburden is between 0.5 and 2 m thick with localized deeply weathered zones and occasional spheroidally weathered boulder core stones and outcrops. The lower left abutment is slightly concave in profile and has a slope ranging from 16 to 30 degrees. Thicker and more extensive soil cover than the right slope characterizes this slope and this is a characteristic of the whole valley. It consists of highly weathered basalt and residual soil, often with spheroidally weathered boulder core stones, boulders and gravelly, sandy and clayey colluvium. Relic superficial slumping is evident on the lower slopes. The thickness of soil and completely to highly weathered rock overburden is variable, ranging from less than 0.5 m in the central abutment area to 5 m in some of the slumps and up to 7 m or more in the deeply weathered residual soil and boulders.

The right abutment is located on the lower slope of a prominent east-west ridge, which was formed because of the erosion of the Senqunyane River meander downstream from the dam site. The ridge is about 800 to 850 m wide at the river level with the crest of the ridge at EL 2180, i.e. approximately 240 m above the river level. The right abutment has an average slope of about 33°. Thin discontinuous deposits of soil with numerous outcrops of exposed bedrock cover the slopes (fig.5.3-6; 5.3-7; 5.3-8 and 5.3-9).

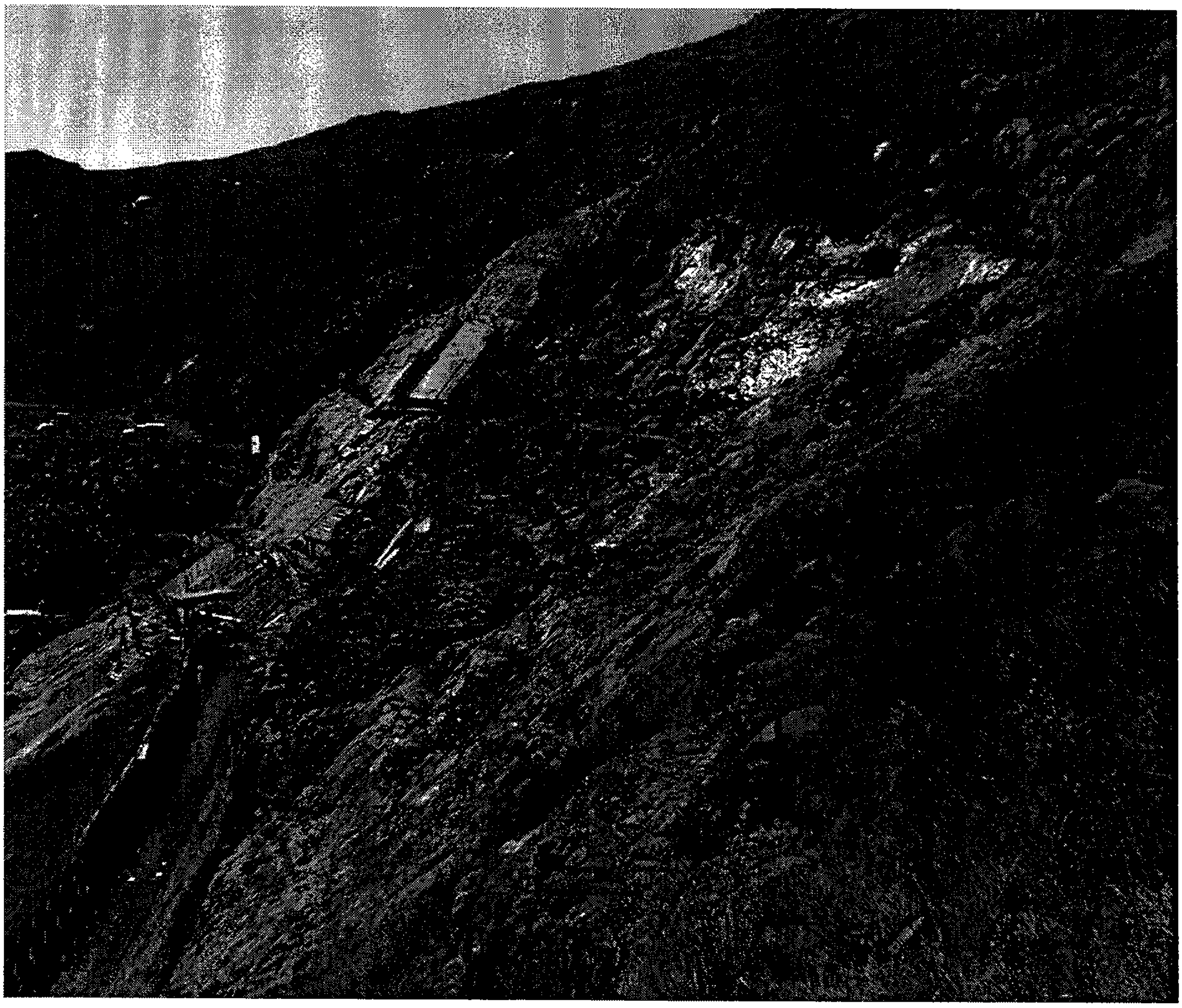


Figure 5.3-6: Right Abutment Slope with thin Soil Cover in Places

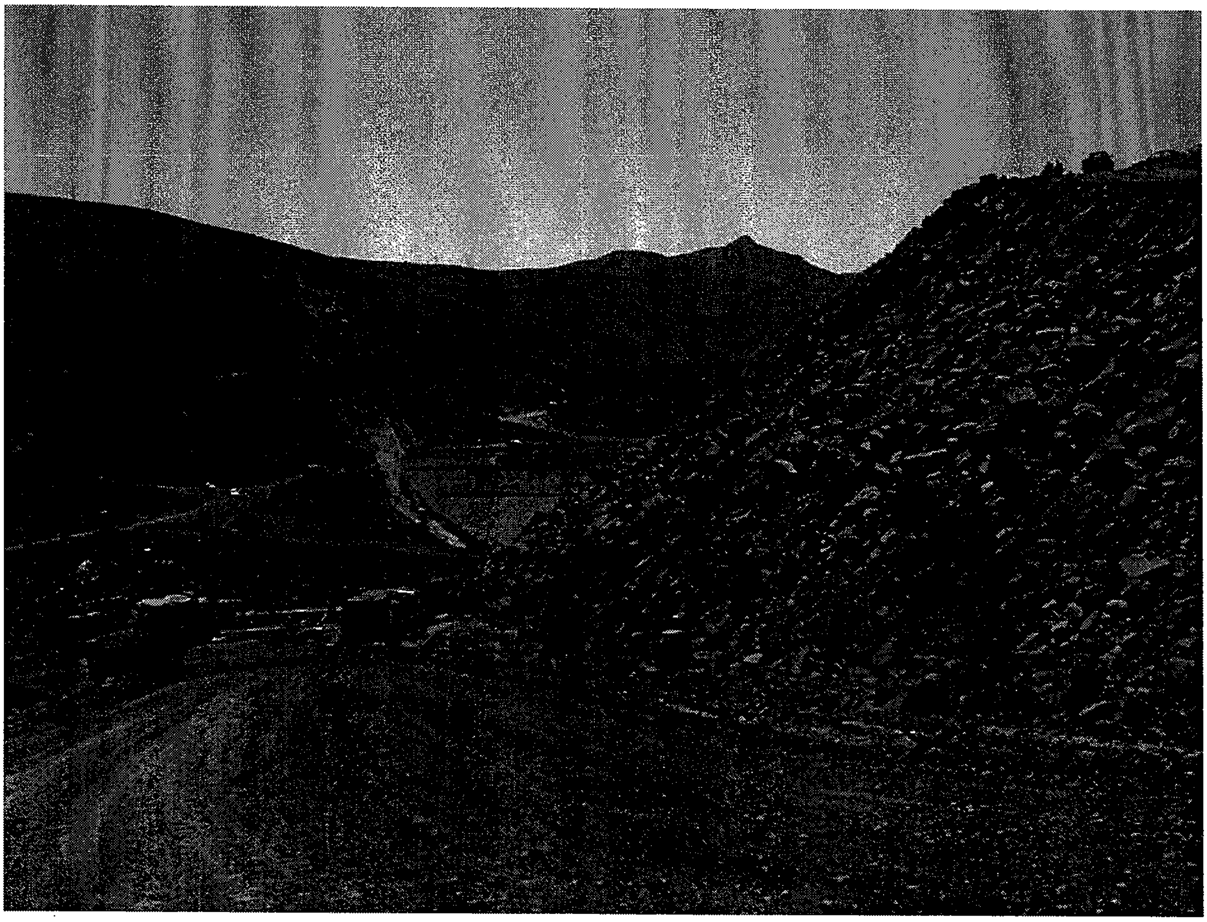


Figure 5.3-7: Left Abutment with thicker Soil Cover (Photograph was taken facing in the NE Direction)

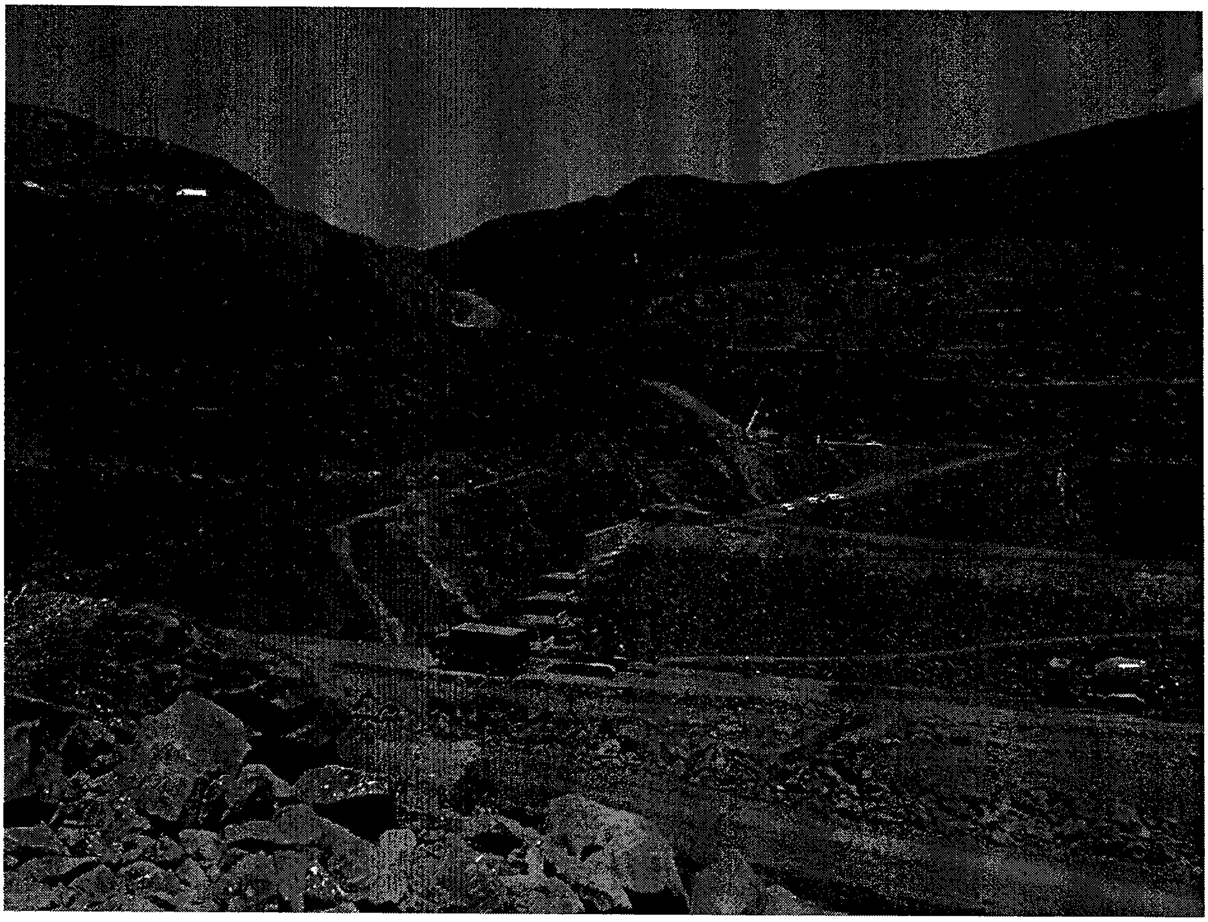


Figure 5.3-8: Slopes on the right Bank above the Likalaneng and Senqunyane Rivers Confluence.

The riverbed and flood plain of the Senqunyane River has a width of about 70 to 90 m at the dam site. Exposed basalt bedrock occurs in the river floor over most of its course at the dam site. Relatively thin deposits of sand, gravel and boulder alluvium are sporadically distributed along the riverbanks and riverbed. A localized 3.5 m deep buried river channel (palaeo channel) was found below the alluvium on the left bank of the river crossing the plinth. Low lying alluvial terraces up to approximately 3 m thick occur on the left bank and sporadically on the right bank. An alluvial terrace was found on the left bank at approximately 18 m above the river in the plinth area.

The Likalaneng River flows through a steep-sided meandering gully where the riverbed is about 10 to 20 m wide. It joins the Senqunyane River immediately upstream from the dam site at the proposed cofferdam location where a shallow (<3 m thick) alluvial outwash deposit has formed. Basalt bedrock is exposed for most of its length within the project area and a 5 m high waterfall is located at about 0.7 km upstream from the confluence, at the Ha Piti oxbow.

5.4.1 Description and Classification of the Basalts

5.4.1.1 Durability Aspects and Field Classification

Since the inception of the Lesotho Highlands Water Project (LHWP), the engineering properties of basalt rocks, especially their durability when used in water retention and transfer structures, has been the subject of a considerable number of studies. These studies have adopted visual, mineralogical, textural, mechanical, chemical and geo-chemical test procedures in an attempt to characterize the durability of the various basalts and to formulate an understanding of the likely causes and mechanisms responsible for the variable durability of the various basalt types. The mineralogical properties of the various basalts types have played an important role in the chemical and physical durability of these basalts and a comprehensive classification has been developed to allow an assessment of the potential durability of the basalt to be made as discussed in chapter 3. The durability aspects and field classification are well documented in the feasibility, planning and design studies of Phase 1A and 1B. The following sub-sections cover in summary zonation, mineralogy, distribution, structures of basalt and lithology in general.

5.4.1.1.1 Zonation of Basalts

The basalt flows at Mohale Dam typically exhibit a vertically graded zonation based on the fundamental division into non-amygdaloidal and amygdaloidal types. Usually, the highly amygdaloidal basalt (HAB) occurs at the top of most flows. This grades downwards into the moderately amygdaloidal basalt (MAB) and non-amygdaloidal basalt (NAB) at the centre of the flow. A zone of tubular amygdales typically occurs at the base of the flow, which may be MAB or HAB. A slightly amygdaloidal basalt (SAB) category, which has less than 1% amygdales has been used in the Feasibility Study and occurs locally within or as a transition to the NAB. This category has been abandoned for the LHWP Phase 1A and Phase 1B projects as its properties have very similar to the NAB. In thicker flows, doleritic basalt (DB) may occur near the centre of the flow. This occurrence has often been observed in the Mohale Tunnel during excavation. Occasionally extremely thick occurrences of DB occur, such as have been found at Mohale Dam left abutment, river area and Quarry 2 area. DB is typically holocrystalline, coarser, and based on the percentage of amygdales and the visible soft clay content has been variably interpreted for the project, see Chapter 3. The interpretations presented in the figures have used the Tender Design classification.

Two other types of basalt have been recognized in the Mohale area, both of which are only locally developed. Micro-amygdaloidal basalt has been recognized in many places occurring as thin bands within amygdaloidal basalt or as distinctive zones within thicker flows. It has been defined as amygdaloidal basalt with amygdales less than 1 mm in diameter. Flow breccia has been often found as a variable concordant body at the top of the flow or as a discordant body associated with the edge of flows and volcanic vents. It typically comprises variable basalt types showing distinct inclusions and veins of other basalt often with zones of intra flows and a welded breccia type appearance. Typically, zones of basalt and zeolite filled veins and blobs occur interspersed with the basalt. The components of a typical flow are shown in table 3.2-1.

A number of variations in the structures on the edges and within flows can be recognized. These have been related to differing forms of emplacement flow-edge or flow-top breccias, intrusive or extrusive veining and inclusions of parts of previous flows in subsequent flows. In addition, various pyroclastic and sedimentary deposits can be interspersed within the basalt layers including tuff, tuffite (a mixture of pyroclastic tuff and basalt) and layered fine silty sandstone. These sedimentary layers are rare and generally less than 300 mm in thickness, and cannot be traced over any significant distance.

5.4.1.1.2 Mineralogy

Plagioclase, pyroxene, olivine, glass (devitrified) and accessory opaques (ilmenite, magnetite and titano-magnetite) are the primary constituent minerals of the basalts at the Mohale Dam site. The basalt successions have experienced extensive smectitisation and/or zeolitisation alteration processes, subsequent to its deposition, which has led to an alteration of its primary mineralogy. The alteration of the primary minerals and original glass has resulted in their transformation or replacement by smectitic swelling clay minerals. Olivine, which is found in the doleritic basalt bodies at Mohale Dam, shows variable alteration in the form of haematisation and smectitisation. Other primary minerals, plagioclase and pyroxene, exhibit less alteration (usually <1%).

Amygdale fillings in the basalts are composed mainly of ziolites or smectitic clay, with occasional calcite, chlorite and silica. The amygdale filling is generally complex with multiple minerals present, such as cores of zeolite with riming of calcite, illite clay, etc. Generally, zeolites comprise the majority of the amygdale filling. X-ray diffraction testing indicates that the zeolites occurring in the various basalt types consist primarily of scolecite, epistilbite, stilbite and occasional accessory amounts of laumontite, natrolite, mesolite and heulandite.

5.4.1.1.3 Basalt Distribution and Lithology

The basalts are predominantly of olivine-rich tholeiitic composition. Plagioclase, clinopyroxene, and olivine are the main constituents. The crystallization stopped when the residual magma solidified as glass. As a result, at the top, glass predominates and in the centers of thick flows, full crystallization was sometimes achieved. The olivine and glass are generally dentrically altered to other minerals, particularly montmorilonite. The processes of alteration are the key to the durability of the basalt. Olivine is usually completely replaced by the clay mineral montmorilonite, although other minerals may be present. Similarly, the dominant replacing mineral for glass is montmorilonite. The montmorilonite is usually visible as dark soft spots on the surface of a rock specimen. The various investigations indicated that the mineral formed in the Mohale Dam area is stilbite, epistilbite, and scolecite with accessory amounts of mesolite, heulandite, natrolite and laumontite.

Occasionally thin tuff and taffite layers occur within the basalt flow sequence and the frequency of such interlayers is believed to increase with depth in the basalt succession. The deuteric deposits in the amygdales vary in composition. The deposits may comprise zeolite (stilbite), calcite, rarely silica, in the form of quartz, agate and chalcedony and in most cases montmorilonite occupies the outer zone.

Basalt Flows: - The site is made up of a series of sub-horizontally layered basalt flows. Individual flows vary from less than 1 m to as much as 40 m in thickness. They occur as lensoid, lobate, tabular or bun-shaped bodies with a limited lateral extent forming a complex pile of variable individual flows. Mapping and drilling show that the thicker major flows are continuous within a 1 to 10 m band for distances in excess of 1 km. Broad zones of thinner flows are consistent across the site while locally inclined banding and flows up to 30° from the horizontal have been recorded. The continuity of the flows tends to vary according to the thickness of the flow with the thicker flows extending for greater distances than the thinner flows. Due to the complex gradational nature of the basalt in the flows both vertically and horizontally, it is difficult to extrapolate geological boundaries over any length. Where possible, the flow contacts have been identified in both the drilling logs and on exposures.

Doleritic Basalt: - A significant variation in the basalt litho-stratigraphic succession over the project area occurs with the presence of a consistent sequence of prominent outcropping jointed doleritic basalt bodies, forming the central core of a number of very thick (up to 70 m) basalt flows. These can be identified over the various parts of the dam and Quarry 2 site. They are:

- An approximately 200 m long outcrop of jointed doleritic basalt that occurs in the upper left abutment from approximately EL 2010 to 2050.
- An outcrop of doleritic basalt at a similar elevation on the right flank of the Likalaneng river in the Quarry 2 area extending from the confluence with the Senqunyane river upstream for approximately 1 000 m;
- A 100 m wide doleritic outcrop is found on both banks of the river in the dam plinth area, with an upstream inclined extension on the right bank.

Various other minor outcrops of doleritic basalt related to the central core of thicker flows could be seen over the site. The doleritic basalt at EL 2010 to EL 2055 on the left bank of the Senqunyane River and on the right bank of the Likalaneng River form part of a distinctive flow body typically 100 to 200 m wide and up to 60 to 70 m thick with a distinctive maroon coloured flow breccia zone at the base. This body has been traced for a distance of 2 km to the south west of the dam site in the Quarry 2 area and as far as 3 km to the northeast where it disappears beneath the Khalo La Tenesolo Range. It extends between EL 2010 and EL 2070 on the left abutment and between EL 2005 and 2075 on the right bank of the Likalaneng River. The doleritic basalt found in the river area at the plinth also forms the central part of a very thick basalt body, which is up to at least 70 m thick. It is approximately 30 m thick and grades downward into non-amygdaloidal basalt with distinctive flow characteristics on the lower boundary between approximately ELS 1917 to EL 1926 (approximately 26 m and 16 m below the river bed). The doleritic basalt forms outcrops on the lower left abutment in the plinth river area, where it grades upwards into coarse non-amygdaloidal basalt above EL 1965. Upstream from the dam plinth on the right bank, the doleritic basalt grades upward into one-amygdaloidal basalt while at the plinth it forms a closely spaced bladed jointed zone on the upper contact with a flow breccia zone.

5.4.1.1.4 Structure

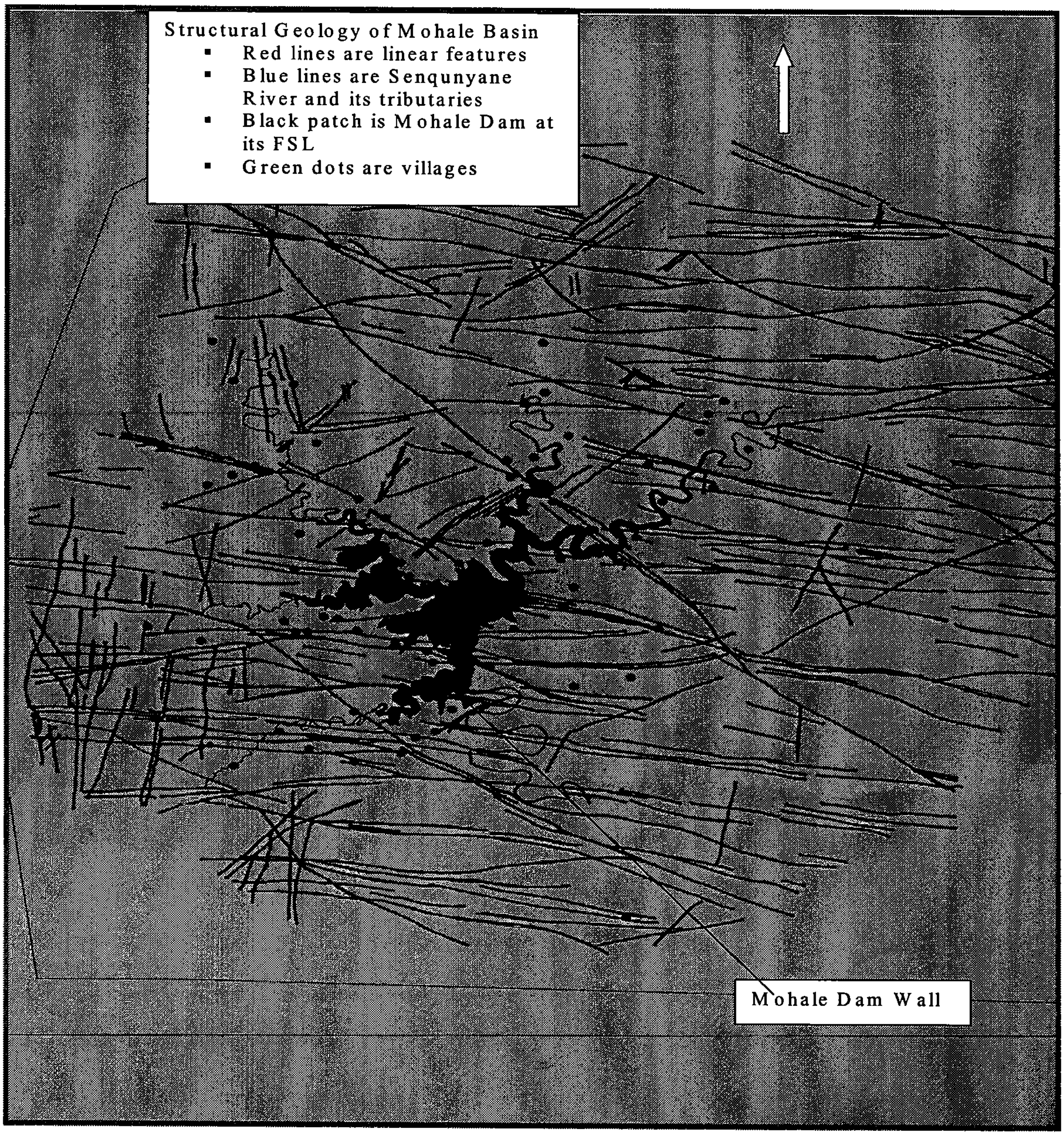


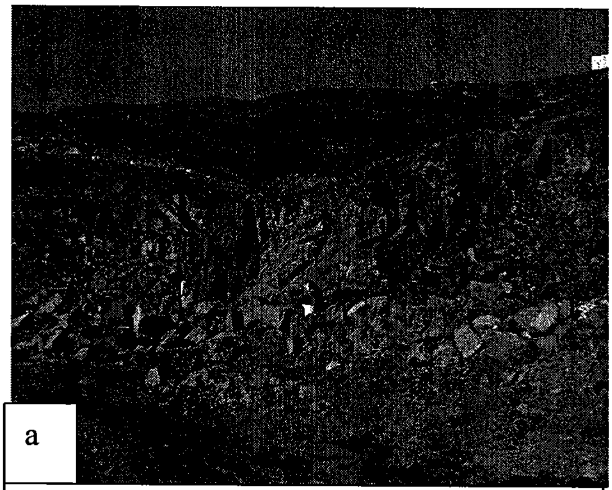
Figure 5.4-1: A Map of the Mohale Basin showing linear Features with a general Strike E-W, NE-SW and NNW-SSE.

The rock mass consists of sub-horizontally layered rock cut by steeply inclined to vertical tectonic joints and fracture lineaments. Three tectonic joint sets with orientations of E-W, NE-SW and NNW-SSE have been consistently found across the site as illustrated in fig. 5.4-1. Joints are of medium to high persistence (3 to 20m), widely to very widely spaced and sub-vertical to vertical. They are often infilled with zeolite, calcite or quartz and features are generally healed. Infilling thickness generally varies from 1 mm to 5 mm (fig. 5.4-2 a, b, c).

The right bank at the dam site is traversed by a series of E-W trending veined fractured lineaments (fig.5.4-1 a, b, c) which are generally very closely jointed and cemented crushed zones with zeolite filling and occasional calcite and silica filling. Two of these features cross the dam, the largest one of which outcrops just above dam crest level at about EL 2085 on the right abutment while a second smaller feature crosses the dam plinth lower down the slope at about EL 2000. Both these features and similar features outcropping higher up on the right abutment slope form part of the Ha Piti lineament swarm. They are extremely persistent and can be traced across the entire project area. The collective lineament zone can be traced over tens of kilometers crossing all rock types including the doleritic basalt body in Quarry 2.

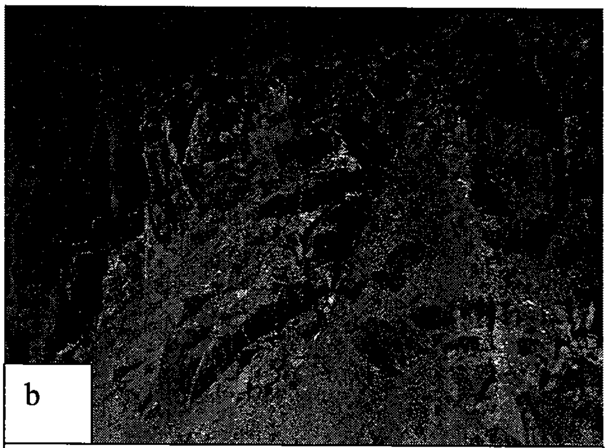
A number of open vuggy joints are found in boreholes LDR 204 and LDR 206 in the river area. They are typically rough zeolite, calcite-filled crystal growth and are associated with drilling water loss, and high water takes during water acceptance tests. These features were interpreted from the core orientation to lie mainly in a NNE-SSW direction. An interpreted structural shear zone feature is aligned NE-SW and crosses the dam plinth on the right bank of the river and was found in boreholes LDR 207 and LDR 217.

A number of lineaments were interpreted from aerial photographs to cross the left abutment area (fig. 5.4-1). However, with the exception of a zone of very closely spaced joints intersected in the spillway at borehole LSR 203, no major structural features could be confirmed in the field in the upper left abutment area. The trench exposures on the plinth in the upper left abutment indicated a number of E-W and NE-SW oriented joints and a zone of prominent medium spaced joints between EL 2045 and EL 2060. Stress relief joints occur along the valley slopes in the project area. Typically, these joints strike sub-parallel to the slope and dip 20° to 50° towards the river. They vary from curvilinear to planar and can persist for more than 50 m at some locations. Boreholes at the site have encountered stress relief joints at vertical depths that are typically less than 20 m beneath the surface and are generally found in the upper 3 to 5 m. Penetrative sidewall weathering along these joints is common.



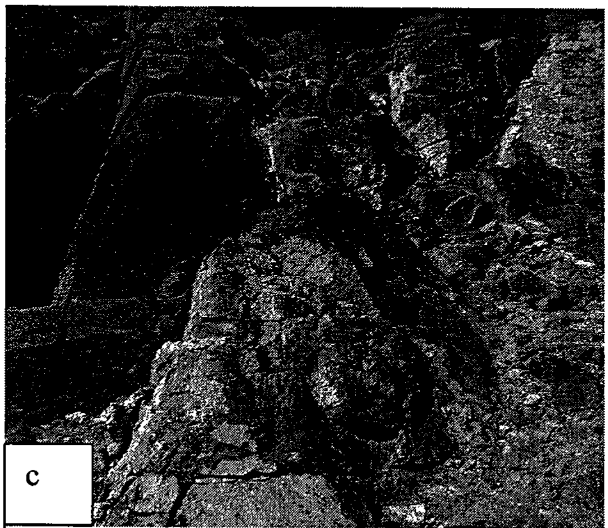
a

a is furthest



b

b is closer



c

c is closest

Figure 5.4-2: Photograph showing lineaments with zeolite infill. The Photographs were take in Quarry 2 facing southeast

5.5 Site Hydrogeology

There is no consistent hydrogeological pattern for the site. The groundwater flow regime appears to consist of a number of aquifers, i.e. a shallow aquifer in weathered, open surface joints and, at depth, a series of perched water tables associated with specific discontinuities, such as open flow contacts, veined lineaments and contacts.

A summary of the range of water levels and water acceptance tests over the site is given in Table 5.5-1. The groundwater levels recorded in boreholes during the various investigations are in general found to be variable. Many of the recordings reflect the shallow perched groundwater conditions while the impermeability of the basalt in most boreholes has resulted in a lack of sensitivity to rainfall and climatic variations. No sustained record of waterlevel readings in particular boreholes has been possible due to vandalism of certain boreholes and piezometer monuments. However, the limited available data indicate that the water levels generally tend to be deeper on the upper flanks of the valley and shallow near the river area. Most of the recorded water levels appear to be related to the presence of ground water in the upper soil and weathered zone at depths of between 5 and 10 m.

Boreholes drilled in the river area (LDR 103, LDR 205, LDR 206, LDR 217) have encountered artesian flow, associated with partially open and zeolite filled vuggy joints or fracture zones. The artesian flow from LDR 103 (drilled during the Planning Study) appeared to be interrupted after the drilling of borehole LDR 206 drilled during the Tender Design, indicating possible connection between joints and fracture zones. The flows do not appear to be affected by the rise and fall of the river. The water level in the raised alluvial terrace deposits approximates to that of the river level.

Figure 5.5-1: Dam Area and Quarry 1. Upstream Main Dam Site in Construction and Diversion Tunnel Outlets. Photograph was taken looking upstream from the Engineer's Office

Fig. 5.5-1 fits nicely with fig. 7.3-1 and 7.3-2. The Diversion Tunnel Outlets, Dam Embankment, Coffer Dam and Quarry 1 can be clearly seen on fig. 5.5-1.

Table 5.5-1 Water Levels and Water Acceptance Test Results, Dam Area (Source: MCG)

LOCATION	RANGE OF WATER DEPTHS RECORDED IN ALL INVESTIGATIONS (m)	RANGE OF WATER ACCEPTANCE TESTS RECORDED IN ALL INVESTIGATIONS (Lugeon)
Dam Left Abutment	3.0 – 51.0	0 – 32*
Dam River	4.8 – Artesian	0 - > 100*
Dam Right Abutment	31.1 – Artesian	0 - > 100*
Spillway	2.7 – 9.1	0.1 – 0.3
Tunnel Outlet Portal Area	6.7 – 14.5	N/P
Quarry 1	4.4 – 43.0	N/P
Quarry 2	4.1 – 57.3	10

*Test Pressure could not be attained in one or more tests
N/P= Tests not performed

The water levels in boreholes drilled through the structural features and the lineaments are dependent on the hydrogeological conditions surrounding these features and were found to be variable. For example in the structural lineament feature at EL 2000, artesian water was found which increased after heavy rains. In the lineament at EL 2080, the water level was recorded at 11 m in borehole LDR 07 during the Feasibility Study and was found at a depth of about 31 m in borehole LDR 210 during the Tender Design. An intermittent spring occurs where this lineament intersects a gully near the downstream toe of the dam.

5.6 Dam Site

5.6.1 Geology

The geology of the Mohale Dam is discussed in the LHWP geotechnical investigations and especially the MCG Final Report where most of the information in this thesis was extracted. Fig. 5.6-1 presents the longitudinal section of the plinth line and its summary logs, dam site investigation points and geology.

Figure 5.6-1: Longitudinal Section – Plinth Line

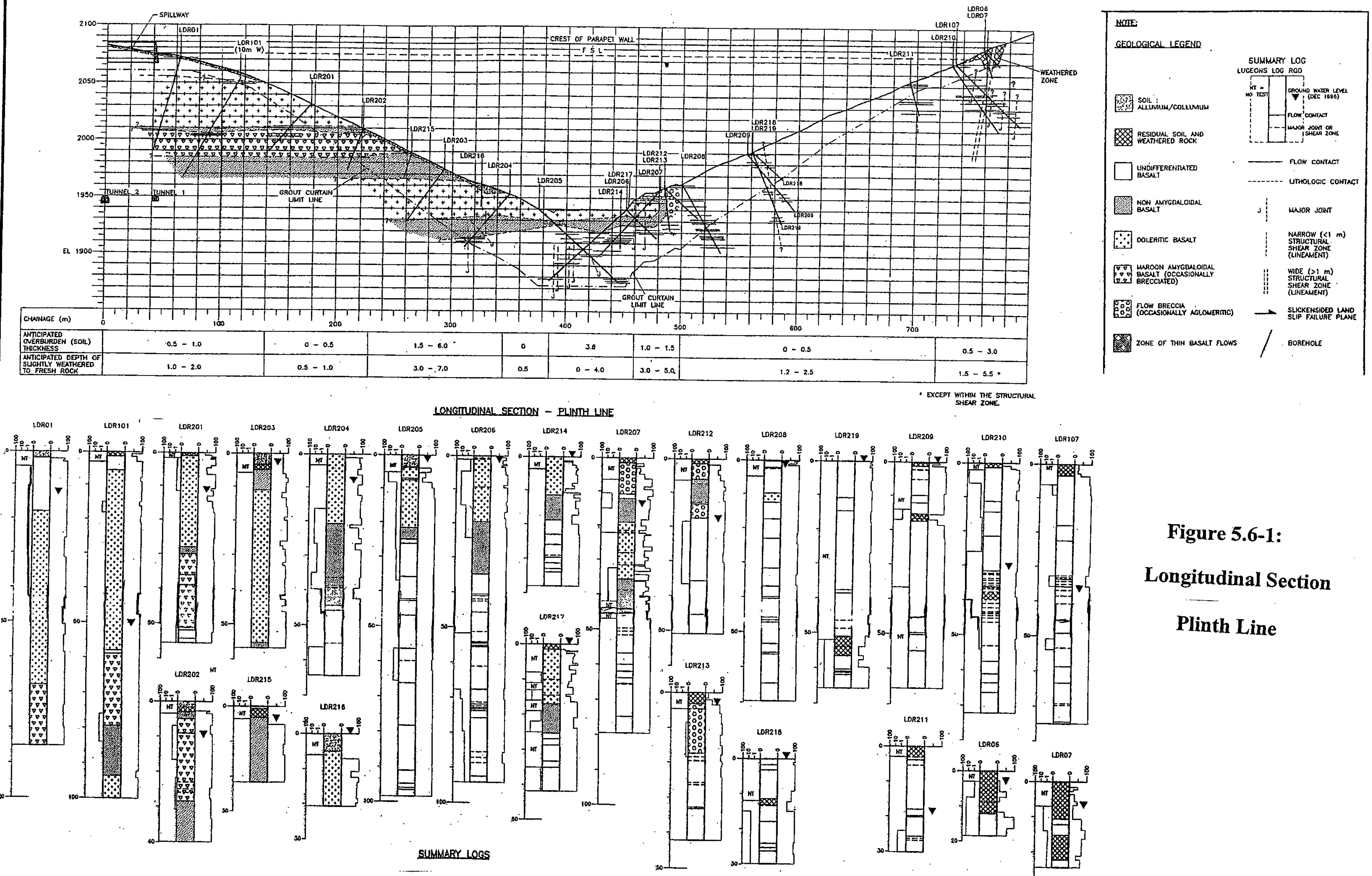


Figure 5.6-1:

Longitudinal Section

Plinth Line

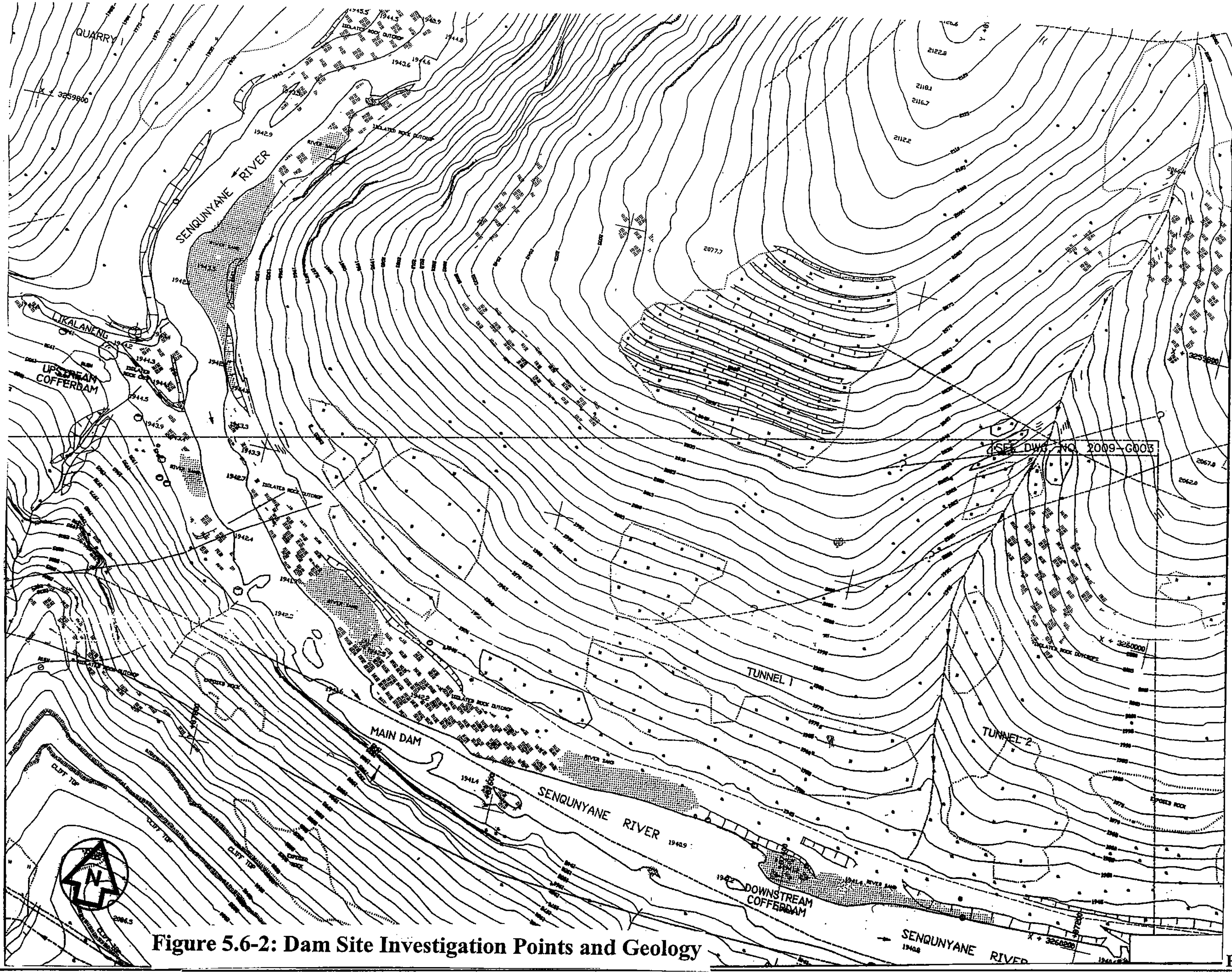


Figure 5.6-2: Dam Site Investigation Points and Geology

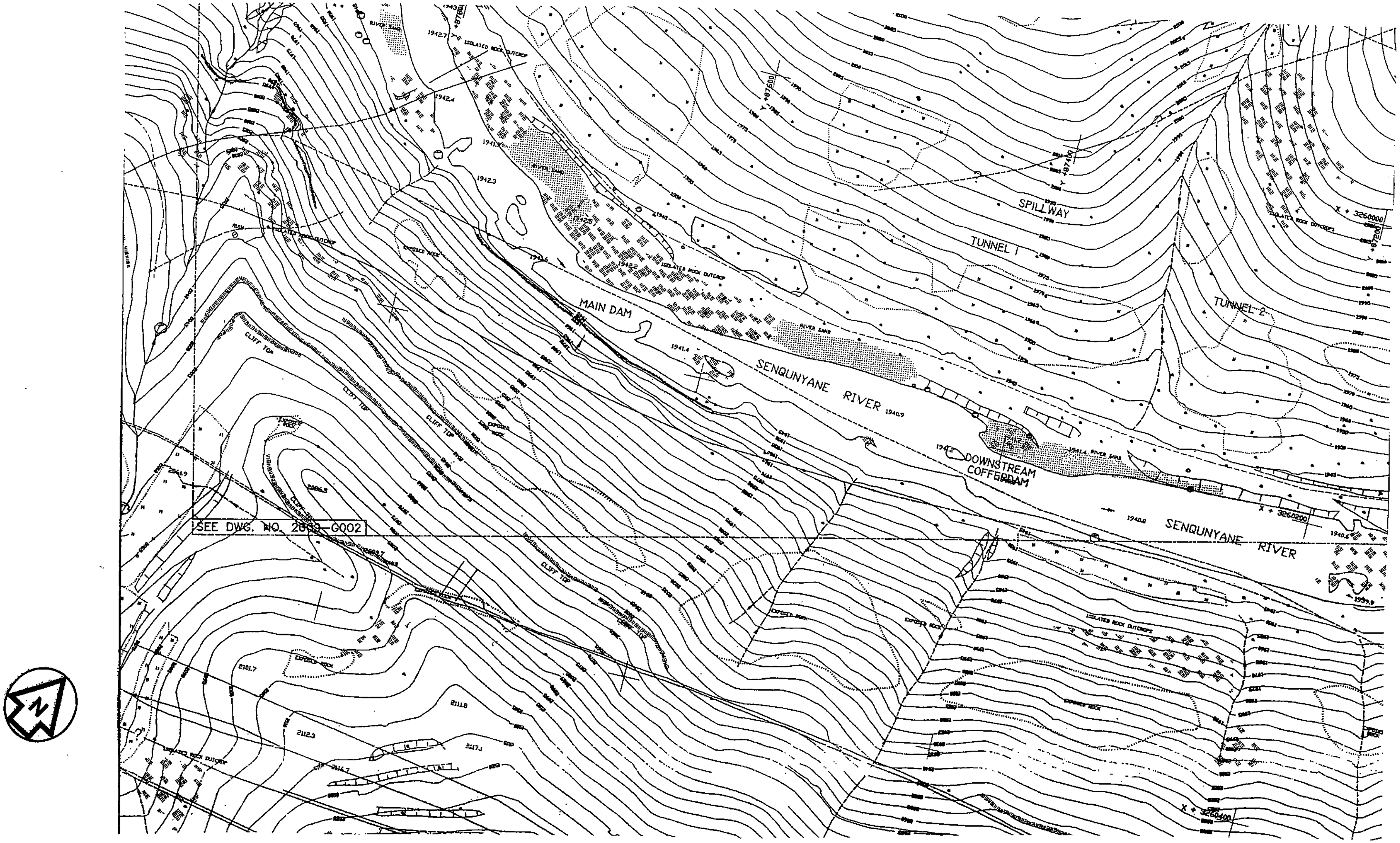


Figure 5.6-3: Dam Site Investigation Points and Geology

5.6.1.1 Left Abutment

A deeper and more extensive cover of overburden material than the right abutment characterizes the left abutment (fig. 5.3-6, 5.3-7, 5.6-4 and 5.6-5). The plinth on the upper left abutment passes through a series of terraced maize fields where clayey and gravelly soil, which is between 0.5 and 2.0 m thick, has been found. The soil cover tends to thin out to the west where scattered basalt rock outcrops are found.

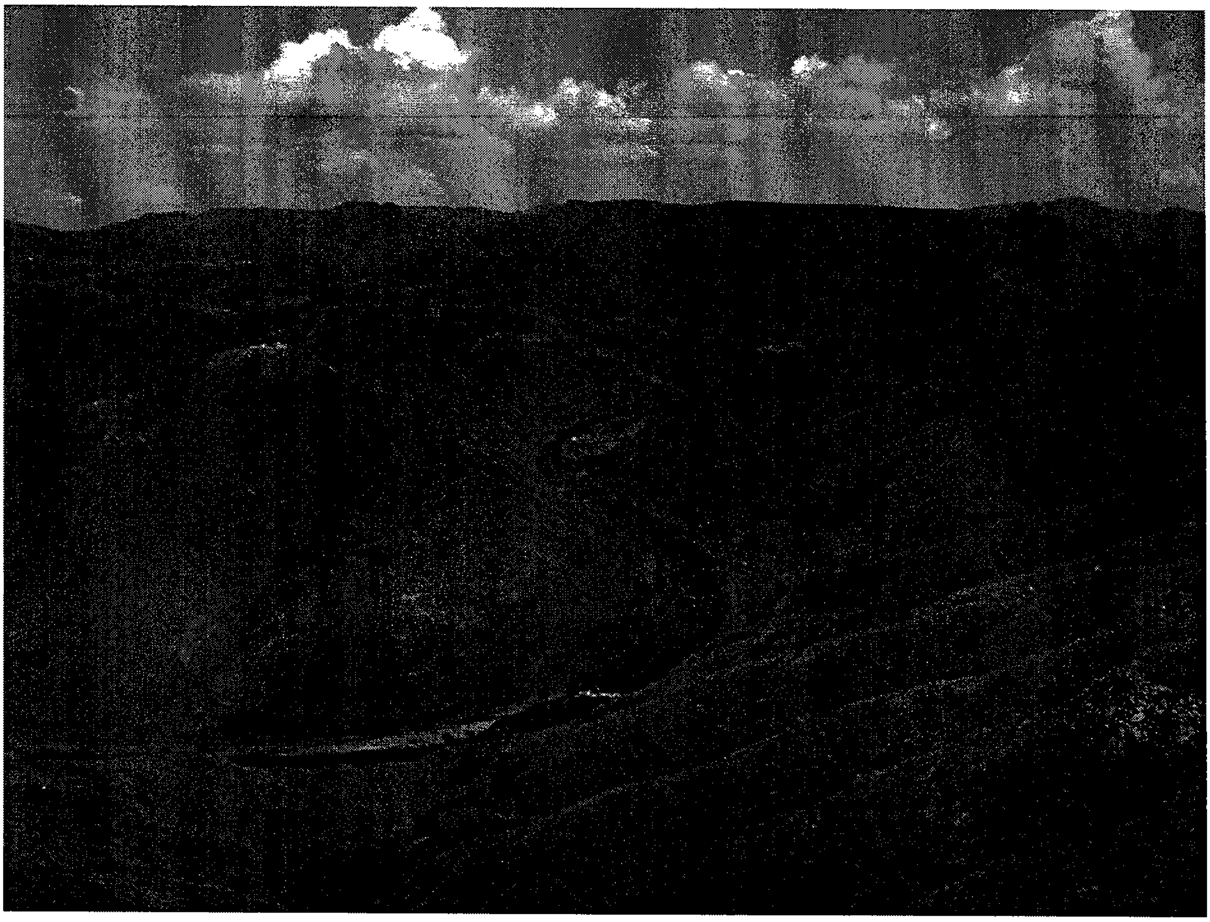


Figure 5.6-4: Right and Left Banks downstream from Mohale Dam Wall

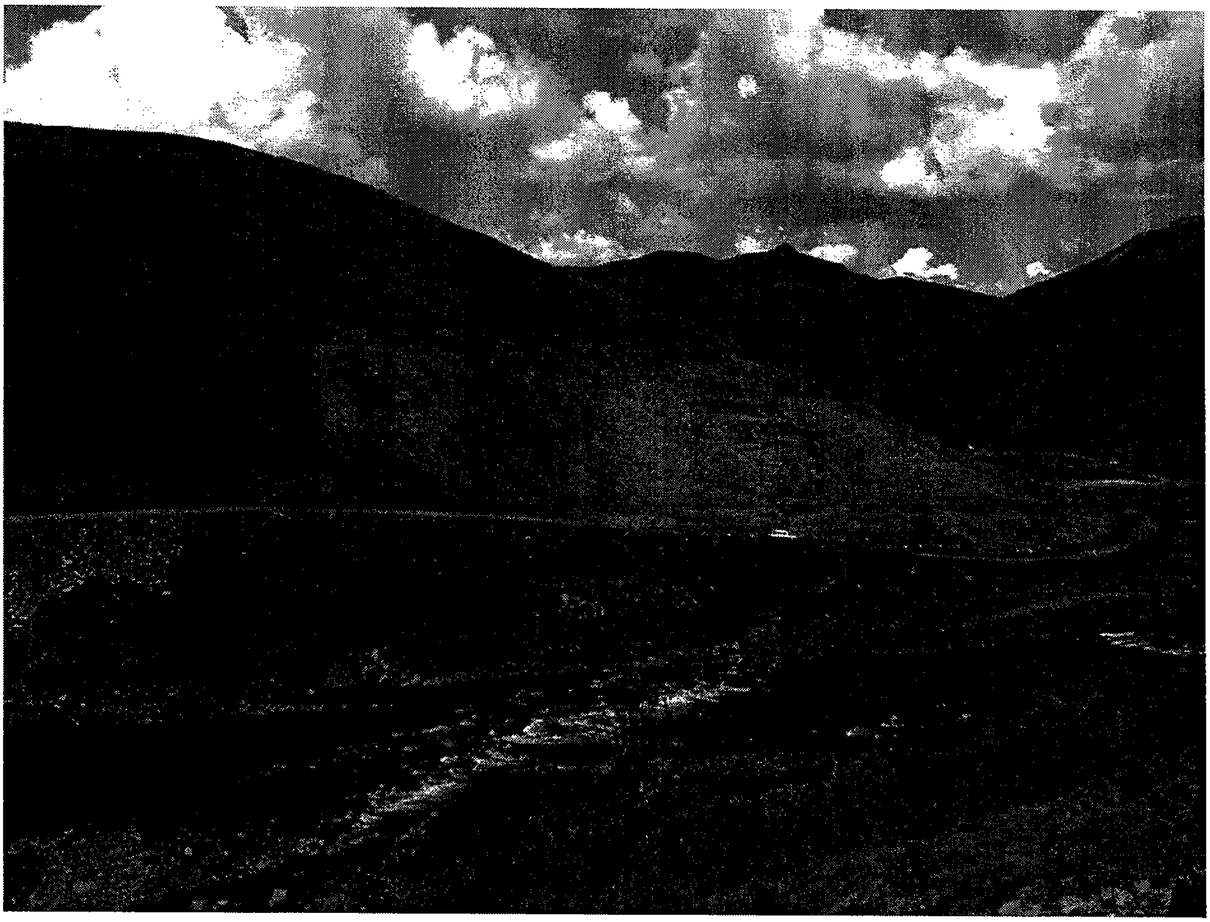


Figure 5.6-5: Left Abutment Slope Soil Cover

On the central left abutment plinth line from approximately EL 2045 down to approximately EL 2010, numerous outcrops of doleritic basalt boulders are found. They become less prominent below EL 2020 down to EL 2010. These boulders extend downstream along contours for a distance of approximately 180 m where they disappear at a natural depression near the dam downstream toe.

The ground surface is slightly concave in the lower slopes below EL 2000 where steeper slopes occur on the upper side of a shallow depression lying between the plinth and the centre line of the dam. A palaeo slump occurs immediately downstream from the plinth below EL 1980. A number of scarp faces can be seen in the upper part with characteristic flattening of the slope in the lower part at EL 1955. The soil is predominantly colluvial, comprised of silty and gravelly clay. Thicker colluvial soils up to 5 m in thickness occur in this area while sandy and gravelly residual soils with large spheroidal boulders were found at depths of between 4.0 and 6.9 m below EL 1975. Locally deeper weathered zones may be present.

5.6.1.2 River Area

The plinth crosses an outcrop of doleritic basalt from approximately EL 1955 and extends down to the river at approximately EL 1946. Alluvial boulders and gravel ranging from <0.5 to 2.5 m thick cover most of the doleritic basalt on the left bank. The outcrop can be traced through the river to the right bank. On the left bank an alluvium filled buried palaeo channel, approximately 6 m wide and about 2.5 m deep below the adjacent rock levels in the river bed was found in the 2.5 m wide trench excavated along the plinth, this channel was nicknamed "Isaac-Pool" because it was discovered by an Engineer named Isaac Diaho of LHDA. He was supervising the excavation. The author was in charge of the operations on the LHDA side. The deepest point recorded in the buried channel was EL 1938.2. The channel is aligned parallel to the river and is cut into the bedrock with near vertical smoothly worn sides and undulating floor. The extent of the channel both upstream and downstream from the plinth is not known although it was encountered at a depth of 3.8 m below surface (vertical depth) in borehole LDR 205 drilled inclined at 45° approximately 12 m downstream from its exposure on the plinth. The buried channel could not be traced further downstream in trenches LDP 129 A and LDP 130 or within seismic traverse LSI 15 or in the upstream cofferdam area.

Downstream from the plinth, the river section is approximately 80 m wide (fig. 5.6-4). The central channel tends to be rock bound over most of the dam area with localized coarse alluvium particularly on the left of the river channel. The doleritic basalt outcrop at the plinth area forms a localized ridge resulting in a shallow rapid in the river channel with alluvium cover. Immediately downstream, washed out gullies up to 1 m deep and locally filled with boulders and cobbles are present in the river channel. A number of large (>1 m diameter) reddish coloured boulders are found in the river area and are interpreted as scree originating from a rock fall in the cliff at EL 1995 on the right bank. Alluvial sand, gravel, cobble and boulder deposits ranging from less than 0.5 m to 4 m thick occur on the left bank of the river with localized outcrops of rock, forming small rapids in the river channel. The alluvium is found predominantly on the left bank. Localized patches of boulders and sandy alluvium up to 1.4 m thick occur on the right bank.

5.6.1.3 Right Abutment

A uniform slope with laterally persistent ledges and cliffs defined by resistant horizontal basalt flows characterizes the right abutment. Outcropping basalt is found from the river area up to approximately EL 1975 with localized thin soil cover. Small rock bound stream gullies occur on the slope, one of which crosses the plinth line at EL 1970. Above this is a discontinuous veneer of colluvial slopewash (<0.5 m thick), which characterized the abutment with numerous rock outcrops in the form of terraced rock ledges and exfoliated exposure surfaces. Overburden on the right abutment consists mainly of clayey silt and gravel colluvium. Recently the excavation blasted for the access road above the right abutment, fig.5.6-1, has led to the deposition of scattered angular blasted basalt spoil lying on the downstream part of the dam.

Figure 5.6-6: Right Abutment Access Road Cut. Two flows can be observed with some podsols lenses. This Photograph was taken facing southeast in Quarry 2.

5.6.2 Bedrock Geology

5.6.2.1 Left Abutment

The stratigraphy on the left abutment plinth is dominated by two thick bodies of doleritic and non-amygdaloidal basalt, which are separated by a distinctive zone of maroon moderately to highly amygdaloidal basalt, which exhibits flow breccia and agglomeritic features. This comprises a very thick flow of about 60 m thick from approximately EL 2070 extending down to approximately EL 2010. The layer comprises an upper non-amygdaloidal basalt zone with less than 10% ddc which grades into a thick lower doleritic basalt core zone below EL 2055. A thin (approximately 2 m) non-amygdaloidal basalt zone with typical flow characteristics occurs on the lower boundary of the body. The body extends upstream approximately 50 m where it abuts against the typical basalt flows and downstream for about 100 m to 130 m where a localized zone of maroon flow breccia outcrop is found in places

The thin non-amygdaloidal layer is underlain by a zone of distinctive maroon to dark grayish red, moderately to highly amygdaloidal basalt up to 25 m thick, extending down to approximately EL 1984. The zone includes a number of intra flows with typical tube amygdales on the upper surface and includes large and smaller veins and bands of micro amygdaloidal basalt and doleritic basalt. Below approximately EL 1992, it becomes non-amygdaloidal with bands of moderately amygdaloidal basalt and welded basalt veins. A breccia structure occurs in places with inclusions of basalt, zeolite stockwork and veining. This basalt structure is, however, completely intact and is not typical of faulted type breccia.

The maroon-to-dark grayish and moderately to highly amygdaloidal basalt layer grades into a very thick (up to 70 m) non-amygdaloidal and doleritic basalt body. It extends to at least 26 m below the river level. The overlying distinctive maroon layer tends to grade into non-amygdaloidal basalt with less than 10% ddc at approximately EL 1984, which in turn grades into doleritic basalt at EL 1980 to EL 1972. The lower (approximately 40 m) body forms the prominent doleritic basalt outcrop in the riverbed and grades into non-amygdaloidal basalt at depth. The upper and lower contacts of the doleritic basalt are gradational with the typically very strong, bladed doleritic basalt forming surface outcrops below EL1955. Typical flow characteristics occur on the lower contact at approximately EL 1917 in borehole LDR 204, EL 1929 in LDR 205, EL 1925 in LDR 207 and at EL 1919 in LDR 206.

The thick non-amygdaloidal basalt and doleritic basalt body grades into typical basalt flows approximately 40 m downstream of the plinth as seen in the boreholes LDR 110 and LDR 103 downstream of the plinth and the exposure in the cliffs on the right bank of the river. This corresponds well with the observed thinning of the non-amygdaloidal basalt and doleritic basalt layer found upstream from the plinth on the right bank between EL 1985 and 1941. This boundary has been interpreted to traverse diagonally across the left abutment of the dam. Downstream from the contact zone the basalt flow stratigraphy is similar to the right bank stratigraphy, i.e. with at least three thick flows (10 to 18 m) extending from approximately EL 1920 up to EL 1980 followed by a series of thinner flows up to EL 2020.

Weathering in the non-amygdaloidal basalt zones on the lower slopes of the left bank is typically penetrative with the formation of pervasive weathered vertical and sub horizontal joints and often spheroidal onion skin type joints with side wall staining and clay coating. Slickensiding has been observed along joints in this litho type probably indicating inter-block stress relief movement. In the bladed doleritic basalt, staining of the upper joints with little penetration has been observed. Stress relief jointing was not evident in the doleritic basalt.

5.6.2.2 River bed

The plinth in the river area is underlain almost entirely by doleritic basalt, which forms a prominent outcrop on both left and right banks and is traced through the river outcrops and by trenching through the alluvium on the left bank. The upper contact of the doleritic basalt on the left bank grades upward into non-amygdaloidal basalt below the thicker soil covers. The outcrop on the right bank forms a prominent ridge of bladed jointed rock, which is partly covered by alluvium on its downstream contact. On the right-bank slope the doleritic basalt grades upwards into closely to very closely jointed doleritic basalt, which is overlain by a distinctive outcrop of reddish flow breccia and agglomerate. This contact dips steeply downstream across the right abutment face, crosses the plinth at EL 1950, and exhibits flow contact breccia characteristics grading to typical basalt flows at approximately EL 1960. Downstream from the plinth in the right bank river area the typical basalt flow conditions are found, with a flow contact at river level EL 1941.5 approximately 30 m downstream while the left bank is covered by alluvium.

Three sub-horizontals to horizontal flow contacts are found to occur within the first 10 m below river level as observed in boreholes LDR 104 and LDR 109. Flow contacts and lithological contacts appear to be tight and welded. Some weathering along these contacts does occur such as observed in borehole LDR 109 where a weathered, tight sub-horizontal joint is encountered within 20 mm of the flow contact at EL 1936.

5.6.2.3 Right abutment

A consistent typical basalt flow has been inferred on the right abutment above the doleritic basalt body. A zone of thick (greater than 10 m) basalt flows occur from river level (1941 m) to EL 1975, immediately downstream from the plinth line, followed by a zone of mainly thin (less than 5 m) basalt flows to EL 2036. These are also mainly thick flows to Full Supply Level at EL 2075. Two thicker basalt flows have been identified, which define the upper and lower boundaries of the zone of thin flows. The lower thick basalt flow occurs from EL 1958 to 1970 at the dam and to EL 1975 near the downstream toe. The upper thick flow is found at the plinth between ELS 2038 to EL 2050. A consistently thick flow can be traced from approximately EL 2060 to EL 2070 in the upper slope area. This flow is overlain by a 10 m thick non-amygdaloidal basalt flow with greater than 25% ddc from approximately EL 2070 to EL 2080 at the dam crest level. The core from the non-amygdaloidal basalt in borehole LDR 101, which has been exposed to the weather at Katse Dam site, shows significantly more deterioration compared to the core taken between EL 2070 and EL 2080 after about 12 months of exposure.

The depth of weathering on the right abutment is generally restricted to the upper 0.5 to 2 m where weathered joints, which strike parallel to the surface, are common. Deeper weathering is associated with the structural lineaments, which strike approximately E – W across the right abutment at EL 2085 and EL 2000.

The structural lineament at EL 2000 has been observed on the surface to consist of an approximately 500 mm wide zone of weak weathered and closely to very closely jointed basalt with soft calcite and zeolite filled veins. A zone of weathering of about 1 m wide surrounding the veined lineament has identified at the surface extending to a depth of 50 m below the surface in borehole LDR 219. The structural lineament at EL 2085 consists of an approximately 3 m wide veined fracture zone consisting in turn of closely jointed rock, calcite and zeolite filled veins lies within an approximately 10 m wide zone of variably weathered basalt rock with calcite and zeolite filled veins and open joints. The weathered rock mass extends to a depth of approximately 10 m while slightly weathered rock mass conditions extend to a vertical depth of approximately 27

m. The main fracture zone is moderately weathered and slickensiding on some of the joints has been observed.

Weathered stress relief joints are well developed in boreholes LDR 211 and LDR 210 located in the vicinity of the veined fracture at EL 2085 but are not as well developed in boreholes LDR 04 and LDR 05 located away from the fracture. It is thus concluded that stress relief jointing increases near veined fractures.

5.6.3 Excavation Depth for the Dam

All over burden material, including the upper completely and highly weathered rock below the foundation of the dam has been removed. The palaeo channel encountered in the plinth area extends towards the downstream and this channel has been cleaned of all loose overburden and alluvium.

5.6.4 Excavation Depth for the Plinth

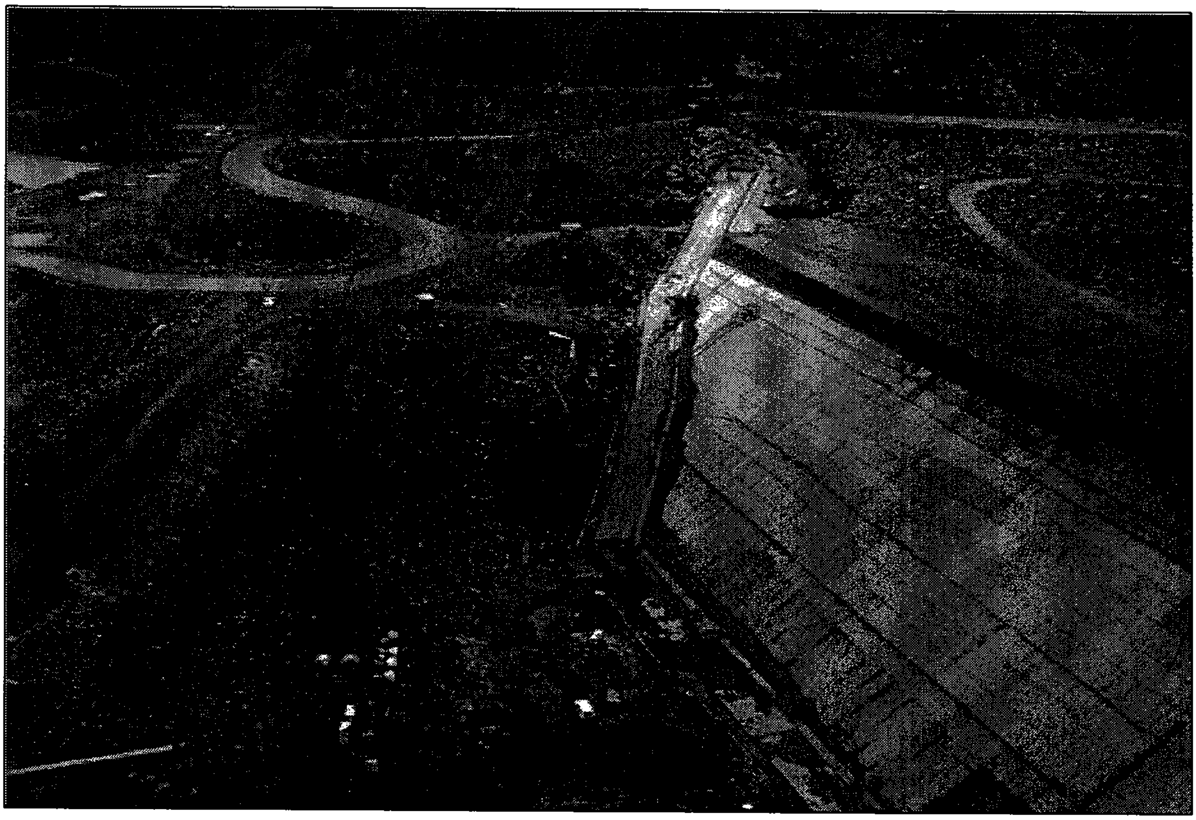


Figure 5.6-7: The Plinth left Abutment and Dam Face

Figure 5.6-8: The Plinth Right Abutment and Dam Face

All overburden including the upper highly and moderately weathered rock has been removed along the plinth (fig 5.6-7 and 5.6-8). The plinth line has been excavated up to competent, slightly weathered rock, according to the LHWP weathering classification; see Table 3.3 in Chapter 3. The topography is the major factor affecting the foundation excavation. In general, the valley slopes are uniform with localized low cliff faces and rocky outcrops. There is an approximately 10 m high cliff face on the lower right abutment. This cliff face required some shaping to fit the plinth geometry.

The alluvium in the palaeo channel (nick-named: Isaacs Pool) encountered under the plinth in the left river flank has been excavated and backfilled with concrete to place the plinth. The lineaments at EL 2000 and EL 2085 crossing the plinth have been excavated and treated appropriately, for example, with grouting and filters.

5.6.5 Foundation Preparation and Treatment under Dam Footprint

Removing overburden from beneath the entire footprint of the dam has been undertaken prior to making preparation of the foundation. The foundation has been specified to be equal or better than moderately weathered rock according to the LHWP weathering classification described. The two lineaments crossing the dam in an upstream-downstream direction (NW – SE) and other geological features have been excavated and treated with filters.

5.6.6 Excavation Stability

Most of the excavation that is being carried out in the plinth excavation slopes on the upper left abutment and right abutment has been shallow. The slopes observed show a certainty of stability. Localized weathered joints sub parallel to the slope and weathered flow contacts and sub-vertical joints have been recorded in the upper portions of the boreholes on both the right and left abutments. Slickensided clay lined planes and micro fractures and sheared zones have been observed and recorded in test pits in the slumps on the lower left abutment. Some instability of the upper weathered portions of the slopes is thus possible.

5.6.7 Plinth Grouting

The foundation under the plinth has been treated with conventional cement grouting except for the weathered zones within the lineaments. Consolidation and curtain grouting have been carried out under the plinth. Curtain grouting has been carried out to a depth of 50% of the hydraulic head with a minimum depth of 30 m. In addition, the lineaments under the plinth have been specially treated to prevent potential erosion of the weathered materials within the lineaments.

The most prominent joint set orientations are aligned E – W, NE-SW and NNW-SSE of which the first two are the most prominent. The grout holes have been drilled in an optimum orientation to intersect as many joints as possible. The orientation and inclination of grout holes are presented in Table 5.6-5. Where changes in direction occurred, overlapping grout holes have been drilled to obtain a continuous grout curtain.

Table 5.6-5: Proposed Hole Orientation for Grout Curtain (Source: MCG, 2000)

LOCATION	DIRECTION (°) #	INCLINATION (°)*
Left Abutment	335	15
River	155	15
Right Abutment	155	15

#Direction measured clockwise from true north

*Inclination measured from the vertical

5.7 Spillway

5.7.1 Geology of the Spillway

The spillway in general has a very shallow soil cover with the only significant occurrence found in a zone of terraced maize fields near the spillway crest and in thicker colluvial and alluvial soils downstream from the flip bucket. The soils in the spillway crest and upper chute area are predominantly mixed colluvial and residual soils ranging from less than 0.5 m to 1.5 m, depending on the location of the test pit relative to the terraces. In the central section of the spillway chute, the soil thickness is generally less than 1 m with many outcrops of rock. In the flip bucket area reached by colluvial and residual soil, a variable thickness is variable ranging from less than 1 m associated with basalt ridges to 5 m in a localized alluvium filled river channel is reached. In general, the upper rock along the alignment of the spillway is moderately to slightly weathered with weathered joints extending to depths ranging from 3 to 5 m. A localized zone of weathering associated with a vertically aligned shear zone is found in borehole LSR 204.

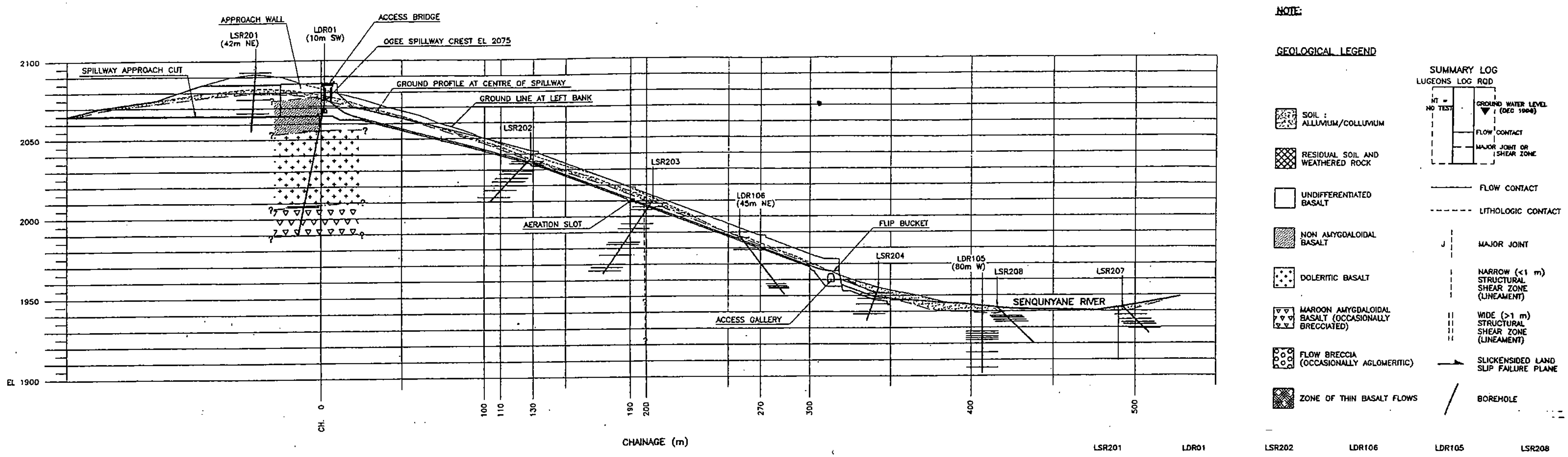
The upper portion of the spillway chute passes through a zone of non-amygdaloidal basalt associated with the thick body of doleritic basalt found in the upper plinth area on the left abutment. This zone shows typical penetrative weathering with the development of spheroidal corestones and boulders surrounded by soft silty clay. The spillway alignment crosses the southeastern boundary of the non-amygdaloidal basalt where a zone of maroon flow breccia is found. The basalt in the central and lower part of the spillway consists of mixed basalt flows ranging from 2.5 to 18 m thick.

5.7.2 Spillway Excavation and Foundation Treatment

The spillway layout has an approximately 30 m high maximum cut over a short section of the approach canal (fig. 5.7-1). The rest of the spillway has shallow cuts ranging from 6 m high near the dam crest to less than 2 m near the flip bucket. The geotechnical factors, which influenced the construction of the spillway, have been the rock mass conditions at the upper cut slope and foundation conditions on the lower spillway slope, as discussed below.

- The lineaments have been interpreted to intersect the spillway, cross it at angles of 80° to 90°. The rocks in these features are closely jointed with deeper weathering confined to the immediate vicinity of the joints and affected the founding condition of the spillway.
- The spillway approach cut area has minimal soil cover with scattered outcrops of basalt rock. The soil cover as it has been interpreted from the boreholes, trench, test pits and seismic survey varies from 1 to 2 m thick. The quality of the upper basalt varies from completely to moderately weathered with fresh rock that has been located at a depth of 1.2 m in borehole LSR 201 and at 4.1 m in LSR 01. The intermediate seismic (velocity layer of 750 – 1500 m/s) is said to be representative of the upper more jointed basalt. It has been found to vary in depth from 3.0 to 6.5 m. Based on borehole LSR 201, the weathered joints occur to depths of at least 8 m. The excavation face, is thus predominantly in basalt rock with weathered joints in the upper zone.
- A zone of non-amygdaloidal basalt with less than 10% ddc has been encountered in the lower portions of the spillway cut and ogee area. Durability of the exposed rock is depended upon the varying ddc content of the non-amygdaloidal basalt.
- Stress relief joints and weathering have been found in the lower slopes of the spillway chute, as noted in borehole LDR 201, LDR 106 and LDR 202. The inclination of these joints varies but generally dips in the direction of the natural slope. These joints together with the vertical joints may control the final cut slope limits. The joints and the surface weathering may affect the excavation conditions in the flip bucket area. The shallow cut, however, can preclude the necessity for any major support.

- The ground contours are aligned more or less perpendicular to the spillway and thus the depth of the cut slopes are low. The interpreted structural lineaments also intersect the spillway perpendicularly and thus the length of the cut slope where these features may affect the stability has been minimal. Allowance for local stabilization of these cuts with rock bolts and/or shotcrete has been provided. As the rock conditions are generally competent, the expected extent of cut slope stabilization treatment has been minimal.



LONGITUDINAL SECTION THROUGH SPILLWAY

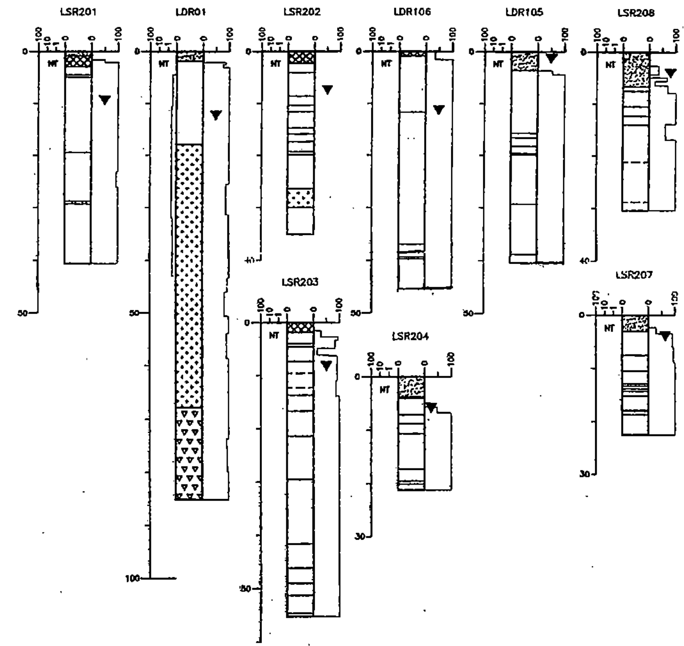


Figure 5.7-1: Spillway Longitudinal Section

SUMMARY LOGS

5.8 Diversion Tunnels, Portals and Intake

5.8.1 General

Two diversion tunnels have been constructed through the left abutment under a maximum rock cover of about 125 m. Both tunnels have a common inlet portal excavation, where after they diverge towards two separate outlet portals located on either side of the spillway flip bucket on the left abutment, downstream of the dam. The outlet of Tunnel 2 is localized downstream from the spillway, while the outlet of Tunnel 1 is located upstream.

5.8.2 Geology of the Tunnels, Portal and Intake

5.8.2.1 Inlet Portals

The inlet portals are established on the left bank of the Senqunyane River at the base of a 40° to 50° sloping cliff face covered by thin patchy colluvial scree deposits. The colluvium consists of angular gravel and boulders with clay and sand and is locally up to 1.5 m thick.

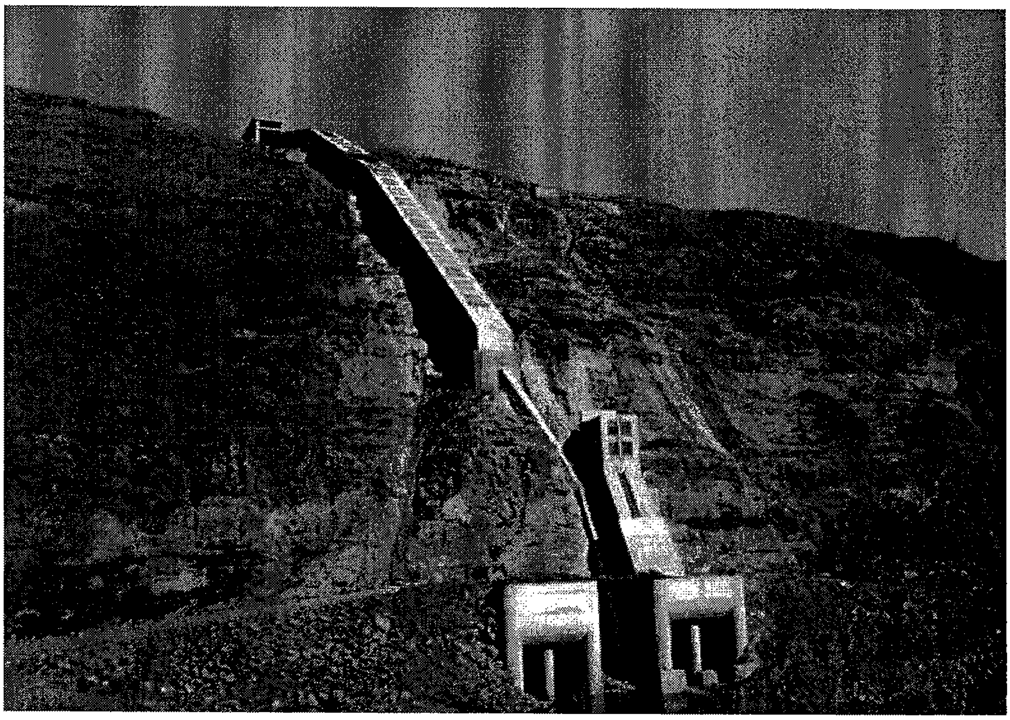


Figure 5.8-1: Compensation Structure and Diversion Tunnels

Figure 5.8-1 shows compensation structure and tunnel intake structures. The basalt is also nicely exposed with very typical thin soil cover in places. The base of the cliff is within a zone of relatively thick basalt flows, interspersed by closely spaced thin flows. The portals are located within and close to the base of one of the thicker flows. The lower contact of this flow outcrops at EL 1944.5, i.e. approximately 1 m below the portal invert for Tunnel 1 at EL 1945.5 and 3 m below the portal invert for Tunnel 2 at EL 1947.5. The portals are predominantly within massive non-amygdaloidal basalt.

A 300 mm wide zone of closely spaced zeolite filled joints crosses the tunnel alignment about 10 to 20 m beyond the tunnel portal. This zone constitutes the tunnel excavations. It has appeared to have no influence upon the stability of the portal. Stress relief joints associated with the steep topography and cliff faces occur on the lower valley sides. These joints display usually orientation parallel to the valley sides and dip steeply towards the valley, intersect the portal excavation and appear to affect the stability of the slope hence support has been required for installation (fig.5.8-2; 5.8-3; 5.8-4; 5.8-5; 5.8-6; 5.8-7; 5.8-8 and 5.8-9). The different flows are clear exposed. Fig. 5.8-4 shows a pocket of HAB while fig. 5.8-3 shows a blocky ground. As it can be seen in fig. 5.8-3 to 5.8-9, that the rock is sound and weathering is minimal. No other major structural features have been encountered in the portal area.

Figure 5.8-2: Slope Stability. Reinforcement next to Compensation Structure

Figure 5.8-3: Slope Stability. Reinforcement at Compensation Foundation

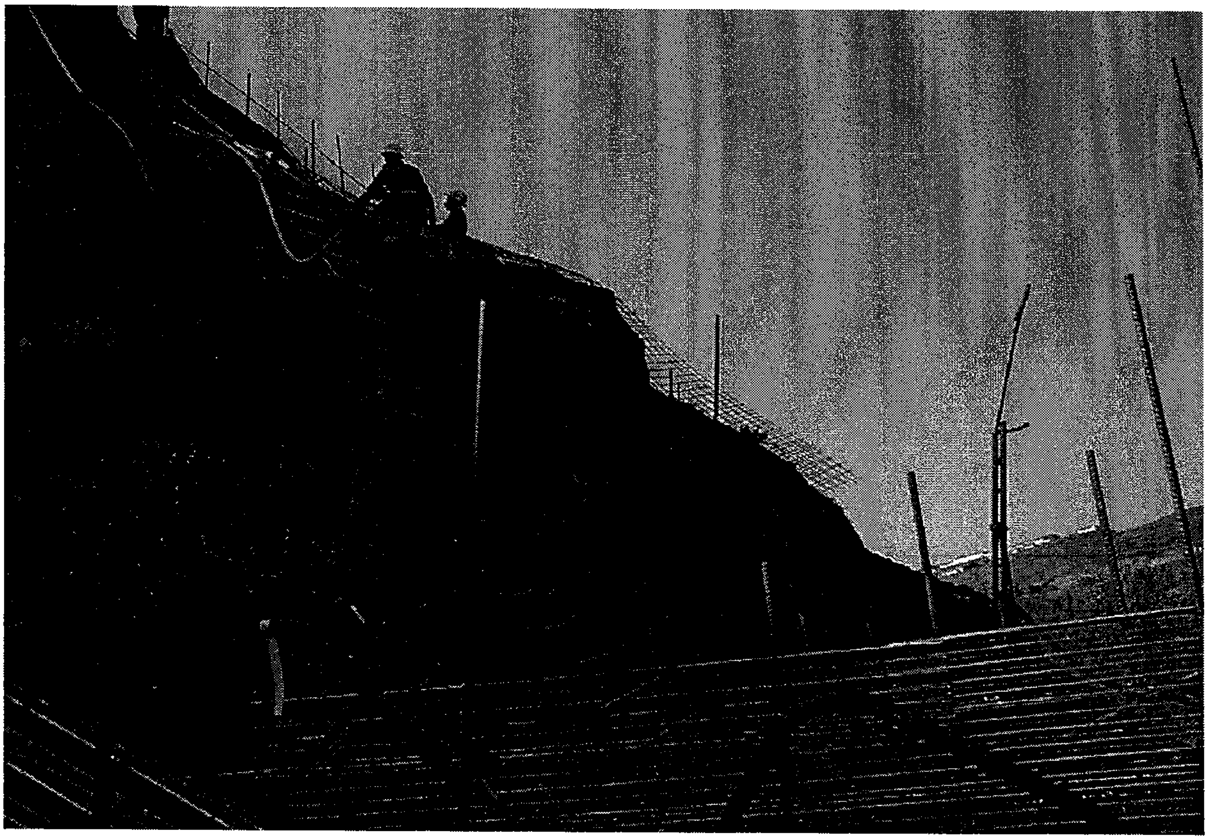


Figure 5.8-4: Slope Stability. Work at Compensation Foundation

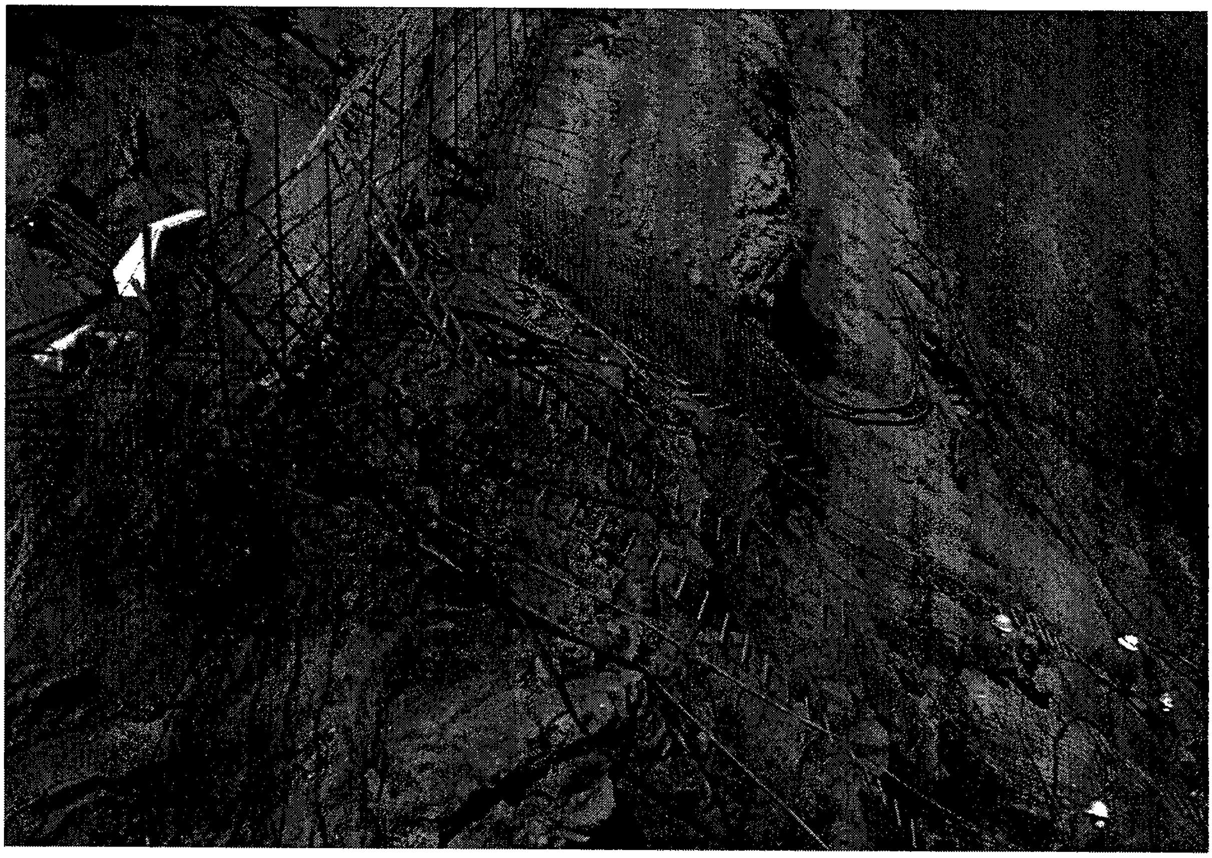


Figure 5.8-5: Slope Stability. Work downstream from the Compensation Foundation

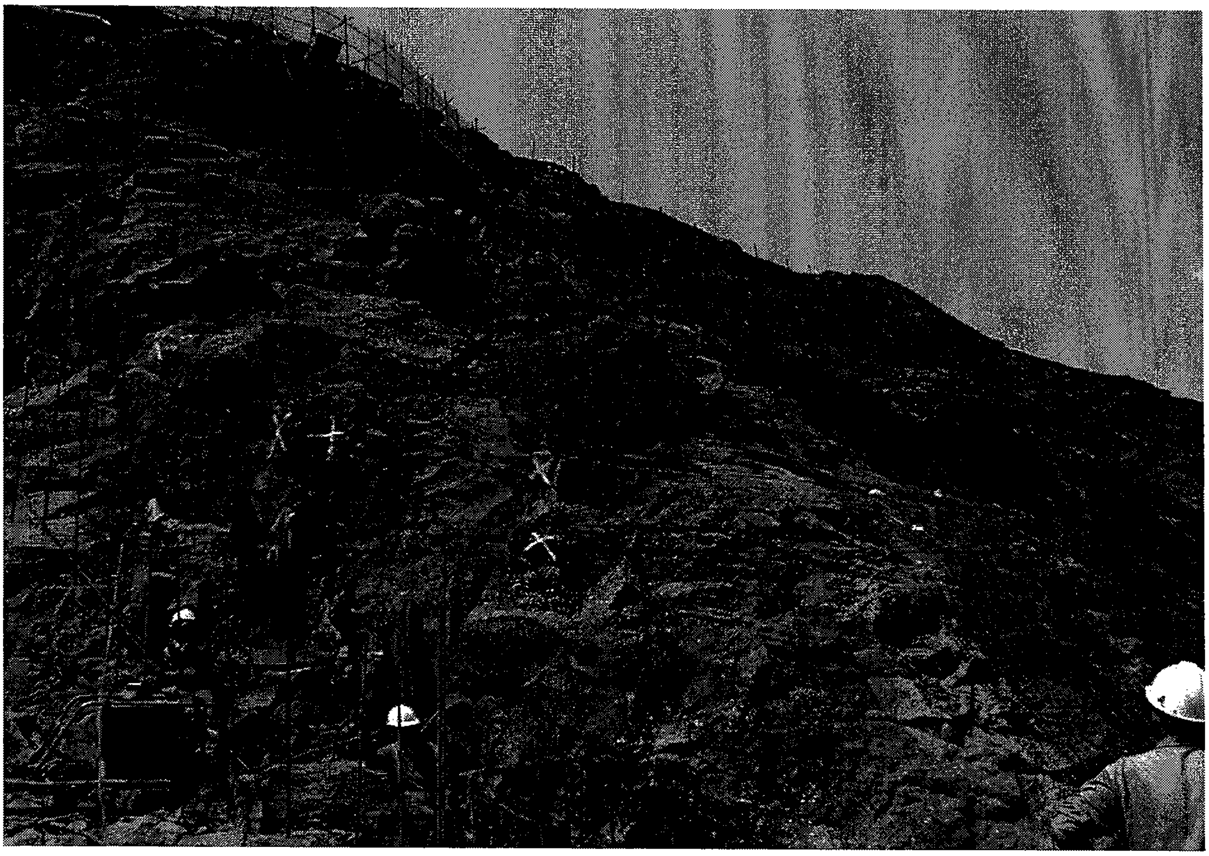


Figure 5.8-6: Slope Stability. Work at Compensation Foundation

Figure 5.8-7: Compensation Foundation Reinforcement

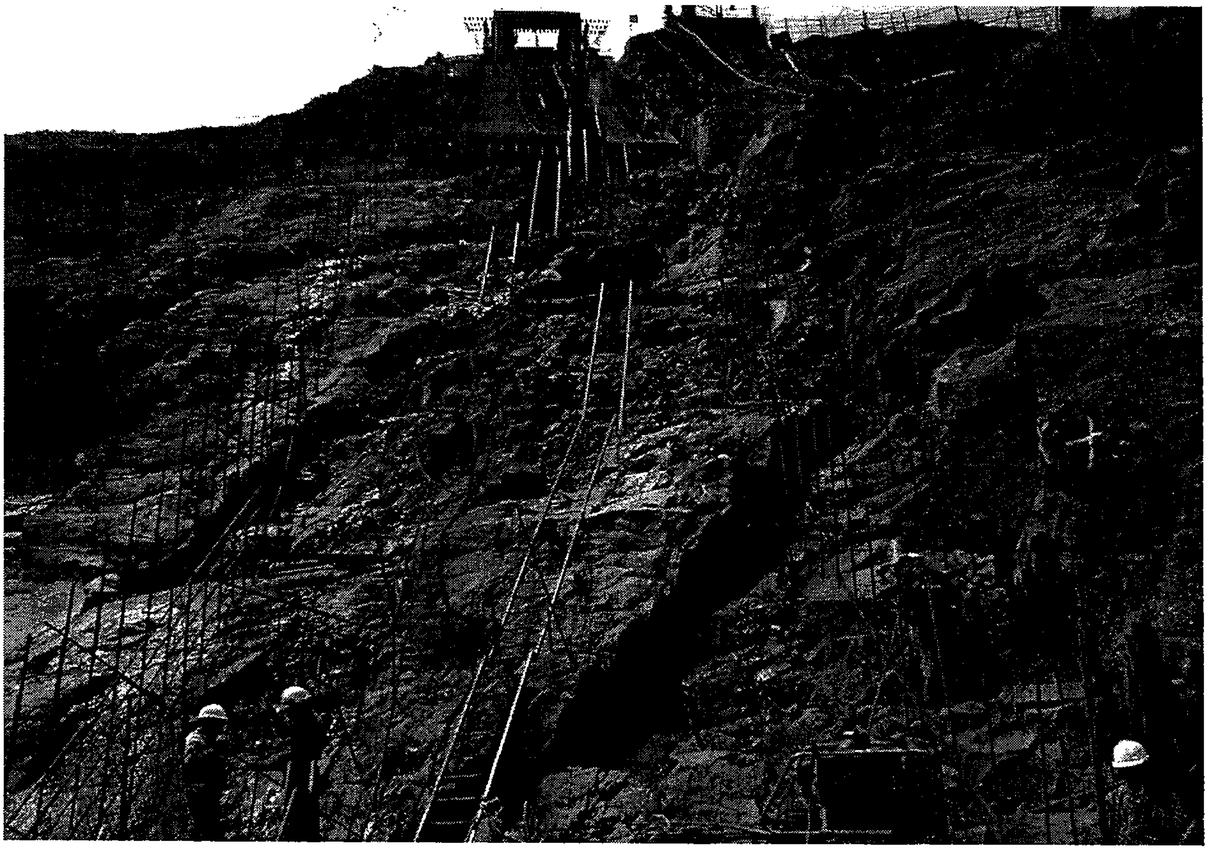


Figure 5.8-8: Compensation Foundation Reinforcement

Figure 5.8-9: Laying Wire-Mesh. Work along the Slope next to Compensation

5.8.2.2 Intake

The foundation for the intake structure consists of mixed amygdaloidal and non-amygdaloidal basalt that often display localized surface weathering. The rock foundation conditions in the intake area are generally observed to be tight with local jointed zones. The geology is nicely exposed on the face walls (fig. 5.8-10).

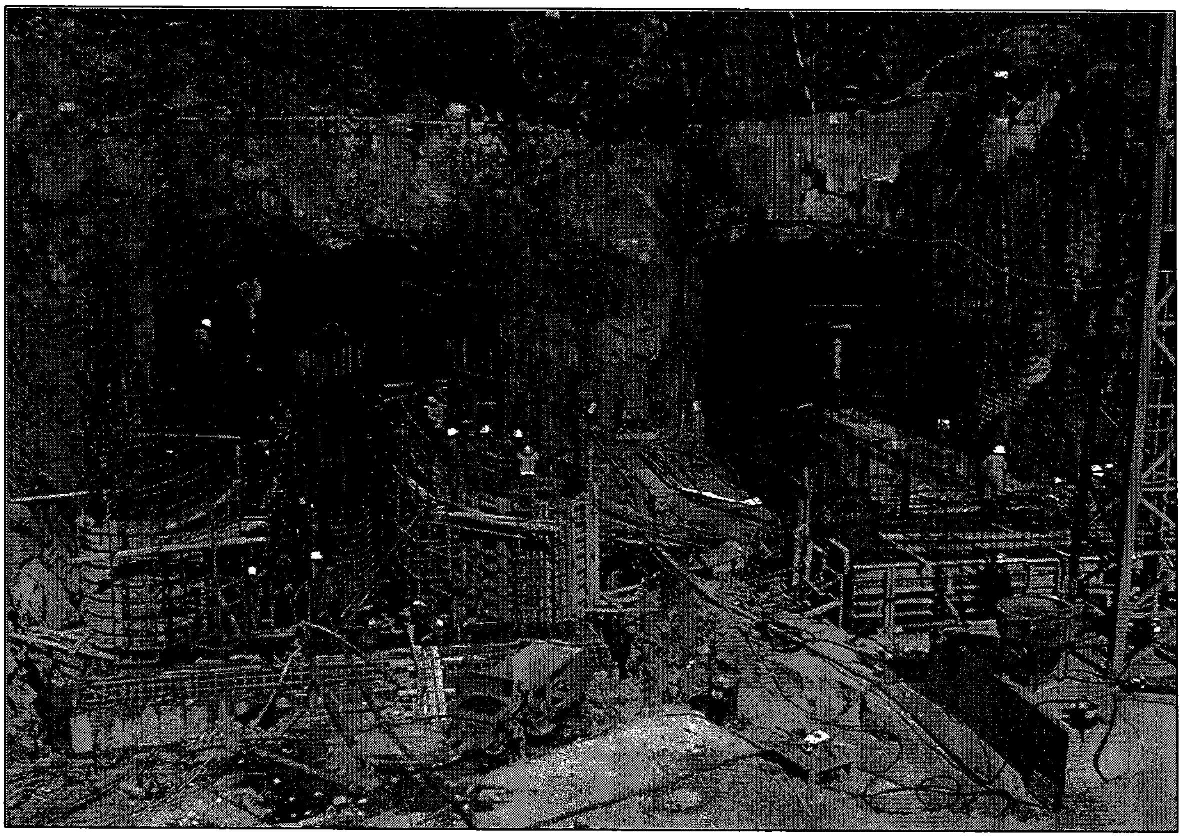


Figure 5.8-10: Concrete Works. Work at Diversion Tunnel Intakes.

5.8.2.3 Outlet Portals

The outlet portals have been located within the sloping ground of the lower left bank of the Senqunyane River, downstream from the dam (fig. 5.8-11). The slope is covered by colluvial and residual soils of variable thicknesses. The occurrence of these soil deposits, as well as penetrative weathering of the rock along joints, required the excavation of relatively long and deep approach cuts so that the portals and the initiation of the tunneling could be located in suitable rock conditions.

The invert of the outlet portal of Tunnel 1 is at approximately EL 1944. This resulted in an excavation depth of about 17 m for the portal. The thickness of colluvial soil varied along the approach cut and ranged from 1 m at the portal to >5 m in test pit LSP 112, the downslope of the portal. Strong seepage has been observed in places. A number of slumps in the colluvial soil have been observed to occur in this area.

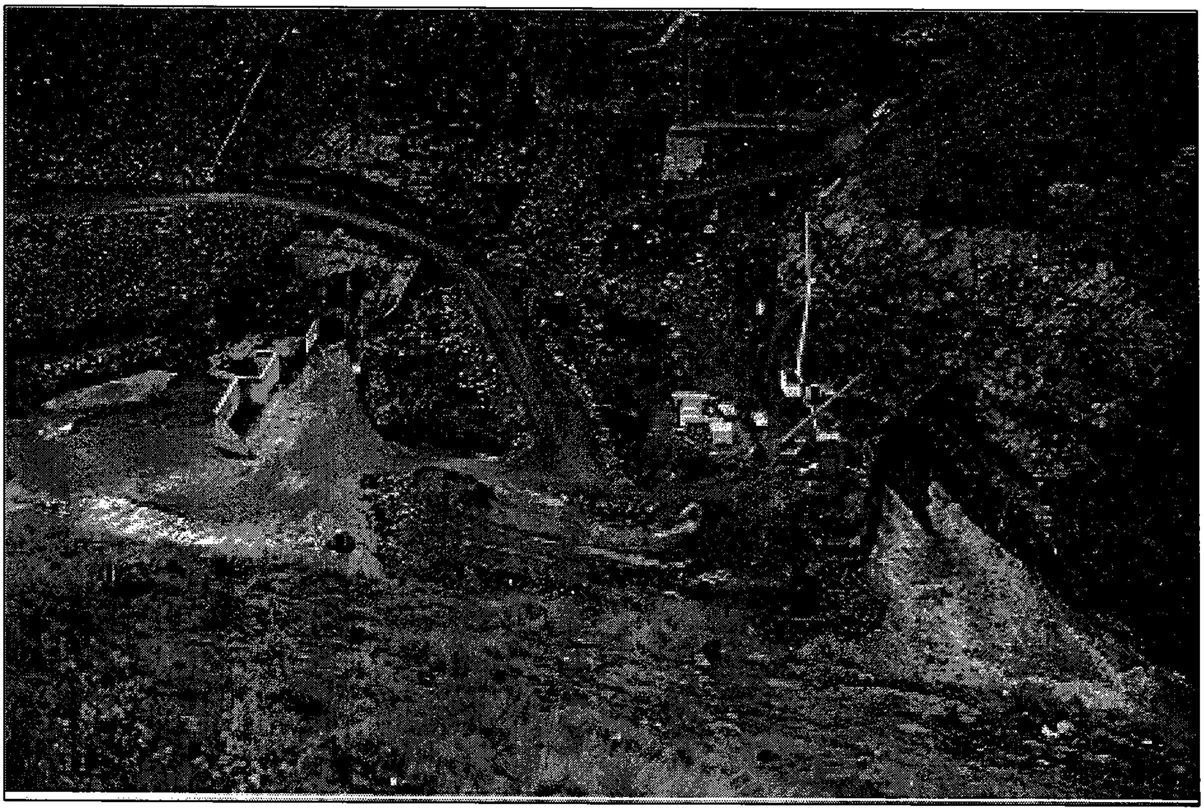


Figure 5.8-11: River Exit. (Tunnel 1 and 2 and Outlet Portals (21/12/99))

The subsurface profile of the outlet to Tunnel 2 is similar to that of Tunnel 1, although the soil and depth of weathering are thicker (fig. 5.8-11). The outlet is located in an area of slumping, with slickensided shear planes evident within, and at the base, of these soils. Colluvial slumped soils have been observed in borehole LSR 206 to a depth of approximately 5 m. The maximum depth of the cut at the outlet portal is about 25 m. The soil cut slopes requires stabilization or alternatively removal or flattening to achieve stability. Since the soils are erodible, they have been protected against surface runoff and seepage flows.

The underlying rock in the area of the outlet portals generally consists of a sequence of relatively thin basalt flows. Deep penetrative weathering of these basalts has been observed, in particular along flow contacts and joint planes, resulting in a variable degree of spheroidal weathering and the development of clay gouge along discontinuities. This phenomenon is clearly observable in fig. 5.8-3, which is very characteristic of basalt. It has been this characteristic of the basalt that duly posed ingress water problem in the LHWP tunnel driving when in association with a dolerite dyke. This is expected to have resulted in blocky ground, which becomes progressively intact and less weathered with depth.

Three sets of vertical to sub-vertical joints intersect the rock mass. The majorities of inclined joints are observed to be dipping steeply out of the slope and interpreted to be stress relief joints, are also present. These joints are variably weathered and often have thin black clay filling extending to a depth of about 19 m. The clay filling in some of the joints has been observed in the rock cores to be slickensided. They are expected to have an adverse influence on the stability of the excavation, in particular on the steep cut face of the portal area, which definitely require support. A narrow zone of sub-vertical joints crosses the portal face of Tunnel 2 (fig. 5.8-12). There is penetrative weathering along this zone resulting in friable rock conditions.

5.8.2.4 Tunnels

Figure 5.8-12: Intake Structure for Tunnels 1 and 2.

Tunnel 1 and Tunnel 2 follow horizontally bedded basalt lava flows over most of their alignment. In the central zone of the tunnel, a zone of very strong non-amygdaloidal basalt and doleritic basalt associated with the thick basalt flow bodies, which has been observed on the left abutment of the dam, has also been encountered. The lower of these thick bodies which occurs at the same elevation as the diversion tunnels, has a thick central core of doleritic basalt where it outcrops at the plinth in the river area and which has been extrapolated to extend below the left abutment to the central sections of the diversion tunnels. Zones of mixed basalt and flow breccia characterize locally the boundaries of these bodies. On either side of these thicker bodies and extending to both the inlet and outlet portals, a zone of mixed thin and thicker basalt flows is anticipated.

The geological conditions along the tunnel routes are based on the rock exposures in the river area, the boreholes drilled for the dam and mapping in the exposed rock outcrops above the tunnels and. Three broad zones can be interpreted:

- A zone of thicker basalt flows with interspersed thinner flows from the inlet portal to the central zone where the rock is generally unweathered massive non-amygdaloidal basalt;
- A central zone consisting of very strong to very strong closely jointed doleritic basalt varying to massive non-amygdaloidal basalt which is unweathered and is locally bounded by a zone of mixed basalt and flow breccia which is weaker and degrades upon exposure; and
- A zone of generally thin basalt flows between the central body and the outlet portals where the rock becomes progressively more weathered closer to the outlet portals.

Three lineaments cross the Tunnel 1 and Tunnel 2 alignment. Additional zones of closely jointed rock have been encountered such as a zone at the Tunnel 2 outlet. These lineaments are typically zones of closely spaced joints, with zeolite infilling (fig. 5.4-2). Their anticipated locations are indicated and are summarized in Table 5.8-1.

Table 5.8-1: Summary of Anticipated Structural Lineaments in the Tunnels (Source: MCG, 2000)

LOCATION (Approximate chainage)		APPROX STRIKE (°)	APPROX. ANGLE TO TUNNELS (°)	DESCRIPTION
Tunnel 1	Tunnel 2			
CH 42	CH 25	005	60	Veined closely jointed shear zone with zeolite filling
CH 370	CH 400	070	80	Possibly associated with zone of close jointing found in borehole LSR 202
CH 460	CH 480	050	85	Lineament, very closely spaced jointed zone with weathered joints near surface found in borehole LSR 203
-	Outlet Portal area	Varies	Varies	Zone of closely spaced joints found in borehole LSR 206. Possible stress relief joints.

The larger part of the tunnel penetrates through variably jointed intact basalt rock. The survey of the joint orientations in the dam area has indicated that the dominant joint set will be in a NE-SW direction, which is crossing the tunnel at right angles and thus providing favourable excavation conditions. The two other sets identified, which is the E – W and NNW – SSE direction, cross the tunnel route obliquely. The depth of weathering along these joints and flow contacts near the outlet portals are responsible for weak clay filling of joints and resultant blocky ground. The mixed zone of basalts and flow breccia on either side of the doleritic basalt seem to be weaker and less durable. Flow contacts are generally strong in the unweathered rock, although they form planes of preferential parting during blasting. Tuff has been observed on flow contacts at approximate EL 1956 in borehole LDR 106, i.e. approximately 5 m to 6 m above the proposed tunnel crown. Flow contacts, especially those weakened by weathering and filled with tuff are potential weakness planes in the tunnel crown area and there presence require support. These planes of weakness in the crown of the tunnel also cause increased over-break and flat tunnel crown profiles over extended lengths of tunnel.

The rock core from boreholes and mapping of the rock indicated that the basaltic rock is generally tight and treatable by normal cement grouting. Water acceptance testing in the plinth and dam area during the various investigations has shown a consistent low water intake on the upper left abutment. Some secondary permeability resulting from joints in the doleritic basalt may result in water seepage and inflows. The groundwater levels in boreholes drilled above the tunnel are variable ranging from a depth of 51 m in boreholes LDR 101 to 5.3 m in LSR 202. Perched water levels are present in fracture zones.

5.8.3 Excavation and Support

Tunnel 1 is concrete lined over the whole of its length while Tunnel 2 has a concrete invert but is otherwise unlined. The downstream section of Tunnel 2 is used as a permanent access. The following potentially adverse rock conditions have been recognised.

- The potential occurrence of thin flows and thus horizontal flow contacts above the tunnel at the inlet and outlet portal area as well as the adjacent tunnel sections, which can easily lead to slabs forming in the crown and haunch sections.
- The possibility of wedges forming on the wall and haunches associated with the E – W and NNW – SSE joint directions.
- The possible formation of wedges in the tunnel crown, haunches and walls because of bladed widely to closely spaced jointing in the doleritic basalt sections of the thick basalt flow body in the central portion of the tunnel.
- The possibility of potentially weaker mixed flow breccia rock zones.
- The localized occurrence of very close to closely jointed zeolite or calcite filled shear zones crossing the tunnel at variable angles and typically resulting in overbreak in the crown and haunches and the formation of release planes for joint bound wedges and slabs in the crown.

The presence of close jointing and dipping clay filled joints in the portal area necessitated full support of the portal face and initial sections of the tunnel.

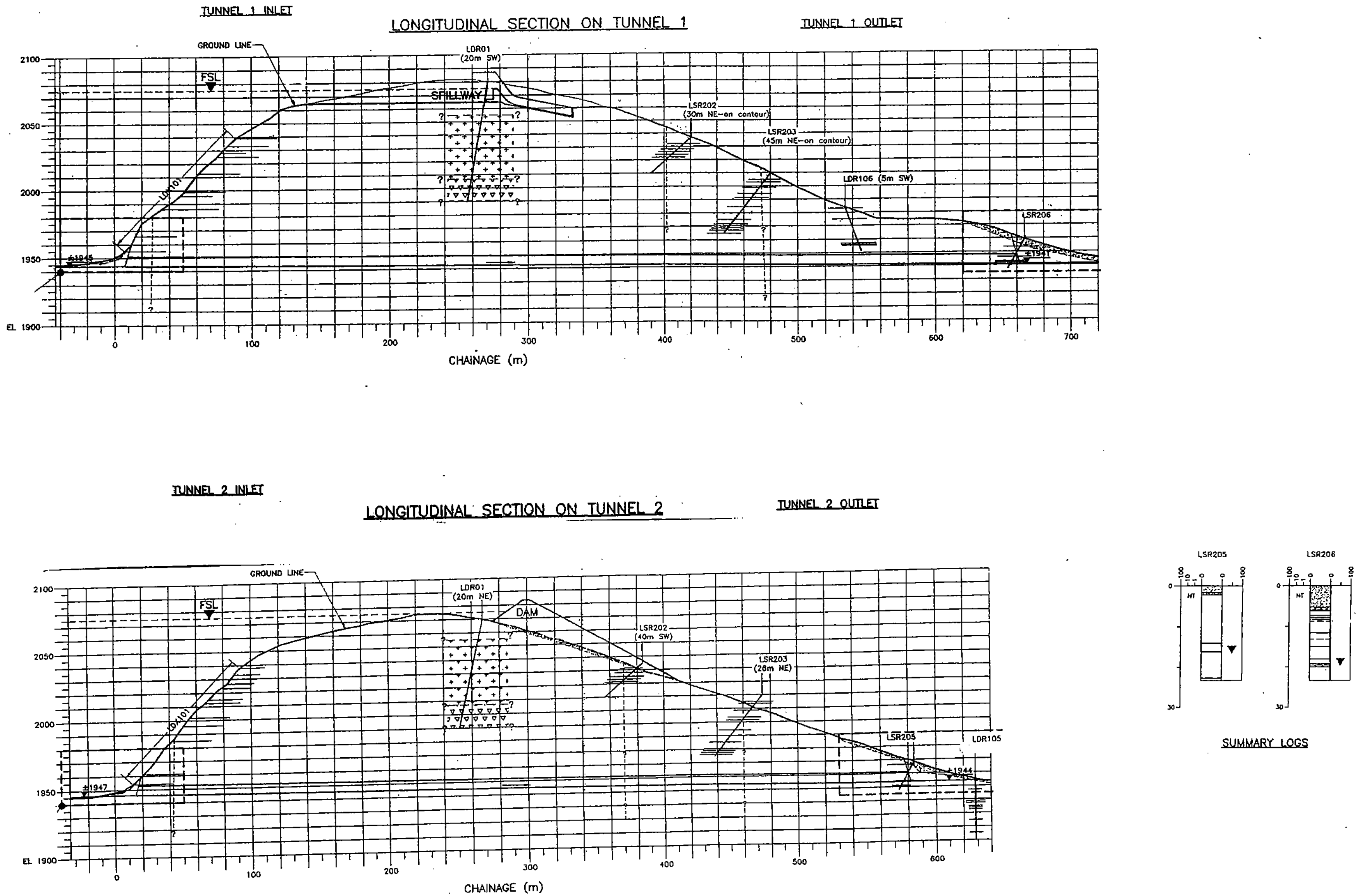


Figure 5.8-13: Longitudinal Section on Tunnel 1

5.9 Upstream and Downstream Cofferdams

5.9.1 Excavation for Upstream Cofferdam Shell

The axis of the upstream cofferdam is located approximately 60 m upstream from the tow of the Mohale Dam toe at the confluence of the Likalaneng River and Senqunyane River. The cofferdam alignment extends from the left abutment to the river level at EL 1945 and traverses the right bank, crossing a narrow gully from EL 1955 to EL 1965. Scattered outcrops of basalt occur on the left bank of the Senqunyane River. They can be traced beneath the thin alluvial deposits on the left bank of the river. Thin colluvial soils occurring on the higher left bank slopes and scattered alluvial sand is found in the river area. A thin (up to 2 m) alluvial outwash deposit is formed at the Likalaneng River confluence on the right bank. An alluvial terrace both upstream and downstream from the confluence is interpreted to vary from 3 m thick at the upstream part to 1 m in the downstream section. A rock bound slope underlies the right bank of the cofferdam where outcrops of doleritic basalt, flow breccia and non-amygdaloidal basalt occurs.

Table 5.9.1: Support Classifications in Diversion Tunnels

CLASSIFICATION	ROCK CONDITIONS	PRIMARY SUPPORT
1	Massive rock, unweathered to slightly weathered, discontinuities, moderately to widely spaced, clean, non-weathering.	Rockbolts: Spot rock bolts. Shotcrete and mesh: Localised, if required.
2	Jointed rock, and/or moderately weathered rock with discontinuities moderately to very closely spaced - < 20% of the wall and crown area	Rock bolts: Pattern over up to approximately 50% of tunnel walls above spring line – elsewhere spot bolts as required. Shotcrete and mesh: 50 mm thick over <50% of the area above spring line, elsewhere as required.
3	Jointed rock and/or moderately too highly weathered with discontinuities moderately to very closely space over 20 – 50% of the wall and crown area.	Rockbolts: Pattern over approximately 100% of the exposed tunnel walls above spring line – elsewhere-spot bolts as required. Shotcrete and mesh: 50 mm thick over 50 – 100% of the area above spring line, elsewhere as required.
4	Jointed rock and/or moderately to highly weathered with discontinuities close to very closely spaced over 50-100% of the wall and crown area.	Rockbolts: Pattern – longer bolts over 100% of the exposed tunnel walls to spring line – elsewhere spot bolts as required. Shotcrete and mesh: 50 – 100 m thick above and below the spring line. Steel Sets: if required.

The excavation for the cofferdam shell has been extended to the base of the colluvial soil. The upper alluvial terrace has been removed but the underlying closely packed boulders could be left in place.

5.9.2 Excavation and Foundation Treatment for Upstream Cofferdam Core

Under the semi-impervious core contact the excavation for the upstream cofferdam has been extended to at least moderately weathered rock which is anticipated at depths of up to 3 m on the left abutment slope and up to 2 m on the right bank of the river below the alluvial deposit. On the right bank slope the excavation depth is expected at between 0.5 to 1.5 m.

5.9.3 Downstream Cofferdam

The downstream cofferdam is about 8 m high and is incorporated into the toe of the Mohale Dam and the excavation is therefore similar to that for the Mohale Dam rockfill Zone 3E. The depth of overburden in this area is about 1.5 to 2 m.

5.10 Slopes in Soil

The soils in the project area are colluvial or residual soils. The soils have been derived from the weathering of basalt. Less than 2 m high cuts in these soils, under dry conditions, are generally stable at a cut slope of 1H: 1V. For the Mohale Dam Project, considering the shallow soil slopes and the nature of the overburden (fig.5.10-1), all excavation in soil is being cut at a slope of 1.5H: 1V with a 2 m minimum width bench at the interface of soil and rock.

Prior to the rainy season all permanent slopes must be:

- Stabilized by installation of gabions, and the area between gabions and topsoil be filled in with granular material.
- The granular fill must be topped with mortar to form a non-erodible drainage surface for runoff.
- Any disturbed topsoil cover other than the slopes stabilized with granular material and gabions must be stabilized by hydroseeding or grassing with sod well in advance of the forthcoming rainy season.

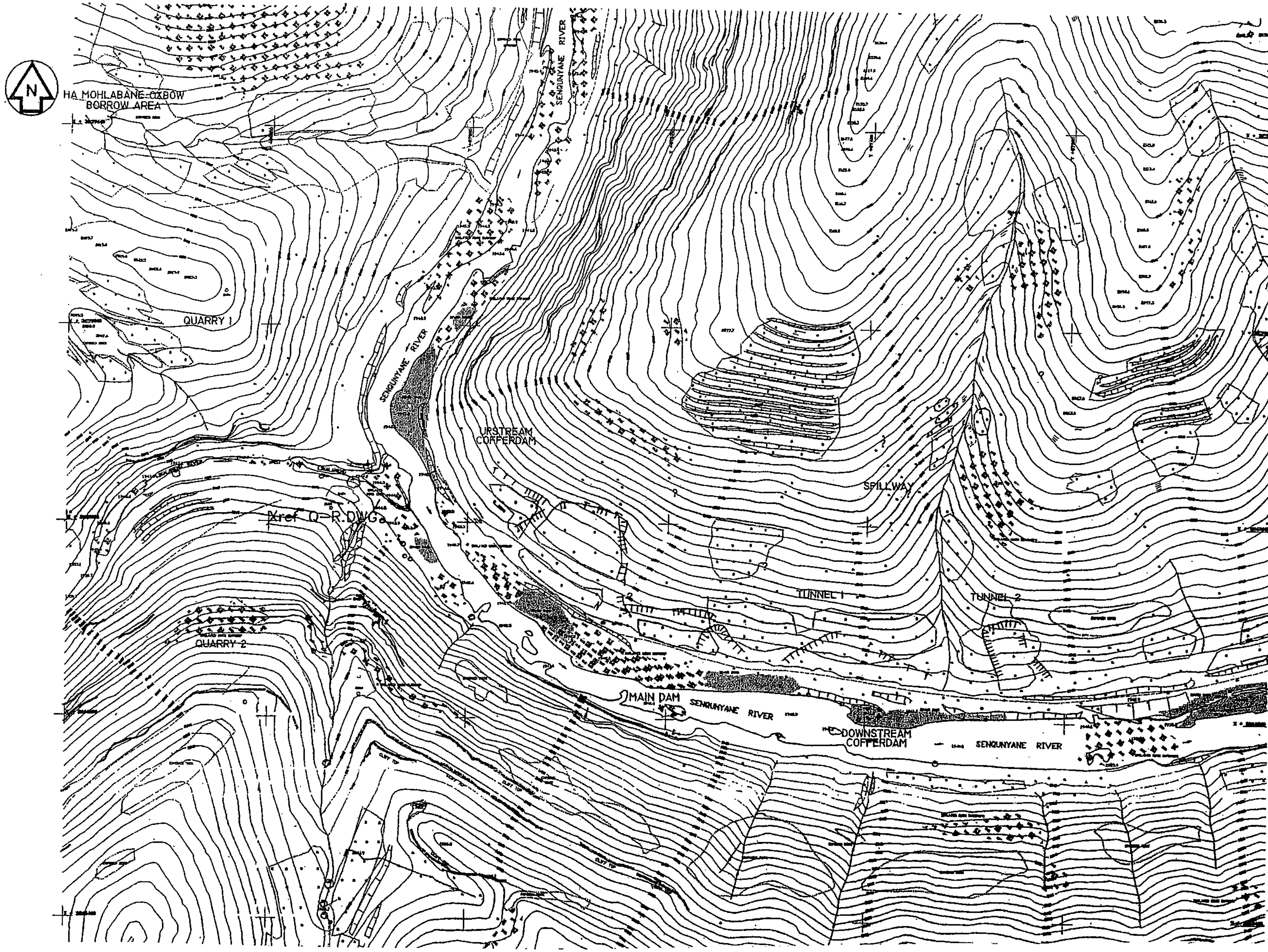


Figure 5.10-1: Dam Soils and Overburden

5.11 Slopes in Rock

In basaltic rock, which generally is characterized by a set of vertical and horizontal joints, it is therefore desirable to cut steep slopes. To secure overall slope stability, 3 m wide benches at vertical intervals of 10 m must be provided to allow later access for the performance of maintenance work. The presence of inclined stress relief joints has been observed in the boreholes where they are often associated with weathered weak rock surfaces or clay. The stress relief joints together with the identified sub-vertical joint sets can lead to unstable slopes or wedges. Allowance has been provided for temporary stabilization measures in all rock cut slopes.

All cut slopes in rock require a minimum slope of:

- 1H: 4V with a 3 m wide bench every 10 m vertical.

Cut slopes in the upper weathered rock and where stress relief joints are present the rock require the minimum slope of:

- 1H: 2V

Individual cut slopes have been adopted to suit local geological conditions, rock mass jointing and weathering profile. Allowance for support of permanent slopes has been provided where the jointing conditions warrant it. Cut slopes have been stabilized with rock bolts, shotcrete, wire mesh and drainage. On each bench, a drainage canal has been provided.

5.12 Quarry and Borrow Areas

5.12.1 Geology of Quarry 1

Quarry 1 is located on the left bank of the Likalaneng River and on the right bank of the Senqunyane River extending down to their confluence 100 m upstream of the dam site. Fig. 5.4-2 shows a dolerite dyke crossing the quarry. The quarry site covers an area of approximately 11 ha and includes part of the Ha Mohlabane Oxbow and the entire ridge between it and the Likalaneng River. The slopes of the area around the oxbow vary from 15% to 20% while the slopes adjoining the Likalaneng River are steeper (40% to 70%) with many near-vertical cliff faces. The oxbow is filled by colluvial soils ranging in depth up to 7 m and consist generally of gravelly and sandy clay becoming black, saturated and organic in the lower lying areas of the oxbow. Thicker (in excess of 1.5 m) colluvial soils are found on the north east and south east facing slopes of the ridge while thinner colluvium and scree have developed on the steeper slopes above the Likalaneng River.

Two boreholes have been drilled in the quarry during the Feasibility Study, i.e. LQR 01 on the left bank of the Likalaneng River near the Senqunyane River confluence and LQR 02 on the west side of the Centre ridge of the Mohlabane oxbow. A more detailed exploration of the quarry area has been carried out during the Tender Design with an additional 10 holes drilled covering the entire quarry area.

The Quarry 1 consists of three broad zones.

- An upper zone of thicker flows consisting predominantly of non-amygdaloidal basalt and thinner moderately amygdaloidal basalt zones. This zone extends from the crest of the main ridge at about EL 2090 to approximately EL 2033. A very thick flow of non-amygdaloidal basalt with variable ddc contents occurs between approximately ELS 2060 down to approximately EL 2033. The central core of this flow showed some doleritic basalt characteristics in the outcrop on the southwest slope above the Likalaneng River but this zone could not be correlated across the quarry.
- A zone of thin flows occurs below approximately EL 2033 and extends down to approximately EL 1980. This zone can be traced throughout the Quarry 1 area although individual flows are difficult to correlate. The flow thickness generally ranges from less than 1 m interflows to 5 m with some thicker flows of approximately 11 m in LQR 209 and LQR 211 at the northwestern side of the quarry area. The basalt rock varies from non-amygdaloidal to moderately and highly amygdaloidal basalt.
- A zone of thick flows between 25 and 15 m thick occurs below EL 1980 and extends down to the Senqunyane river level. These flows are characterized by large zones of non-amygdaloidal basalt, which typically have a high percentage of ddc. The lower flow has been used as a source of road material for the current temporary causeway built across the river. A zone of micro-amygdaloidal basalt at EL 1965 to EL 1975 identified during the Feasibility Study was confirmed in borehole LQR 212 but this variation in the basalt could not be extrapolated throughout the quarry. A zone of disturbed moderately to highly amygdaloidal basalt with agglomeritic features has been observed below EL 1975 on the left

bank of the Likalaneng River in geological traverse. This zone is not extensive and can only be traced over a length of about 100 m along the left bank.

All the material encountered in the quarry is suitable for use as general Rockfill Zones 3B and 3C of the Mohale Dam and Upstream Cofferdam.

5.12.2 Geology of Quarry 2

The Quarry 2 is located on the right bank of the valley formed by the Likalaneng River, upstream of the confluence with the Senqunyane River, fig. 5.5-1. The quarry is developed to mine the very strong doleritic basalt, which outcrops above approximate EL 1990 to 2000 and extends to between EL 2035 and EL 2048. The thick doleritic basalt layer from this quarry is the nominated source for durable rock to produce concrete aggregates, filter, transition and good quality rockfill for use in slope protection and drainage layers. The doleritic basalt forms the lower part of the thick body of very strong non-amygdaloidal and doleritic basalt occurring between approximately EL 1990 and EL 2070 and has been correlated with the similar doleritic and non-amygdaloidal basalt on the left abutment of the Mohale Dam.

Surface exposure of the doleritic basalt consists of a jointed rock cliff face approximately 35 m high extending from west of the dam site for approximately 800 m to the southwest, ending at the Ha Piti oxbow. The end of the distinctive bladed jointed outcrop demarcates the extent of the doleritic basalt towards the southwest while to the east it pinches out some 60 m west of the right abutment of the Mohale Dam.

The lower boundary of the doleritic basalt material grades in places into a thin non-amygdaloidal basalt zone, which forms the base of the body which in turn overlies a distinctive maroon amygdaloidal basalt. The lower flow contact has typical flow characteristics such as tube amygdales as well as tight welded contacts, which have been identified as intrusive. The maroon amygdaloidal basalt layer forms a distinctive marker at the base of the proposed quarry. It can be traced along the entire outcrop along the Likalaneng River. The lower boundary of the doleritic basalt is gradational and has been taken as the point where visible ddc is distinctly observable in the core.

The upper boundary of the doleritic basalt is gradational. It is variable and tends to grade into very strong non-amygdaloidal basalt. This extent of the gradational boundary varies with the amount of ddc, the presence or absence of cooling joints and variation from holocrystalline to hypocrystalline structure. This gradational zone is variable across the quarry with approximate EL 2025 in the western end of the quarry to EL 2030 in the eastern end.

The upper boundary of the flow body is formed by a flow contact at EL 2070 and forms the crest of a distinctive cliff extending along the whole of the exposure on the right bank of the Likalaneng River. The doleritic basalt in Quarry 2 is thus interpreted to be the lower part of a very thick flow body ranging from 72 m thick in borehole LQR 03 to 32 m in borehole LQR 103. The overall shape of the flow can be regarded as lensoid, pinching out within the mountain ridge to the southeast and removed by the formation of the Likalaneng River valley to the northwest. It cannot be traced on the opposite (left) bank of the Likalaneng River. The boundary in the southwest appears to be gradational and is represented by the end of the distinctive bladed jointed outcrop above the Ha Piti oxbow is expected to pinch out to the south west of borehole LQR 205.

A summary of the interpreted boundaries of the doleritic basalt in boreholes and geological traverses is presented in Table 5.12-1.

Four E – W trending fracture zones intersect the quarry, all of which are continuations of the veined lineaments mapped on the right abutment of the dam site. Two of these are located near the top of the ridge above the quarry (borehole LQR 103), and pass through the southwest end of the quarry. The smaller of the features was intersected by borehole LQR where a number of prominent sub-vertical joints were intersected. A fracture zone was intersected by borehole LQR 205 which consisted joints with calcite and green (possibly chlorite) mineral filling over an approximately 2.5 m wide zone. Two veined fractures pass through the doleritic basalt cliff face and traverse the east end of the quarry. Boreholes LQR 03 and LQR 203 intersected some of these features where extensive alteration was noted in the form of clay/calcite/zeolite veining with green chloritic minerals. These altered zones vary in width between approximately 0.5 to 3 m.

Table 5.12-1: Approximate Elevation of Boundaries of the Doleritic Basalt in Quarry 2 (Source: MCG)

BOREHOLE	LOCATION	ELEVATION OF UPPER BOUNDARY	ELEVATION OF LOWER BOUNDARY
Feasibility Study LQR 03	Central Area	2042*	1997
Planning Study			
LQR 101	N – E side	DB not found	DB not found
LQR 102	N – E side	2037#	2007**
LQR 103	N – E side	2049	2023
LQR 104	Central Area	2037	2011
LQR 105	Central Area	2033*	2002*
Tender Design			
LQR 201	N – E side	2031	2004
LQR 202	N – E side	2044	2000
LQR 203	N – E side	2044	2001
LQR 205	S – W side	2036	1991
LQR 206	S – W side	2030#	1994
LQR 216	S – W side	2031	1997

* Boundary elevation re-interpreted during Tender Design.

** End of Borehole

\# Borehole commenced in doleritic basalt (DB).

A characteristic jointing pattern is developed within the doleritic basalt. Jointing is typically vertical to sub-vertical comprising a bladed to almost columnar joint pattern. On the surface, etching along prominent joints and healed, discontinuous joints are ubiquitous. The borehole core indicates that there are closely spaced joints within the doleritic basalt, which are generally planar, clay and/or calcite/zeolite filled or coated (mainly less than 2 mm). Occasional joint infill may be as much as 20 mm thick.

5.12.3 Sources of Earthfill Material

A number of sources of earthfill material for use in Zone 1A, Impervious Earthfill or Zone 1B, Random Earthfill have been investigated. Three types of deposit have been recognized.

Table 5.12-2: Types of Sources of Earthfill

	Type 1	Type 2	Type 3
Size of deposit	Relatively small &discontinuous	Relative larger isolated bodies of	Various thicknesses ±16 m
Occurrence	Along the River courses &lower slopes of valleys	At Ha Tsiu to the NE of the dam site	In Oxbows Slopes as residual and colluvial
Origin	Mixed alluvial found mainly on the inside of bends or south facing slopes e.g. around the dam site, especially on the left abutment	Residual with some colluvial soil & minor alluvial deposits, e.g. plateau deposits found at Ha Tsiu to the NE of the dam site and along the Likalaneng valley slopes	Mixed origin soils found in the oxbows
Geotechnical Investigation	During the Tender Design	During the Planning and the Feasibility Studies respectively	During the Feasibility and Planning Studies and confirmed during the Tender Design

The Tender Design evaluation has concentrated on the source closest to the project area. These are the Ha Mohlabane and Ha Piti oxbows situated on the northern edge of Quarry 1 and the northwestern edge of Quarry 2, respectively, and the material available from the stripping of the dam and quarries fig. 5.12-1, 5. A summary of the sources of earthfill investigated is presented in Table 5.12-2 while a summary of types of sources of earthfill in Table 5.12-3.

Table 5.12-3: Summary of Sources of Earthfill Material for the Main Dam and Cofferdam

SOURCE	TYPE	INVESTIGATION
Ha Mohlabane Oxbow Borrow Area (Likalaneng Area 1)	3	Feasibility Study and Tender Design
Ha Piti Oxbow Borrow Area (Likalaneng Area 2)	3	Feasibility and Planning Studies, Tender Design
Likalaneng Area 3	2	Feasibility Study
Likalaneng Area 4	3	Feasibility Study
Ha Tsapane Borrow Area	3	Feasibility and Planning Studies
Ha Tsiu	2	Planning Study
Dam Plinth – Left Abutment	1	Tender Design

The bulk of the soil and overburden stripped for the construction of the Mohale Dam is found within the left abutment and the river area. The Tender Design testing programme concentrated on these soils. The alluvial soils in the river consist predominantly of alluvial boulders while the alluvial terrace soils consist of mixed alluvial gravel, localized scree, and will be unsuitable for impermeable fill although it can be used in the outer zone. The thickest soils are found in the lower slopes associated with the slumps and the testing of these soils has indicated they are suitable as impermeable fill. The black organic topsoil should not be used in any earth fill and must be stripped from these deposits.

Generally, a thin topsoil layer composed of dark gray silty to very silty clay covers the oxbow areas. Thicknesses of this material range from 0 m to 1.7 m in the Ha Mohlabane oxbow and from 0.1 m to 1.4 m in the Ha Piti oxbow. The high organic content of the material renders it unsuitable for use as earthfill.

The colluvial horizon is underlain by residual soil derived from the basalt. The residual soil comprises generally yellowish brown and orange brown silty clays, often almost indistinguishable at times from the overlying colluvial material. The residual material ranges in thickness from 0 m and 1.3 m in the Ha Mohlabane oxbow and from 0.7 m to 3.1 m in the Ha Piti oxbow. Two Boreholes drilled in the Ha Mohlabane oxbow show a thickness of unconsolidated and weathered material of 14.4 m and 16.2 m respectively. These soils are suitable for use as impermeable fill.

The occurrence of groundwater within the oxbows increases towards the valley floor. On the slopes, no seepage was encountered in the test pits, but in some of those pits excavated on the lower slopes and on the valley, floor groundwater seepage was encountered. However, these depths of occurrence do not necessarily indicate the true groundwater levels within the soil profile.

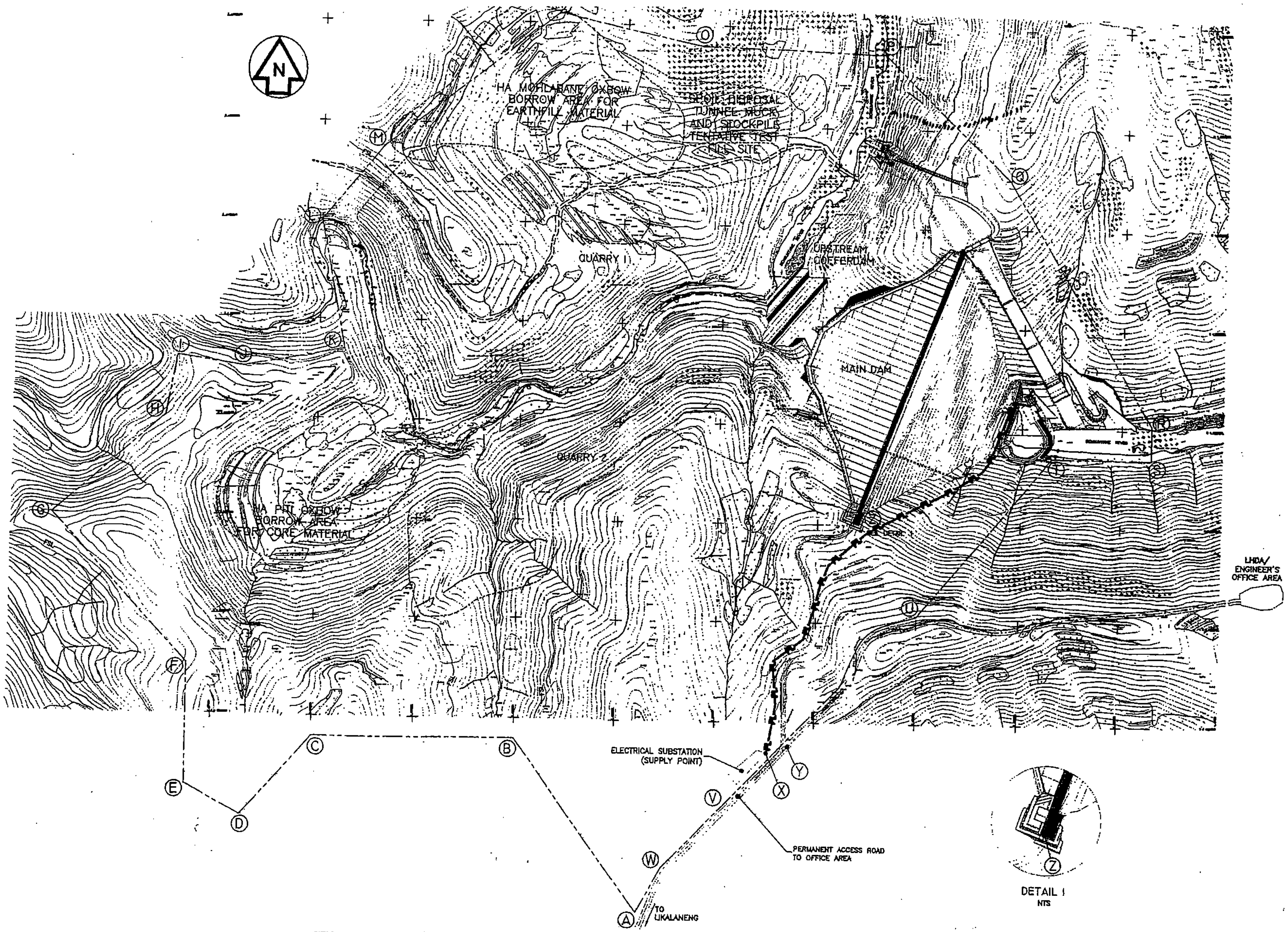


Figure 5.12-1: Dam, Quarries, Barrow Pits and Engineer's Office

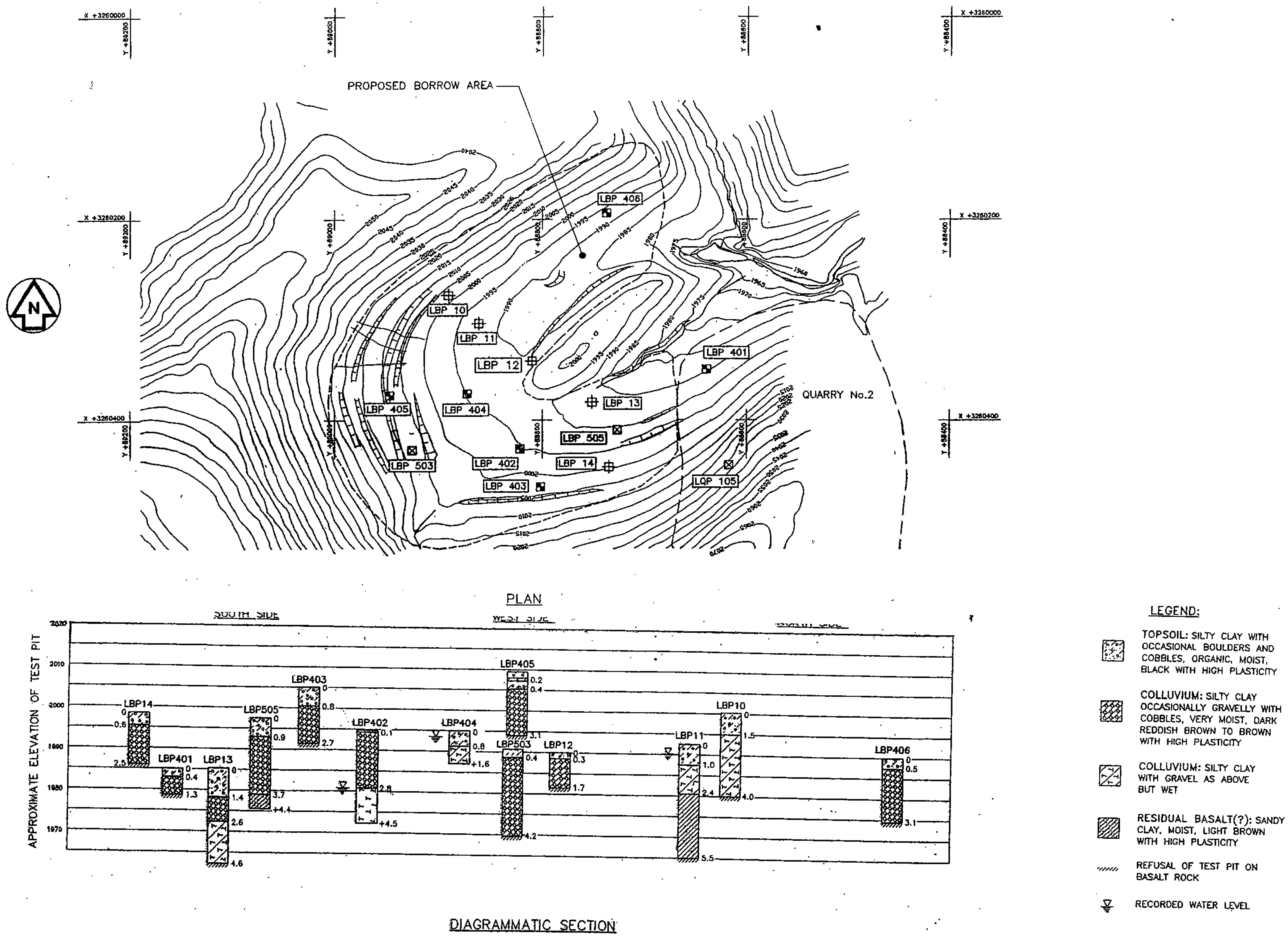


Figure 5.12-2:Test Pits Excavated. Work during Feasibility and Planning Studies

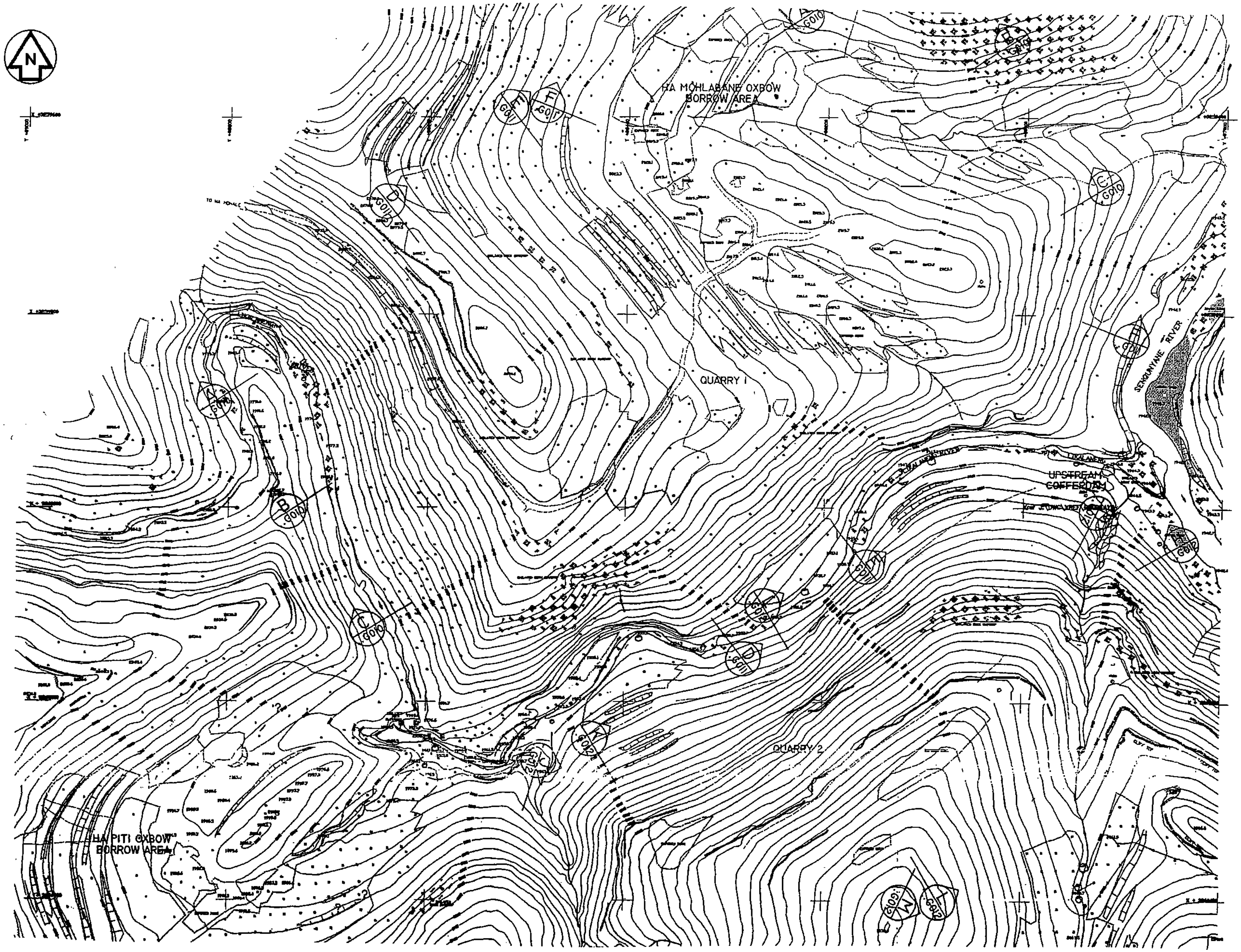


Figure 5.12-3: Quarry and Borrow Areas: Soils and Overburden

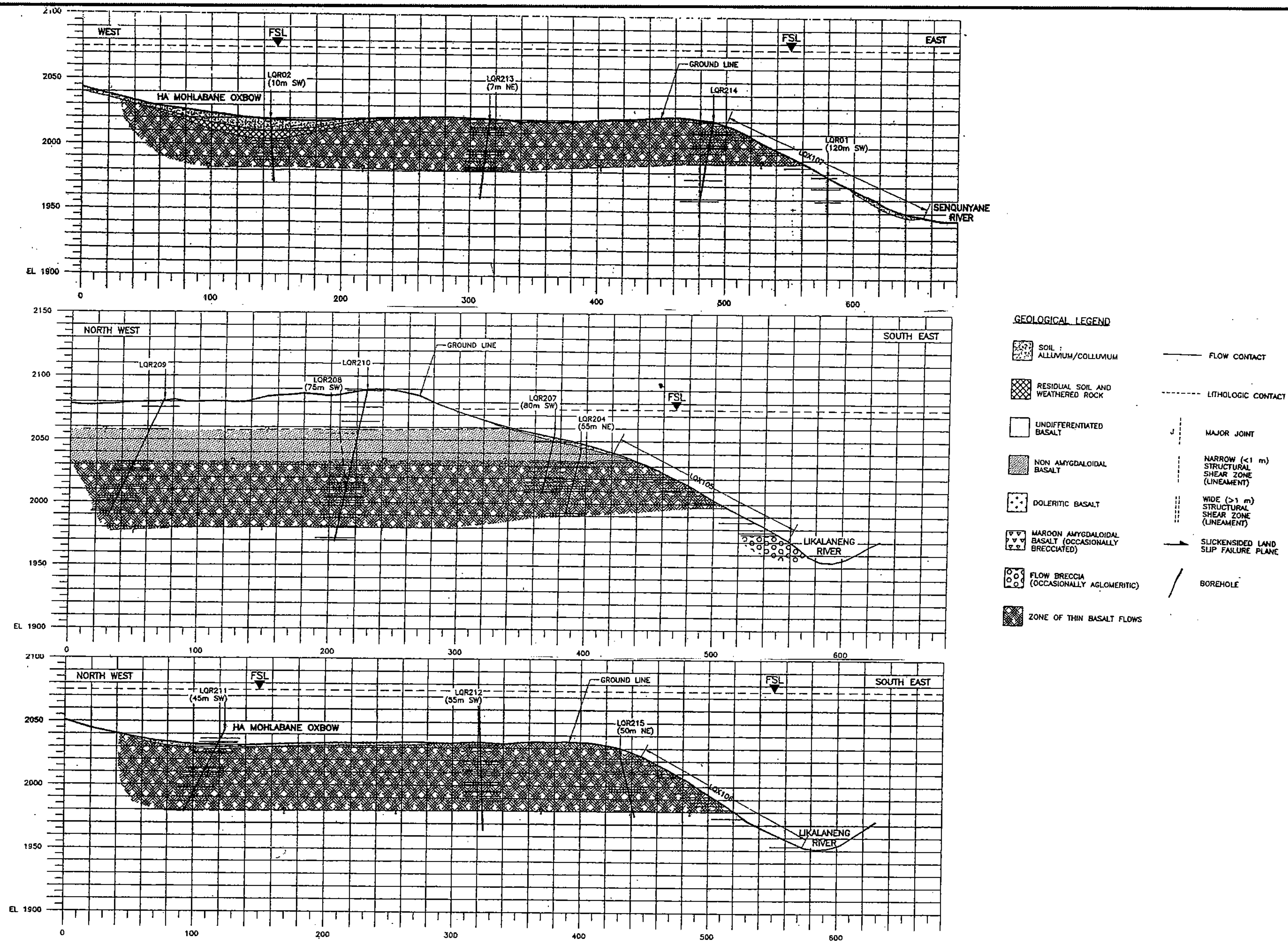


Figure 5.12-4: Quarry 1Geological Sections. (northwest – southeast and west – east)

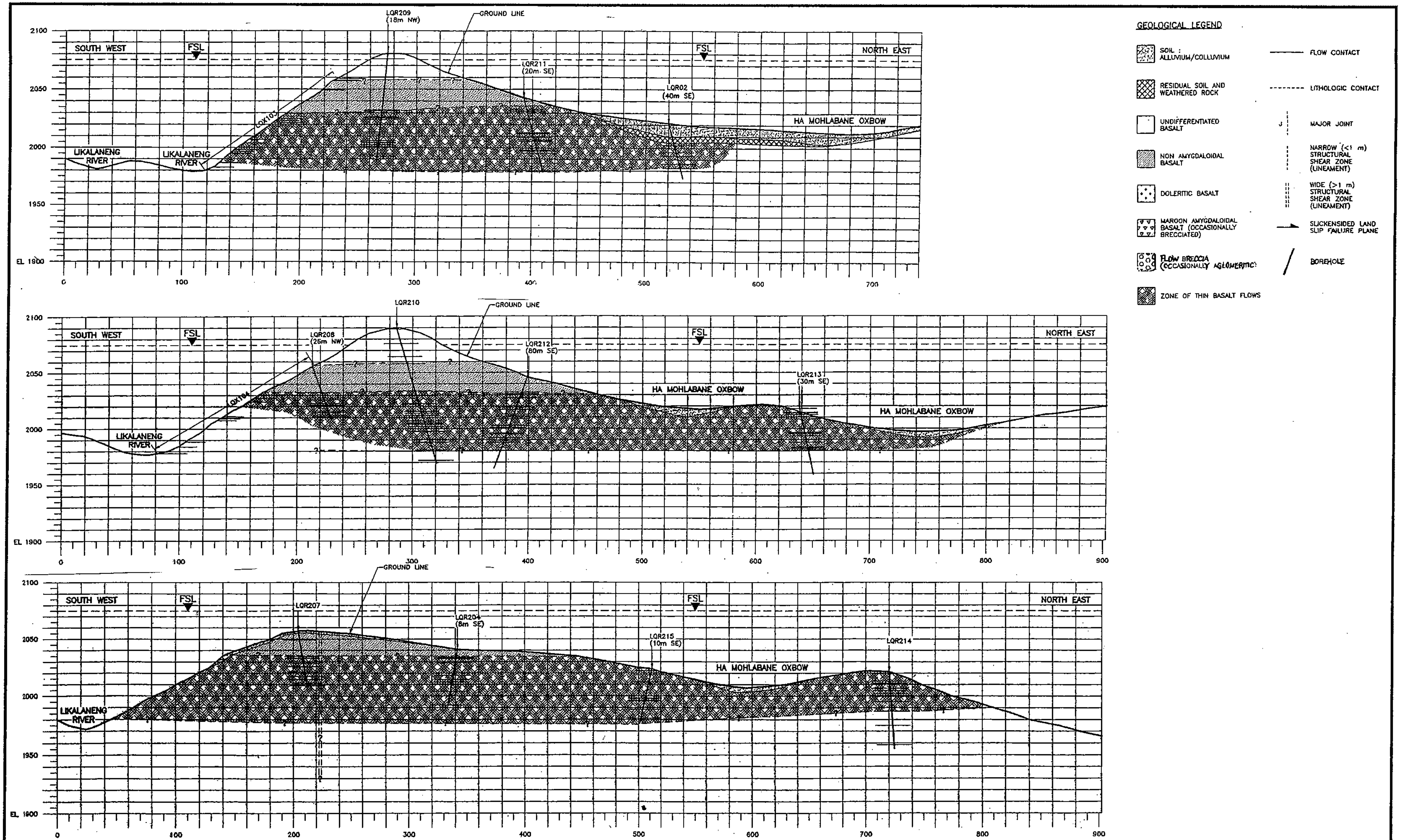


Figure 5.12-5: Quarry 1 Geological Sections. (Southwest – northeast)

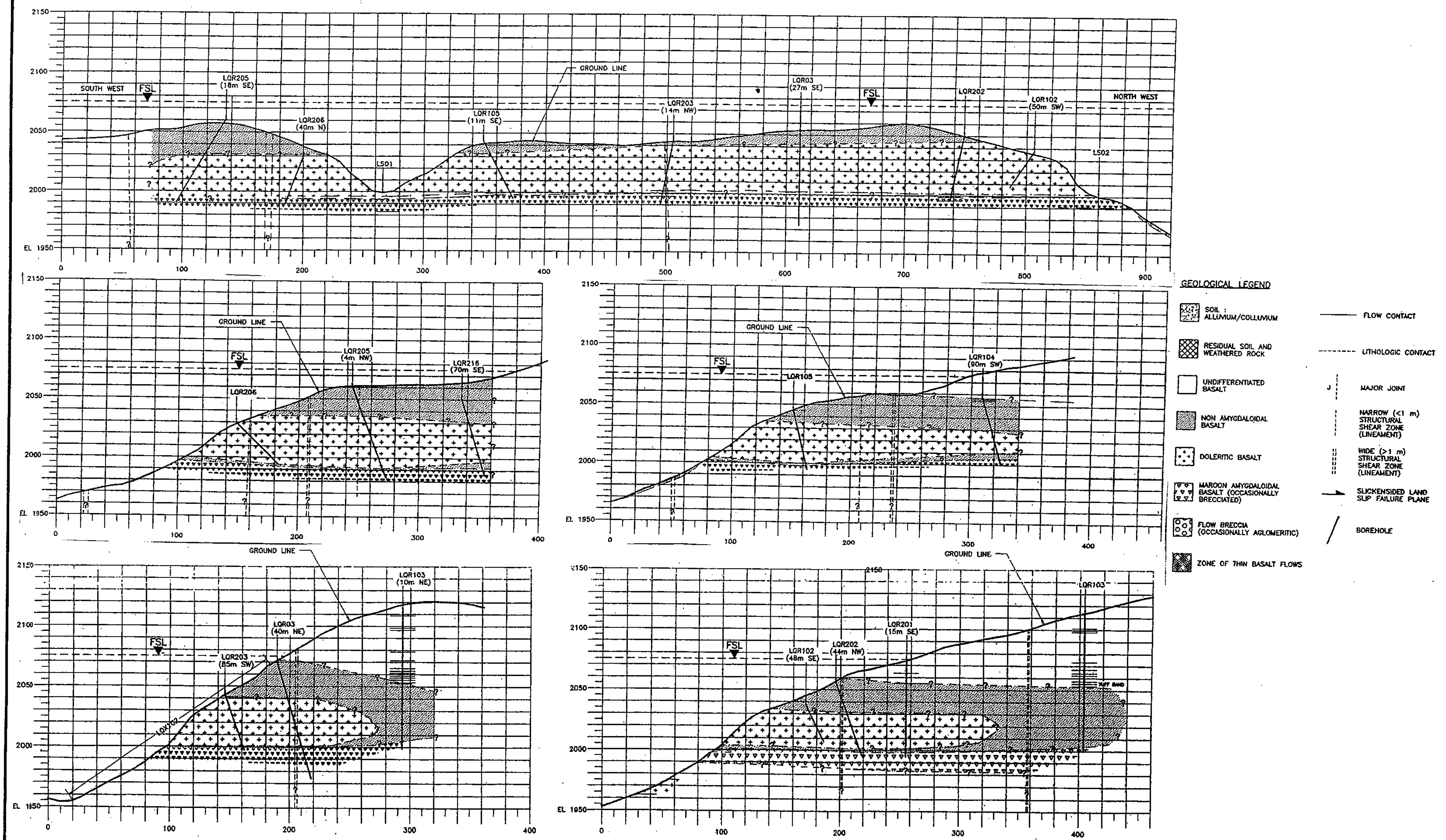


Figure 5.12-6: Quarry 2 Geological Section. (Southwest – northwest)

5.13 Geological and Environmental Conditions

The geological conditions of reservoir, tunnel and any other construction site are of fundamental importance in assessing the environmental impacts.

The geological conditions of the site can be outlined as follows:

- Sufficient geological data must be provided in order to assess and predict environmental conditions and impacts respectively
- Geological conditions of the site must be assessed prior to construction, during and after

Leggert, (1987) stressed the importance of geology in planning physical facilities and individual structures and in the most judicious and best use of land. He pointedly noted the obvious, that land is the expression of underlying geology so that land –use planning can only be done satisfactorily if there is a proper understanding of the geology concerned. Leggert, (1987), further states that the development of land must be with the full realization of the natural forces which have brought it to its present state, taking into account the dynamic character of nature so that development does not upset the delicate balance any more than is essential. AGI Environmental Geoscience's Advisory Committee, (1995), also shares this point of view. Geology must therefore be the starting point of planning.

In many countries, laws and regulations are in place, which enforce the investigation and evaluation of all engineering works. An environmental impact statement therefore usually involves a description of the proposed scheme and its impacts on the environment. Leopold emphasis's the inevitable involvement of a geologist in environmental impact studies.

The engineering of structures, in particular dams and tunnels, is based on detailed measurement and interpretation of the behaviour of the soils and rocks liable to be affected by construction works, hence posing an environmental danger to the area and its habitat. In this context, probably the most significant features of soils and rocks are their capacity for bearing loads without excessive settlement. Regarding the Mohale Dam, quantities/calculations are in chapter 7, their volume change characteristics, their excavatability, and their intrinsic stability. The behaviour of rocks and soils is a product of their composition, microfabric and mass structure. In turn, these features are functions of their mode of formation and subsequent history. It is important to appreciate that due to geological processes, especially weathering, the engineering properties of soils and rocks change with time. The geology of the site is very important because the character of the geological material concerned and the environmental conditions influence the nature, extent and rate of occurrence of these changes.

5.14 Reservoir Sites

Among the many considerations involved in the choice of a site for a surface storage reservoir, a number are geological; in nature, these are, both literally and metaphorically, fundamental according to Edmunds, (1960). He defines or describes an ideal site for such a reservoir as a large, elongated, and steep-sided valley that has been eroded out in impermeable rocks and is a place of high precipitation. Such valleys are not only common in the Lesotho Highlands region but are invaluable in the decision making for a site and that is where the LHWP is located.

He goes on to say that many valleys, which superficially may meet requirements, have been developed along deep-seated lines of weakness resulting from geological faults, along which water would escape. Although water may not be lost along it, under natural conditions, the changed conditions produced by filling a valley with water may produce the reversal of the natural underground direction of flow. For instance: the sides and bottom were initially thought to be entirely satisfactory, but a wash of surface material masked a permeable bed up the valley side, rather lower than the proposed ultimate level of the impounded water as controlled by the Dam. On impoundment, water may be accumulated as planned and if there is a permeable bed it may flow through and form springs in adjacent valley or valleys, depending on the geology.

5.14.1 Geohydrology of Filled Reservoirs

Failures of slopes on the sides of reservoirs are especially frequent during drawdown and may not produce unmanageable problems, especially when the reservoir is nearly empty. This phenomenon is common along the shores of the Katse Reservoir, except that the volume of material gravitating into the reservoir is somewhat negligible. Of generally more critical concern are rock and earth slides into a reservoir when it is full, and suddenly destructive overflow of the dam is a consequence as it was the case with the Italian Dam, the Viont Dam in 1965.

The filling of a reservoir causes adjustments in the groundwater table in adjacent materials. When the reservoir is full for an extended period, a groundwater table is established, which at its lowest elevation, coincides with the elevation of the water surface. Seepage from surface recharge tends to build up a free surface below, in which the material is saturated and which slopes towards the reservoir. It can be assumed that the configuration of the surface changes with seasonal fluctuations in recharge rates, so that a dynamic system is created in which discharge velocities and pore pressures below the surface (the groundwater table) also fluctuate in materials that were well drained prior to reservoir filling.

Because the groundwater table is a free surface in contact through unfilled pore spaces with the atmosphere, changes in atmospheric pressure are accompanied by changes in pore pressure in the saturated zone, so that the dynamics of the system are further complicated. An additional factor promoting instability in slopes above the perimeter of a reservoir is wave action, which undercuts slopes and makes them steeper. Any one or a combination of the processes outlined above can create disastrous collapse of the sides of reservoirs involving masses of rock on earth of large to small dimension e.g. Schematic Diagram after Wahlstrom, (1974). Failure of the slopes might occur at any time, that is, during reservoir filling, or during drawdown.

Water can seep through the fractures (faults, joints filled with cal/zeo/clay material) from the reservoir. This may happen for a considerable distance into the rock mass, and by slow penetration of the fractured zone, greatly reduces their strength under these circumstances. A highly unstable condition is created, especially where the sedimentary layers dip into the reservoir. Bell, (1980), considers joints, faults and shear zones to be directly responsible for most unsound rock encountered at dam sites on plutonic and metamorphic rocks. Again, he says that if they are not sealed they may permit leakage through foundations and abutments. According to Bell, (1980) thick massive basalts make satisfactory dam sites but many basalts of a comparatively young geological age are highly permeable, conveying water via their open joints, pipes, cavities, tunnels and contact zones. This is a situation, which does not exist in the LHWP site.

5.14.2 Stability of the Reservoir Slopes

The frequency and orientation of joints, bedding planes and other discontinuities as well as their properties affect the quality or strength of the rock mass, which influences the stability of the rock mass. The occasional presence of water in discontinuities or infilling of weak materials in the discontinuities creates local zones of weakness. The timing of noted failure during the generally wet winter and spring periods, strongly suggests that water infiltrating the joints plays some part in destabilizing the rock mass. There is considered a negligible probability of a slide involving the basalt rock mass, apart from the superficial zone of highly weathered material. The type of movement that could occur would involve the clayey colluvium. The thickness colluvium is on the lower slopes, however, and is unlikely to cause problems.

5.15 Seismicity

5.15.1 Investigation of Seismicity

There is no method at present of forecasting the exact location, size or time of an earthquake. All authors in the seismological field share this view; e.g., Jaeger, (1972), Jaeger, (1979), Bell, 1980; Duncan, (1969), Wahlstrom, 1976; Talebi, (1999); Gupta, (1976), Evans, (1966), Talebi, (1999) Rastogi et.al., (1995), Rastogi, (1990) Simpson et.al., (1990, 1988 and 1981) and Simpson, (1976 and 1986), to mention a few. However, it can be assumed that a probable prediction is reasonable and that past patterns of seismic activity will continue. Hence earthquake risk reports should take into account an appraisal of known faults, the distance from major faults, the number of recorded earthquakes, the history of damage, and an estimate of the magnitude and intensity of the strongest shock expected. The latter must take the ground conditions at the site into consideration (Wahlstrom, 1974).

5.15.2 Seismic activity

The engineer is particularly interested in two aspects of seismic activity, namely

I. Whether natural earthquakes, of an intensity that may cause damage to the dam or appurtenant structures, are likely to occur in close proximity to the dam; and

II. Whether the filling of the reservoir might induce earthquake activity - with the possibility of damage to the dam or liability for damage to other structures or persons. Though the magnitude of the shocks may be low, the proximity of the epicentres could produce serious effects.

Two scales rate earthquakes, namely the Mercalli scale of intensity and the Richter scale of magnitude.

5.15.2.1 Intensity

"Most intensity scales are related to human experience and structural damage. Earthquakes intensity scales depend on human perceptibility and the destructibility of earthquakes. The Mercalli scale is based on observations of actual earthquake effects at a specific location." Bell, (1980).

Thomas, 1976, claims that there is no clear relationship between ground acceleration and intensity, although he acknowledges various empirical formulae have been put forward. The commonly adopted scale is the modified Mercalli scale; such as is described by Wood and Newman in 1931 and slightly modified by Richter in 1956 (Bell 1980) and by the U.S. Atomic Energy Commission as well as the scale used in Japan. Because of wide variations in intensity within an area as a function of variations in surface and subsurface geological structure, the emphasis is on the effects observed at a particular point of reference, rather than at the origin (Wahlstroun, 1974). This means that the intensity of an earthquake at a particular location is closely dependent upon subsurface geological conditions as expressed by the kinds and distributions of rocks and unconsolidated deposits, according to Thomas, (1976).

5.15.2.2 Magnitude

The intensity of an earthquake is characterized by its effects and is a qualitative concept, whereas its magnitude is an instrumentally measured quantity related to the total energy released during an earthquake. (The magnitude of an earthquake is an indication of its absolute size, or total energy released. It is measured by the Richter scale, an arbitrary logarithmic scale, which defines magnitude in terms of the maximum amplitude of a standard seismometer at a distance of 100 km from the epicentre). The intensity of an earthquake is a measure of its effects at a particular place, and depends on both the magnitude and the distance from the epicentre.

5.15.3 Induced Seismicity

Guha, (2000) says that induced seismicity broadly depends on geotectonic setup, structural features and fracture patterns in the reservoir area, hydrological conditions, maximum depth and rate of filling up of the reservoir. He says that a normal or strike-slip fault environment in the reservoir area and moderate seismicity favour RIS. He mentions that patterns of induced seismicity differ very much and some of them are characterized as follows:

- Irregular microseismicity for some years,
- Intense microseismicity for short time immediately following impoundment,
- Moderate RIS activity with maximum earthquake of magnitude between 3.0 and 5.0 and
- Intense RIS activity with maximum earthquake of magnitude 6.0 and above such as Koyna (India), Hsingfengchiang (China), Kariba (Zimbabwe / Zambia) and Kremasta (Greece).

Induced seismic activity has been ongoing on at Hsingfengchiang and Koyna for the last three decades. Guha, (2000); W.Y Chung et al., (1992); T Lane, (1978); D.W Simpson, (1976 And 1986); Simpson et.al., (1981; 1986; 1988; 1990), D.T Snow, (1972); Gupta, (1985); Gupta et al., (1972); Gupta, (1992); Thomas, (1976); Bell, (1980), Lomnitz, (1974) and Wahlstrom, support the idea that earthquakes might be triggered by artificially induced changes in the pore pressures in the rock mass.

To support the idea of artificially induced earthquakes, Evans (1966) gives an example in which he suggests that the earthquakes, which affected Denver between April 1962 and November, 1965, numbering over 700 in total, with magnitudes not more than M4.3, were the result of the injection of waste fluids into a 3 660 m deep disposal well. His opinion is that it appeared that the movement took place as the pore pressures were raised by the injection of the waste. Bell, (1980) agrees with Hubbert et al., (1959) that more specifically rising pore pressure reduces resistance to movement along fracture planes, releasing elastic energy, thus generating an earthquake.

The Colorado School of Mines investigated the Denver earthquake and they arrived at the conclusion that water injected into the arsenal well could indeed have contributed to the earthquake activity (Bell, 1980). Similar activity has been noted on the Himalayas where the frequency of such small earth tremors is related to flood conditions in the rivers, where a high rate of change of flood intensity being particularly likely to trigger off minor swarms (Guha, 2000). Thomas, (1976); Guha, (2000), give as an example on the Witwatersrand Gold Mines, where the stopes are effectively plane, about one metre high, and greater than thirty metres in length. These constitute slits at a depth greater than 3 km and their prolongation causes rock bursts, which are seismic events of a magnitude up to 5.0. The greater part of the energy goes into the fracture but some passes into the earth as seismic waves.

Seismicity can as well be induced using underground explosions. A good example of this is underground nuclear explosions. Thomas, (1976), gives an account on carefully monitored underground nuclear explosions as powerful sources for study of seismic events in 1969. He says from over twenty observations made on nuclear explosions underground, he concludes that it is quite evident that they are capable of triggering seismic events. He suggests that they may be activating stressed fault zones that are already highly stressed tectonically.

5.15.4 Reservoir Filling

Houser, (1969), Healey, (1974) and Healey et al., (1968 and 1965), are of the opinion: that the filling of the reservoirs is somehow connected to seismic activity, and the seismic activity at the reservoir sites may be attributed to a number of factors. Those factors are listed below after, Bell, (1980):

1. The load of the water could be sufficient to initiate seismic activity.

2. Reservoir water permeating the underlying strata may increase the in-situ pore pressure thereby decreasing the effective normal stresses so that shear strength along any local faults is reduced.
3. Increase in hydrostatic pressure due to the percolation of reservoir water may cause a redistribution of stress, which initiates seismic shocks.
4. Increased pore pressures and saturation may reduce the strength of rock masses sufficiently to facilitate the release of crustal strains.

It is evident that a number of earthquakes that have been recorded have their epicenters below or near large reservoirs. This evidence is, however according to Bell, (1980) and Thomas, (1976), not conclusive. They assert that in the case of reservoirs this is not surprising since very few reservoirs are instrumented to record local seismic events. However, they quickly acknowledge that this has changed as dams are built higher and reservoirs impound large volumes of water resulting in this cause and effect relationship needing to be resolved, since induced earthquakes may cause serious damage. These thoughts have been proven correct by a number of reservoirs from all over the world in the passed and in recent years. It is therefore true that impoundment does bring about seismic events or increase the their activity. Nevertheless, what is not clear is whether the impoundment can stabilize or reduce seismic events of the side as it has been mentioned elsewhere in literature.

5.16 Reservoir Induced Seismicity (RIS)

Generally, very small seismic events (M=3.0) are induced almost after initial loading of the reservoirs while medium seismic events (3.0 to 5.0) are induced with certain 'time lag', several; months to a few years or even more (Guha, 2000). It has been observed at the Katse Reservoir that immediately during impoundment seismic events of the above mentioned magnitudes have been experienced. The smaller seismic events (M=3.0) may be induced by direct sagging of reservoir crustal blocks due to water load and subsequent adjustments along faults, fractures etc (Guha, 2000). He says that the medium seismic events (M3 to 5) with 'time lag' are, however triggered due to pore pressure migration to deeper levels through permeable strata. According to Talebi, (1999), Guha, (2000), this time lag can be partially explained by the diffusion mechanism of Howells 1974 and thus lends credence to the hypothesis of reduction of effective stress of faults by pore pressure and subsequent seismic slips. The order of observed 'time lag' in RIS cases world-over is roughly corroborated from the theoretical diffusion process of Howells 1974.

Intensive studies have been used to isolate some site characteristics, such as:

- Moderate seismic environment
- Normal and strike-slip fault

Following the impoundment of the reservoir and deep fluid injection according to the following authors; Gupta et al., (1972), Gupta, (1992), Gupta et al., (1997), Visser et al., (1990), Guha, (2002), Cook, (1981); Gough, and Gough, (1970a), Simpson, et al., (1976), Wahlstrom, (1976), Thomas, (1976), Bell, (1980).

Guha, (2000), is of the opinion that, inspite of this large number of case studies, they still constitute a very small fraction of the total number of existing water reservoirs of various dimensions. He argues that since only a few reservoirs induced intense seismicity (M=6.0), that, in addition to the above site characteristics, there are definitely other unknown factors favouring RIS activities, especially intensely induced seismicity (M=6.0).

5.17 Dams and Earthquakes

Generally, dams withstand shaking remarkably well, ICOLD, (1995). There are very few recorded instances of dam failures resulting from earthquakes, although many dams have suffered deformation and damages.

ICOLD, (1995), has reported in a survey of dam failures 183 occurrences. Five of which are mostly related to earthquakes. In comparison, forty-five occurred due to overtopping by floods. ICOLD gives earthquakes as the cause of failure of:

- Embalse Amoros Dam, Chile, in 1985
- Lliu-lliu Dam, Chile, in 1985
- Van Norman Dam, USA, in 1971

Liquefaction, presumably resulting from earthquake, was the cause of failure of:

- Sheffield Dam, USA, in 1925
- Niznhe Svirskaya Dam, USSR, in 1935

Free drainage rockfill dams with a thin impervious element are regarded as inherently stable under earthquake shaking. This is particularly so for concrete faced rockfill dams and upstream sloping core rockfill dams, which have a large mass of drained rockfill so that earthquake effects cannot cause reduced stability due to high pore pressures. It has been pointed out that concrete faced rockfill dams have not yet been subjected to strong earthquake shaking (over 0.20 m/s^2) and their performance remains untested.

In 1984, the Leroy Anderson Dam (California) is said to have experienced a peak ground acceleration of 0.40 m/s^2. The 72 m high, earthfill core rockfill dam (dumped and sliced rockfill) sustained two systems of longitudinal cracks, which apparently resulted from differential settlement between the core and the shells. The crest settled 15 mm and moved 9 mm downstream (Bureau et al., 1985 cited in ICOLD).

In 1943 the Cogoti Dam (Chile), experienced ground accelerations estimated to be in the range 0.15 m/s^2 to .30 m/s^2. This 159 m high dam is a dumped rockfill structure with an impervious upstream facing of laminated concrete. The resultant settlement of the crest was 280 mm and subsequently there were minor rockslides on the downstream face.

Earthquakes can trigger reservoir rim slides that in turn could lead to dam failure by creating a wave that overtops the dam as occurred at Hebgen Dam, (ICOLD, 1994). Outlet towers, bridges and other appurtenant structures have failed due to seismic loading: Such failures do not usually endanger the dam, but could result in an uncontrolled loss of storage.

5.18 Discussion

Every dam site has unique geological characteristics. Gaining a thorough understanding of these characteristics is expensive and time consuming: million of rands may have been spent on a geological survey before it finds that a site is unsuitable for a dam. It is said that it is normal for the dam to be designed with only a partial knowledge of the local site conditions and that the builders just have to hope that they will not find any unstable formations which will fail to support their foundations or cause the roof of their tunnels to collapse. A very important statistic that more than three quarters of 49 projects assessed in a 1990 World Bank study of hydropower construction costs are found to have experienced unexpected geological conditions of some kind. The study concludes that for hydrodams ' the absence of geological problems should be treated as the exception rather than the norm.'

- Reservoir induced-seismicity is well established, although it is a little known fact that large dams are capable of triggering earthquakes. The first observation of possible reservoir induced-seismicity (RIS) has been noted for Algeria's Quedd Fodda Dam in 1932; the first extensive study of the correlation between increased earthquake activity and variations in the reservoir depth has been made in the 1940s on Hoover Dam. Today, there are evidence linking earth tremors, reservoir-induced earth tremors, and reservoir operation for more than 70 dams. Reservoirs are believed to have induced five out of the nine earthquakes on the Indian peninsula in the 1980s, which have been strong enough to cause damage. All earthquake activities around Katse Dam are believed to be a consequence of Katse Dam itself. As with most of seismology, the actual mechanisms of RIS are not well understood, and it is impossible to predict accurately which dams will induce earthquakes or how strong the tremors are likely to be. Most of the strongest cases of RIS have been observed for dams over 100 m high.

It is also believed that dams with half the above height have induced quakes. Reservoirs can both increase the frequency of earthquakes in areas of already high seismic activity and cause earthquakes to occur in areas previously thought to be seismically inactive. The latter is the most dangerous as structures in areas thought to be quiescent are not built to withstand even minor earthquakes. The poorly built houses at Mapeleng on the eastern shores of Katse Dam fell prey to this. Complicating the picture further is the existence of five reservoirs, including Tarbela in Pakistan, where a reduction in local seismic activity has been noted after impoundment. The most widely accepted explanation of how reservoirs induce seismicity is related to the extra water pressure created in the microcracks and fissures in the ground under and near a reservoir. When the pressure of the water in the rocks increases, it acts as a lubricant for faults, which are already under tectonic strain, but are prevented from slipping by friction of the rock surfaces and this has been the assumed case at Mapeleng immediately during the impoundment of the Katse Dam.

For most well studied cases of RIS, the intensity of seismic activity increased within about 25 km of the reservoir as it is filled. The strongest shocks normally occur relatively soon - often within days but sometimes within several years – after the reservoir has reached its greatest depth. After the initial filling of the reservoir, RIS events normally continue as the water level rose and fell but usually with less frequency and strength than before. The pattern of RIS is, however, site specific for every reservoir.

The most powerful earthquake thought to have been induced by a reservoir is a magnitude 6.3 tremor which flattened the village of Koynanagar in Maharashtra, Western India, on 11 December, 1967, killing around 180 people, injuring 1 500 and rendering thousands homeless. The tremor has been felt 230 km from its epicenter. The epicenter of the tremor and numerous fore- and aftershocks have been all either near the Koyna Dam or under its reservoir.

RIS is suspected to have contributed to one of the world's most deadly dam disasters, the overtopping of Vaiont Dam in the Italian Alps in 1963. The 261 m Vaiont, – the world's fourth highest dam - was completed in 1960 in a limestone gorge at the base of Mount Toc. During the impoundment of the reservoir, seismic shocks have been recorded and a mass of unstable rock debris on the side of the mountain started to slide toward the reservoir. After reaching a maximum depth of 130 m in late 1960, the reservoir has been partially drained, and the seismic activity and slope movement have almost stopped. The reservoir has then been re-impounded, provoking a new increase in tremors. Despite the tremors, the engineers and geologists, have decided 'that the mass would keep moving so slowly that no problem would occur.' The experts have been proven in this instance wrong, because heavy late summer rains in 1963 have been swelling the reservoir, in the first half of September 60 shocks have been registered and the movement on Mount Toc started to accelerate. On the night of 9 October, 350 million cubic metres of rock broke off Mount Toc and plunged into the reservoir. The gargantuan wave resulting from the impact overtopped the dam by 110 m – the height of a 28-storey building. About two minutes later the downstream town of Longarone was leveled and 2 600 people died, almost all its inhabitants killed. The actual relationship between seismic activity and landslide is not certain, but it is likely that the numerous shocks at the very least hastened the collapse of the mountainside.

Leonard Seeber, a seismologist at the Lamont-Doherty Earth Observatory at Columbia University, New York, believes that official maps, which show the areas most at risk of earthquakes, should also indicate the increased risk near many reservoirs. If this were to happen, communities near reservoirs could presumably demand compensation to 'earthquake-proof' buildings, greatly increasing the cost of dams. The dam industry would probably strongly oppose any such measures, which would raise awareness of RIS. Seismologist Harsh Gupta, Vice-Chancellor of Cochin University in India and a professor at the University of Texas, notes a general reluctance in parts of the engineering community, worldwide, to accept the significance or even the existence of the phenomenon of reservoir-induced seismicity.

5.19 Reservoir Geology

5.19.1 General/background

More than any other form of civil engineering construction, adequate geological knowledge of dam site conditions is an essential (Duncan, 1969, Thomas, 1976, Olivier, 1976).

5.19.2 Geology of the basin

5.19.2.1 General

The entire reservoir area has been intensely dissected by the active erosion of the Senqunyane River system and its tributaries. The higher sloping pediments above EL 2500 on the mountains surrounding the reservoir are considered to be related to the African Erosion Surface while a distinct pediment is found to the east of the reservoir in the Ha Tsiu area above EL 2200. A terrace above EL 2100 occurs in the upper Jorotane River catchment, immediately upstream of the Mohale reservoir inundation area. The numerous oxbows formed from cut offs of the Senqunyane River and its tributaries form the only other less steep areas.

Structural discontinuities, such as major fractures, jointed shear zones and intrusive dolerite dykes have significantly controlled the development of the topography, such as the resistant Jorotane dolerite dyke, which forms a number of elongated capped ridges. Drainage courses and galleys are often linear and controlled by lineaments and shear zones. The overall drainage pattern is generally dendritic in form. The dominant direction of drainage is in a southerly direction.

In the reservoir basin area, the Senqunyane River flows in a southwest direction to the dam site after which it follows a more south – southeast route to join the Senqu (Orange) River some 50 km downstream (fig. 5.1-1 and fig. 5.3-2). Two major tributaries join the Senqunyane River in the reservoir area, i.e. the Bokong and Jorotane Rivers that flow in a general southerly direction while the smaller Likalaneng River enters the reservoir from the west (fig. 5.3-2). The Bokoaneng River is a tributary of the Bokong River and drains the western Mohale Dam catchment, while the two significant stream courses draining from the south east and entering the left bank of the Senqunyane River are the Litsuming and Tsoelike streams. All the major river and stream courses follow incised meandering routes along the main drainage directions with structural control only affecting the routes in isolated cases.

Nine oxbows will be affected by the inundation, most of which are relatively minor features. The Ha Tsapane covers some 140 ha. It occurs in the centre of the reservoir. It will be completely inundated. Other significant oxbows that will be completely inundated are the Ha Piti and Ha Mohlabane oxbows immediately west and north west of the dam site, respectively, while the Ha Makhobalo and Khamolane oxbows on the Bokoaneng and Senqunyane River will be only partly inundated.

The slopes are generally steep with many terraces consisting of bedrock ledges or abandoned flood plains such as in the oxbows. Rapids are common in the main rivers, associated with more resistant and durable rocks and structural control. The only major waterfall is the 12 m high Semonkoaneng Waterfall on the Senqunyane River, 3 km downstream from the dam site. Waterfalls and rapids are common in the minor tributaries and drainage courses where the streambeds are generally rockbound.

5.19.2.2 Soils

The geology in the reservoir area generally comprises thin colluvial soil cover on rock slopes with extensive rock outcrop on the steeper mountain slopes. Deeper residual soils are developed on some of the older landscapes at higher elevations, such as the remnants of the African erosion surface and pediments. The deepest soils associated with the oxbows occur in the old river channel. Alluvial deposits occur along the banks of the major river courses but they are not extensively developed. Talus or scree deposits occur below the steeper slopes and are generally very thin where found. Localized slumps in the colluvial soils have been recognized along the lower valley slopes of the Senqunyane River at the dam site but have not been recognized in other areas of the reservoir basin.

5.19.2.3 Lithology

The bedrock of the reservoir area consists of horizontally layered basalt flows of the Lesotho Formation. Flow thicknesses vary between 1 and 10 m with occasional thicker flows of up to 40 m. A number of dolerite dykes have been recognized in the reservoir area. These are variably oriented in a NW-SE and WNW-ESE direction with one dyke to the south of the reservoir oriented in a NE-SW direction. The most prominent among these is the major Jorotane dyke system, which crosses the reservoir in the Ha Tsapane oxbow area forming a prominent outcropping ridge (fig. 3.3-1, 3.3-2 and 3.3-3). It has a length in excess of 20 km and forms one of the major regional dyke systems through Lesotho. The Ha Takatso dyke can be traced from the village of Phomolo in the west of the reservoir, through the village of Ha Takatso and crossing the Senqunyane River one kilometer downstream from the dam. It consists generally of a 3 to 10 m wide zone of jointed dolerite, intruded into basalt with typical conjugate shearing on the boundary.

5.19.2.4 Geological Structure

The Lesotho basalts are crossed by significant continuous structural discontinuities of which at least three major alignment directions can be identified across the reservoir area. The most prominent of these directions are the ESE ranging to SE and E trending broad topographical lineaments found throughout the mountains of Lesotho. These lineaments are typically associated with veined fracture zones and coincide with closely spaced sheared jointing with filling by zeolite, calcite and to a lesser extent silica. Individually identifiable fractures are typically less than 5 km in length although they combine to form swarms of structure extending over the whole study area. They are crossed by two significant structural fabric orientations (a NNW_SSE alignment and an NE_SW alignment). These structural features in the reservoir area are typically in the form of prominent individual joints or swarms of joints. They are often associated with zeolite filling, occasionally form prominent veins, and shear planes. They are usually near vertical aligned and occasionally sub-vertical and are continuous, extending both laterally and vertically through the basalt pile. These features are usually associated with springs, higher permeability and deeper weathering at surface.

At least six broad structures can be identified crossing the reservoir. They are:

- Likalaneng-Sekokoaneng zone
- Ha Piti – Mohale Dam zone
- Ha Takatso – Ha Tsiu zone
- Ha Tsapane – swarm of dykes and lineaments
- Ha Mokhati – Masaleng zone
- Letlapeng – Ha Mokoto zone

These zones, which form a series of prominent structural lineaments, are typically zones of sympathetic sheared jointing and zeolite veining, and are sub parallel to the dykes where present. A strong developed N – S topography lineament set has been recognized to the east of the reservoir area, crossing the catchment in the upper Bokoaneng River. This set crosses the Mountain Road in the Blue Mountain Pass area. Similar but less prominent N – S trending lineaments cross the upper Jorotane River in the vicinity of Ha Rapokolane and the upper Likalaneng River to the west of the Likalaneng village. These structural lineaments may be associated with a southern extension of the major Seepe Fault line in the underlying Clarens and Molteno Formations as seen in the Mapheleng River valley below the escarpment to the north of the catchment.

5.19.3 Groundwater

5.19.3.1 General

Seepage from the high ground surrounding the catchment towards the river dominates the ground water conditions in the reservoir. The primary source area for the ground water in the catchment is locally developed pediments and sponge areas generally higher than EL 2400. In general, the basaltic lava is dense rock mass that can be considered as impermeable due to little structure and open jointing. The sequence of layered lava flows results in a restricted vertical permeability with only the widely spaced structural discontinuities, weathered open flow contacts and relatively shallow stress release jointing forming secondary drainage paths.

The high natural permeability of the open and continuous jointed structural features passing through the reservoir will result in the rating of the hydrostatic head in these permeable features and in interconnected jointed zones. This can potentially lead to the formation of springs or seepage points outside of the reservoir basin, where lower hydrostatic heads are present. This can potentially lead to a movement of ground water away from the reservoir through these features. Conditions where this could occur are limited, and can only be identified at two places:

- Along the eastern extension of the Ha Takatso – Ha Tsiu fracture zone where the stream draining the Ha Tsiu pediment is deeply incised to a depth of between 50 and 70 m below the reservoir full supply level (FSL)
- Possibly, along the eastern extension of Ha Piti Fracture zone on the left abutment of the dam.

5.19.3.2 Groundwater at the Foundation

The ground water regime is complex, with perched water table and several confined aquifers, as is usually the case in basaltic rock mass consisting of many lava flows. During the dry season deeper water levels were recorded on the higher flanks (>60 m) and shallower levels near the river area. A number of boreholes encountered artesian flow from zeolite filled joints below the river area.

About 100 water pressure tests have been conducted in boreholes along the plinth line. A summary of the water pressure test results along the plinth line area is presented in Table 5.19-2. The results of the water acceptance tests indicate that the permeability of the foundation rock under the plinth line is very low. However, localised veined zones and lineaments, which were filled in with fresh and weathered zeolite, exhibited high lugeon values (Table 5.19-1).

Table 5.19-1: Range of Average Lugeon Values

Borehole No	Study	Length (m)	Location	Range of Average Lugeon Value
LDR 01	Feasibility	84.05	Left Abutment	0 - 0.3
LDR 101	Planning	100.08	Left Abutment	0 - 0.2
LDR 201	Design	55.6	Left Abutment	0 - 0.4
LDR 202	Design	39.9	Left Abutment	0 - 0.8
LDR 102	Planning	64.5	Left Abutment	0 - 0.2
LDR 215	Design	21.5	Left Abutment	0
LDR 216	Design	20.6	Left Abutment	0
LDR 204	Design	64.3	Left Abutment	0 - >100
LDR 205	Design	98.5	River	0 - 8
LDR 206	Design	94.1	River	0 - >100
LDR 214	Design	37.9	River	0
LDR 217	Design	41.8	River	0.13
LDR 207	Design	79.5	Right Abutment	0 - >100
LDR 213	Design	42.1	Right Abutment	0
LDR 208	Design	69.6	Right Abutment	0
LDR209	Design	65.4	Right Abutment - Lineament EL 2000	0 - >100
LDR218	Design	29.8	Right Abutment - Lineament EL 2000	0 - >100
LDR219	Design	65.7	Right Abutment - Lineament EL 2000	0 - 4.7
LDR211	Design	30.1	Right Abutment	0 - 3.3
LDR210	Design	72.2	Right Abutment- Lineament EL 2084	0 - >100
LDR06	Feasibility	18.5	Right Abutment- Lineament EL 2084	0.8 - 20
LDR07	Feasibility	24.4	Right Abutment- Lineament EL 2084	0.3 - 1.5
LDR 107	Planning	75.1	Right Abutment- Lineament EL 2084	0 - 3.7

Table 5.19-2: Water Pressure and Grouting Pressure in Bladed Basalts and Lineaments

Depth (m)	Grouting Pressures (kPa)	
	Water	1:1.5 mix or thicker
0 - 5	200	250
05-Oct	325	375
Oct-15	450	500
15 - 20	575	625
20 - 30	700	750
> 30	1 200	1 250

The required pressure could not be attained in certain zones due to insufficient pumping capacity to overcome the high water loss. Low take has been recorded at EL 2000 and the other at EL 2080 in general (the upper portion but high water loss at depth). Moreover, the infill material within the lineaments is considered to be potentially erodible.

5.19.3.3 LHWP- Tunnels

5.19.3.3.1 Transfer Tunnels

During the excavation of the 'Muela tunnels, claims have been received from villages about reduced spring yields. The information about spring flows being very little prior to excavation; hence, assessment of that situation has not been possible.

5.19.3.3.2 Mohale Tunnel

Groundwater inflows confined to the first 5.80 km of Intake drive have been in general consistent with that predicted by the hydrogeological model. From Ch. 5850 to the end of the Intake TBM Drive, numerous locations have been encountered showing significant different geological conditions . These conditions have been responsible for the rate of decay of inflows, in association with the regional lineaments or fault zones. Tables in the Appendix B display the behaviour of such geological structures from the intake to Ch. 15000.

The fault conditions have been predicted to be mainly healed and tight, lined and infilled. In fact there have been numerous fault zones, which comprised variably healed, incompletely infilled fault veins, with an irregular, interconnected system of pipe-like openings. The openings range from 10 mm to 100 mm (classified as wide to very wide). The extent of the interconnected openings system could be inferred from a fissure grouting operation between Ch. 10455 and 10465, where grout has been found to travel over 330 m, to where it has been encountered in the face at Ch. 10788. Actual encountered conditions during tunneling are displayed in Appendix B.

Predicted Ground Water Conditions versus Actual Encountered Conditions.

- There are 33 lineaments predicted, comprised of six dolerite dykes and 27 joint zones/unidentified lineament. Eighty-four lineaments of which nine are faulted dolerite dykes, 47 faulted zones and 28 jointed zones have actually been encountered.

- Of the 33 lineaments predicted, 24 have been predicted to be water bearing. Sixty-nine of the 84 encountered lineaments has been associated with ground water ingress.
- Generally, the predicted ingress has been modeled to decay after 12 hrs to 33 % of the initial ingress. Of the actual number of lineaments encountered, only 33 of the 69 have behaved according to the modeled criteria of decay.
- The hydrogeological model has defined fracture classes and associated representative ingress associated with these classes. The actual ingress data has been divided into similar classes to represent the associated decay within each ingress class, which has behaved in accordance with the decay criteria as specified above.

5.19.3.3.3 Impounded Katse Dam

The dam has been impounded during the latter part of 1995. The piezometric measurements from the left and the right banks at different elevations do not show any significant change in the groundwater regime. This is clearly demonstrated by the large number of water level and seapage relation-graphs produced and they can be seen at the Operation Centre at Katse dam offices. In this case, the fluctuations of the reservoir level have little or no influence in the water regime below the dam.

The springs in the villages along the shoreline do not show any change in their yields since the impoundment, with the exception of one spring at Ha Paepae Village. This spring is just 20 m from the reservoir FSL and is situated on a fault zone. The villagers have initially reported the change in discharge, with the hope of making some compensational claims. The spring has been closely monitored and it has been observed that not only the reservoir contributed to the change but the seasons as well. The discharge peacked with the rainy season and dropped with the dry period. It has therefore been concluded that there is a direct connection between the discharge with the rainfall rather than the lake level. All other springs higher up behaved similarly. They peak up with the rainfall and it has been observed that the spring can go dry, while the reservoir is still full. The springs flow rate does not correlate with the periods when the rate of filling of the reservoir increases significantly. This kind of behavior is also demonstrated by the springs along the Mohale Tunnel alignment.

5.19.4 Slope Stability Background

Figure 5.19-1: The steeply incised Slopes of the Senqunyane River. Senqunyane Valley flowing towards you from downstream from Dam Wall

The natural steeply dissected slopes of the basalts of the Lesotho Formation (fig. 5.19-1), together with the many cliff faces and waterfalls are indicative of the overall stability of the basalt formation in the reservoir basin). The two structural discontinuities affecting the stability are the horizontally aligned flow contacts and the vertical aligned joints and lineament structures. The probability of major rock slope instability because of the combined effect of these geometries and saturation by the reservoir filling or rapid dewatering is unlikely.

6 THE LESOTHO HIGHLANDS SEISMOLOGICAL NETWORK

6.1 Seismograph Instrumentation

- Data Logger: a multiplexer connected to a desktop runtime system (DRTS)
- Seismometer: three-component short period, Mark products L4-3D with multi-seismometer (MS) event recorder.
- Instrument timing: events are transferred every 5 minutes from the remote sites to the multiplexer

6.2 Network Design

Dr. Graham of CGS gives the following information/data on the design of the Lesotho Highlands Development Authority Seismological Network (LHDASN):

- The Lesotho Highlands Development Authority Seismological Network monitors the seismicity of the Katse Dam area. The original network of three stations has been installed in 1991 and has consisted of three stations KTS, KT1 and KT2. An additional analogue station, KT3, has been added to the network in February 1996 at the village of Mapeleng. A digital station has been installed in March 1996 near the main office of the Lesotho Highlands Consultants (LHC) close to the retaining wall.
- Construction of the seismological station buildings and vaults starts in April 1997. The equipments at KTS and KT1 have experienced problems during the beginning of 1997, and as they have become obsolete after the upgrading of the network, a decision has been made not to facilitate any repairs. Furthermore, the equipment at KT2 and KT3 start experiencing pen problems, but the operator/analyst, provided by the Council for Geoscience (CGS), has to keep them running through on-site repairs.
- During the beginning of December 1997, the only digital equipment in operation, at this period, (KTE) has been supplemented through the installation of four new digital seismographs. This has involved the relocation of the seismological station at the Mapeleng village (KT3) and the replacement of the Portacorder equipment at KT2 and KT3 with digital recorders, 3-component seismometers and radio telemetry equipment. A third seismological station has been installed at Ha

Suoane, and in order to complete the coverage of the Katse Dam a fourth remote station has been installed at Ha Soai, to the south of the reservoir. The following station codes are in use:

i KTS 1 for the station at the LHC office building (this station has been previously known as KTE). Although the system has been originally designed to function as a stand-alone station, it can be operated as part of the KTSN at its current location. Data from this station is relayed to the office via cable and is incorporated with the data transmitted from the other seismological stations.
ii KTS2 for the station at Mamohau (has been KT2)
iii KTS3 for the station at Mapeleng (has been KT3)
iv KTS4 for the station at Ha Suoane
v KTS5 for the station at Ha Soai

6.2.1 Network Management

The council for Geoscience of the Republic of South Africa has been contracted to run the network and train a local seismologist, with the aim of handing over the day-to-day running of the network. A mining engineer has been trained for this purpose and he has been very efficient in running the network. Unfortunately, he has passed away in a car accident and the network has once again to be run by the CGS until a new Bsc.graduate has been recruited and trained.

6.2.2 Data Analysis

The seismological events are located using a modified version of HYPOCENTER software which uses a rectangular coordinate system and 'flat-earth" layered velocity model. The layered model travel time calculations follow the procedure developed by Eaton, 1969, for a one-dimension stack of layered with uniform thickness.

Local magnitudes are calculated according to the duration magnitude relation developed by Lee, et al., 1972:

$$M = 2 \log (T) + 0.0035(D) - 0.87 \qquad (1)$$

Where T is the coda length in seconds (measured from the P – wave on set to where the trace merge with background noise), log is the logarithm to the base 10 and d is the epicentral distance in kilometers. Focal depths for known and suspected explosions are being determined using different focal depths to obtain the best convergence.

Data recorded at the remote stations is relayed to the data processing facility at the LHC offices via radio link. The more distant stations (KTS2 and KTS5) have been equipped with 1 Hz seismometers. In addition, a 1 Hz seismometer has also been installed at KTS4. To increase the frequency coverage near the wall, KTS1 is recording 6 channels of data through both 1 HZ and 4.5 Hz seismometers. All these seismometers are three-component units and are oriented north south, east west. The sampling rates for 1 Hz and 4.5 Hz seismometers are 250 and 2000 samples per second, respectively.

6.2.3 Data Dissemination

Seismological bulletins are compiled every month and distributed to the relevant authorities. These bulletins have been compiled since the installation and running of the network in 1991. The purpose is to keep all interested parties up to date on the seismological events in and around the reservoir.

6.3 Induced Seismicity at the Katse Reservoir

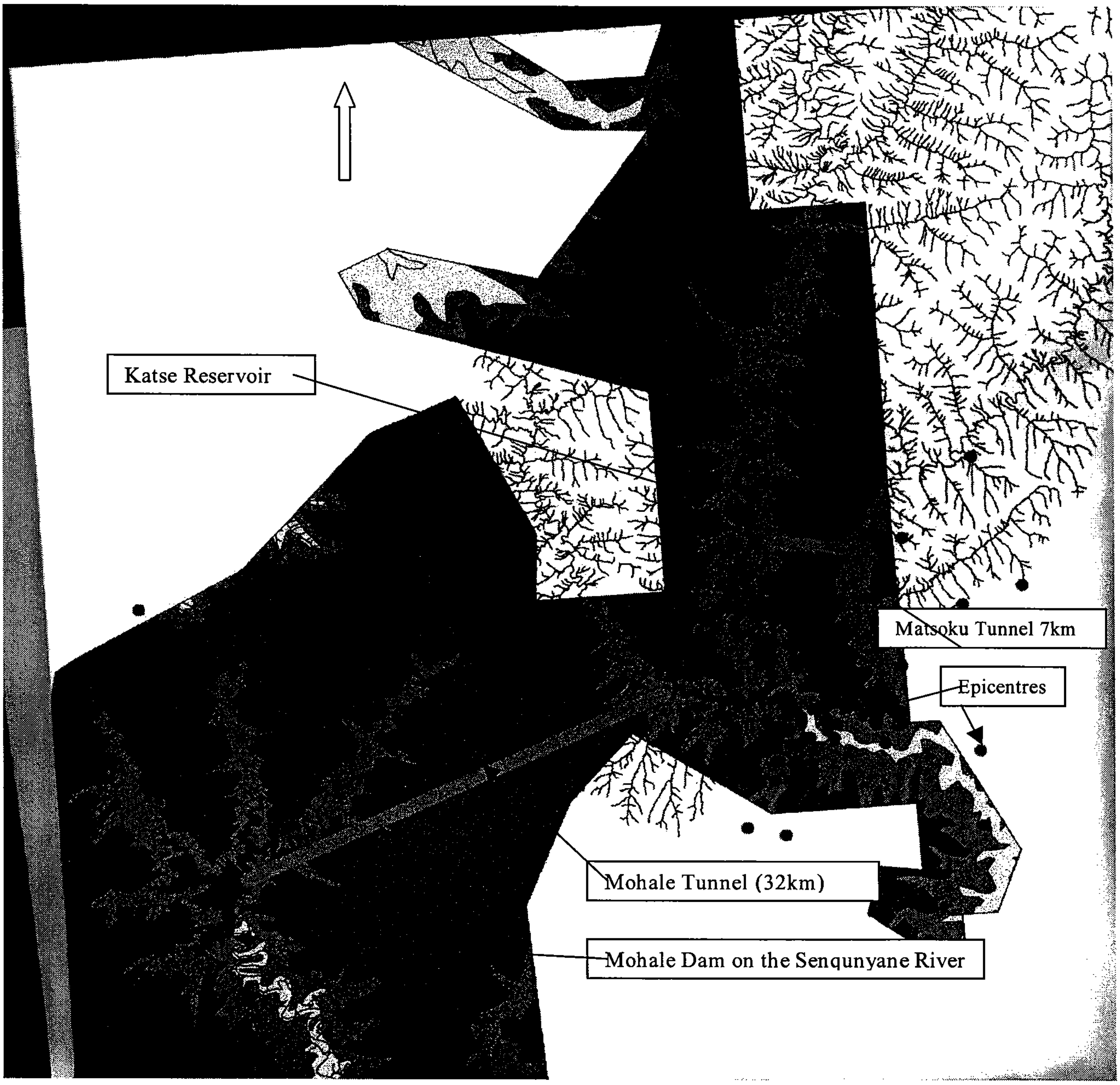


Figure 6.3-1: Epicentres around the Katse Reservoir

The Katse Dam is situated in an area that is believed to be aseismic. The highlands of Lesotho have induced micro- and ultramicro earthquakes following its impoundment in November 1995. Impoundment and seismic events (M≥3.3) history of the reservoir area within 10 km are shown in (fig. 6.3-1). The event data starts in November 1995 at the initiation of the impoundment of the Katse Dam. Initially within the first, few months, until April 1996, only the date, time, estimated epicenter distance and the coda magnitude of events have been reported (Graham, 1998). The first events begin when the maximum depth is 30 m. In that period, between100 and 200 events (M≥3.3) occurred in quick succession in close vicinity of the reservoir (≤10 km) thereby supporting the hypothesis on reservoir-induced seismicity at the Katse Reservoir. In the period from April 1997 a further 56 events have been recorded near the Katse Reservoir. Figure 6.3-1 presents a map of the reservoir with the epicentres and their magnitudes.

A crack, five kilometres long, has developed in the ground from the eastern side of the reservoir across Mapeleng village up to the mountain adjacent to Mapeleng and thereby caused cracking to the poorly built houses (stone and mud as building material). Graham, (1998), suggests that the triggering mechanism is indicative of the effect of the depression of the crust by the weight of water in the reservoir. His argument is in line with other scientific conclusions that have been made on reservoir induced seismicity elsewhere; such as those of Gupta et al., (1976); Simpson, (1976); and Thomas, (1976). The extent of microseismicity is related to the elastic strength of the crust and existing weaknesses. Hence, micro-earthquakes could be detected along all fracture zones. The micro-earthquakes are so small in terms of magnitude that they are hardly felt by the residents along the shores of the reservoir. However, some events have occurred within five kilometres of the Mapeleng Village and one has been actually located on the Mapeleng shear zone. The Mapeleng Villagers has felt this and others as just unusual noise.

6.4 Discussion

Dr. Graham of CGS says it is evident that most of the current events occur within six to eight kilometres of the reservoir, Figure 6.3-1. Most of the initial seismicity has occurred along the east bank of the reservoir and it is evident that a significant number of events have occurred on the west bank of the dam. Events have been recorded to the south east of the dam and the Bokong branch of the reservoir, since the upgrading of analogue to a digital network. He concludes that in the area of Mapeleng and Soai those microseismic events are associated with the Mapeleng shear zone. The fact that, according to Graham, (1998), those microseismic events are detected in different areas along the same fracture zone could be an indication of small-scale movement along that entire fracture zone in the vicinity of the reservoir.

The seismicity north of Mapeleng traverses the Ha Seshote fault zone and does not cross the reservoir. He would like to have another seismological station on the west of the reservoir along the Ha Seshote fault zone, so that seismicity observation could be increased.

The study of induced seismicity around the Katse Reservoir continues. The upgrading of the existing network and provision for increasing the number of stations of the network would provide better evidence of the influence of the Katse Reservoir as well as the future the Mohale Reservoir on seismicity. Upgrading and an increased number of stations would be especially important in providing an insight into induced seismicity and aid in the hazard assessment of the LHWP area. It is concluded that the filling of the Katse Reservoir brought about the seismic events.

6.5 Seismicity Potential at Mohale Reservoir

The Mohale Dam will be impounded soon. The Dam is a concrete faced rockfill dam with a height of 147 m and a reservoir volume of 946.9 million cubic metres, with a surface area at full capacity of 21.2 square kilometres. It is part of phase 1 of the LHWP. The dam has been built in the basalt rock of the Lesotho formation across the Senqunyane River. The reservoir lies totally in the one and only Lesotho Formation of the highlands of Lesotho. The area is regarded to be seismically quiet like the whole project area is. The experience gained during filling of the Katse Reservoir, suggests that moderate RIS can be anticipated in the Mohale Reservoir area, especially upstream of the dam. S. Talebi, (2000), according to Skempton's effect, the temporal association of RIS with filling of the reservoir shows that in some cases, shallow, small earthquakes are associated with reservoir impoundment. Since the Katse Reservoir also lies in the same formation, it is worthwhile to extrapolate the lessons learned during its impoundment. Monitoring of seismicity is at par with the international standards at the Katse Reservoir. The Council of Geoscience of South Africa is one of the world's leading specialists in seismicity and they have been entrusted with the responsibility to install and run an analogue seismological network well before the impoundment of Katse Reservoir. Reliable data has been collected and compiled. Due to the ageing seismological equipment at the stations, the whole network has been out phased and has been replaced with a state of the art telemeter digital network. The Katse reservoir has therefore been seismically monitored since its impoundment. It is noted here that similar geological conditions prevail at the Mohale Reservoir site. (One possible mechanism responsible for the earthquakes is that of the loading effect of the reservoir and the substantial rainfall.). Those similar geological set-ups suggest that moderate RIS in the reservoir area can be anticipated, especially upstream from the dam. It is still poorly understood why impoundment of reservoirs induces seismic activities (Talebi, 1999). Nevertheless, the amount and global scale of evidence of reservoir-induced seismicity and the studies thereon, have given to seismology an interesting topic that continues deservedly to draw the attention of many seismologists (Talebi, 1999). Hence, on the same note, Mr. Nthako, General Manager of the Engineering Group has organised a weeklong workshop on induced seismicity with special reference to the Katse Dam. It has been held at the Maseru Cabanas Lesotho in 1998. Distinguished academicians and specialists on earthquakes have presented their valuable papers focusing on the LHWP.

Dr. Graham at this workshop gives a summary review of the reservoir-induced seismicity at the Katse Dam. He has highlighted the following points:

- History of the Katse Dam Telemeter Network (KTSN)
- Seismic activity
- Completeness of data
- Rate of energy release

Moderate, though in some instances destructive, seismic activity has been monitored and located around Katse Reservoir during the initial period of impoundment. More than 150 seismic events of coda magnitude 3.3 and below are taken to be as events of induced nature. The seismological network has been able to capture events of magnitude 1.5 and greater. The seismic activity rate correlates with the periods when the rate of filling of the reservoir increases significantly. The destructive event of magnitude 3.1 in January 3, 1996 has not recurred. A larger event of magnitude 3.3 has occurred in April 1996, but with no reported negative consequences. Katse area has no evidence of any historical seismicity. These facts give credence to the hypothesis that the filling of the reservoir triggered the January 3, 1996 event and associated seismicity.

Three analogue seismological stations have been located since 1991 near the Katse Reservoir site. Review of the stations prior to the impoundment of Katse reservoir reveals that there have been no earthquakes recorded. Thus, all evidence suggests that the Katse Reservoir has indeed brought about seismic activities in the area. With the above information on local seismicity, increasing and a running review of the seismic events it can be concluded that Mohole reservoir is likely to induce seismic activity during its impoundment.

The Katse Reservoir is Lesotho's largest reservoir and it is one of the world highest reservoirs, if not the highest in terms of altitude, with the reservoir maximum volume at 2053 masl. Maximum Supply Level is at 2053 and the Minimum Supply Level is at 1978 m.a.s.l. Dead storage is $2.3*10^9$ m^3, dam height, 187 m and maximum storage $6*10^9$ m^3. The geological evidence suggests that the studied area has not been tectonically active in recent geological years. For monitoring earthquake activity prior and after the filling of the Katse Reservoir, the Lesotho Highlands Development Authority has contracted the Council for Geosciences of South Africa to equip and maintain a seismological network since 1991.

The Katse area is geologically in a stable continental region and so is the Mohale Dam site. Given all the conditions that prevail at the Katse Reservoir it is to be expected that seismic events would be experienced around the Mohale Reservoir.

6.6 Probabilistic Seismic Impact of the Mohale Dam

The Mohale Dam is smaller than the Katse Dam, in terms of reservoir storage capacity and in terms of damwall height. The Mohale Dam is a concrete face rock-fill dam while the Katse Dam is a double curvature concrete dam. Both dams are located in the Lesotho Formation. The seismic events at the Katse Reservoir during the filling are regarded in this work as a model according to which any prognosis for the impoundment of the Mohale Dam may be made. It has been noted that all of the events observed at the Katse Reservoir since its impoundment are very small in magnitude, posing no serious threat to the dam except to the poorly built houses near the reservoir. The largest event recorded measured only 3.3 as the maximum magnitude to be observed. The fact that both basins are located in a stable region implies that very little threat is expected from reservoir-induced events.

A magnitude 6.5 at 10 – 20 km has been used as a maximum design earthquake for the Mohale Dam. The existing state of stress around the reservoir will dictate the probability of induced earthquakes. The earthquakes will be small to moderate (=4), MCG, (1996). There is nothing in the Mohale Dam area, or anywhere in Lesotho, to suggest that a deep-seated earthquake is imminent, MCG, (1996). The panel of experts agrees to the draft Mohale Dam Seismic Design Report on the likelihood of occurrence of induced seismic events due to the history of seismicity in the area. However, their magnitudes will be smaller than those of potential natural seismic events will. This has also been observed at other reservoirs as mentioned in Rastogi, et.al., (1995), Rastogi (1990), Gupta, (1985), Gupta et.al., (1972), Guha et., (1992), Chung, (1992), Gough et.al., (1976), Lomnitz, (1974) and Lane (1974).

7 EMBANKMENT OF THE MOHALE DAM

7.1 Introduction

The Mohale Dam is the first major compacted concrete face rockfill dam (CFRD) to be built in Southern Africa. The dam is about 145 m high, with an approximate fill volume of 7.8 million m^3. With full supply level at EL 2075 the dam reservoir provides a storage reservoir of 947 million m^3.

The basic design aspects of the Mohale Dam are addressed in this section. The design is also well documented in the final report on the Mohale Dam by MCG, from where most of the data and information in this section has been extracted. The design is empirical, based on experience relating to previously constructed dams. The fig. 7.1-1 shows the pre-cofferdam, upstream cofferdam, plinth with consolidation grouting and grout curtain, impervious fill and transition zones, Mohale Dam Embankment (with its concrete face, full supply level, rockfill and oversized rock zone) and the downstream cofferdam.

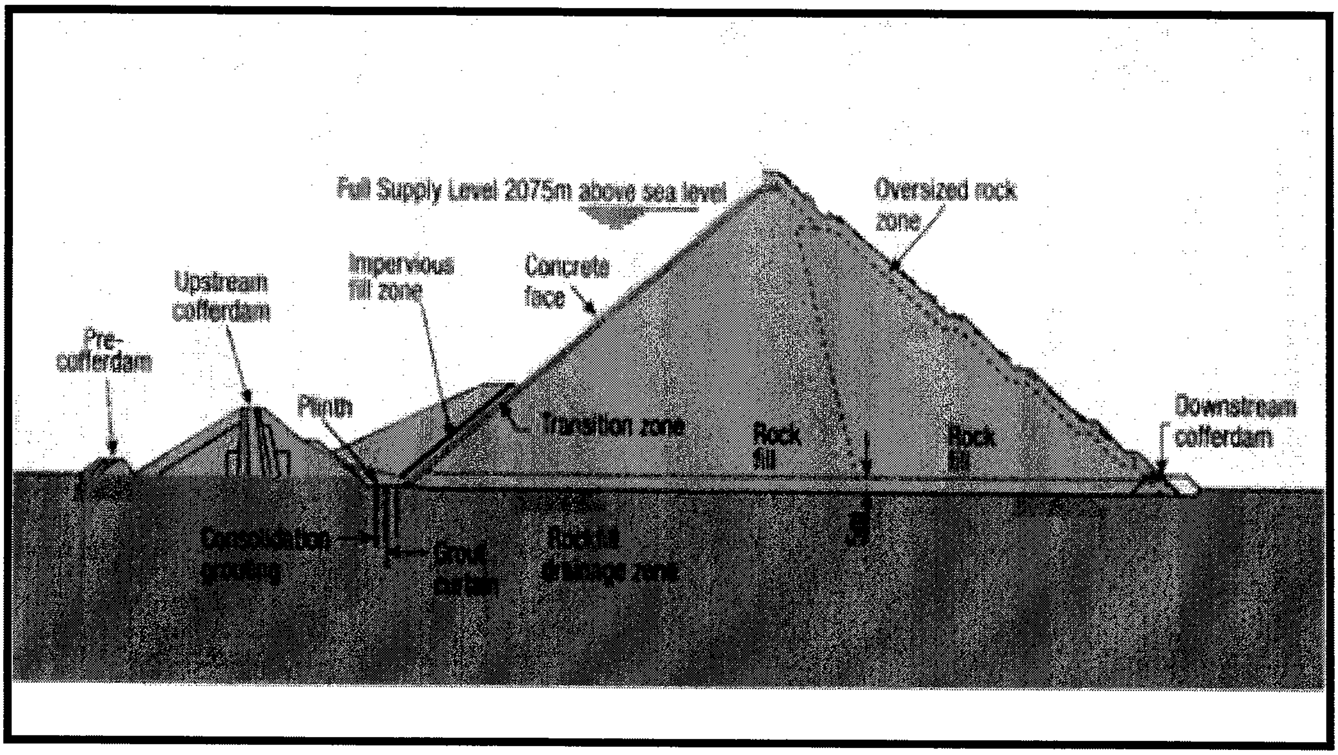


Figure 7.1.1: The Mohale Dam Embankment

The principal project data are given in Section 5.1 with the principal data for the Mohale Dam shown in Table7.1-1.

Table 7.1-1 Principal Data for the Mohale Dam

Description	Quantity
Maximum Height	145 m
Crest Length: Left to Right Abutment Walls	540 m
Crest Width	9.9 m
Elevation of Top of Rockfill (Maximum Section Including Camber)	EL 2084 m
Elevation of Top of Parapet Wall (Maximum Section Including Camber)	EL 2086.0 m
Excavation Volume	243 000 m^3
Rockfill Volume: Zones 3B, 3C, 3D and 3E	7.35 x 10^6 m^3
Rockfill Volume: Zones 2A, 2B and 3A	0.35 x 10^6 m^3
Total Rockfill Volume	7.70 x 10^6 m^3
Face Slab Area	81 000 m^2
Face Slab Concrete Volume	34 000 m^3
Plinth Length	727 m
Upstream Slope	(1V: 1.4H) m/m
Downstream Slope (Between Roads)	(1V: 1.25H + Berms) m/m
Downstream Average Slope	(1V: 1.45H) m/m

7.2 Selection of Type of and Height of Dam

The Mohale Dam site is considered to be very suitable for the construction of a CFRD. This conclusion has been reached after assessment of the geological as well as the environmental conditions of the site. The dam type is defined in the Engineering Services Contract LHDA 1017.

The Feasibility Studies for the Katse Dam, Mohale Dam and Matsoku Diversion undertaken by the Lahmeyer McDonald Consortium in 1986 conclude that the full supply reservoir elevation (FSL) of the Mohale Dam should be at EL 2083, and that the FSL at the Katse Dam should be around EL 2040. Later these levels were changed to FSL for the Mohale Dam at EL 2075 (lowered by 8 m) and for Katse at EL 2053 (raised by 13 m). The dam type and FSL at EL 2075 were defined during the Planning Study in 1995. During the tender design phase yield analyses were performed based on updated hydrological records, and the full supply level at optimum cost was re-assessed.

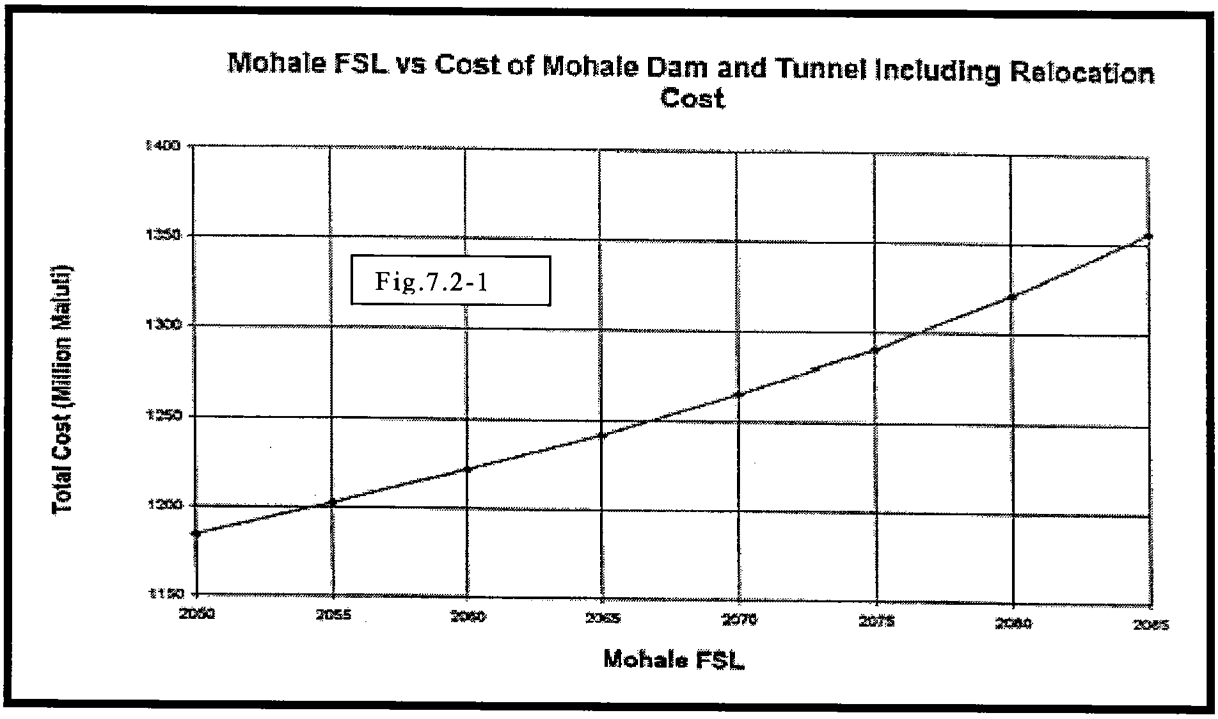


Figure 7.2-1: The Mohale Dam Full Supply Level vs. Cost of the Mohale Dam and Tunnel including Relocation

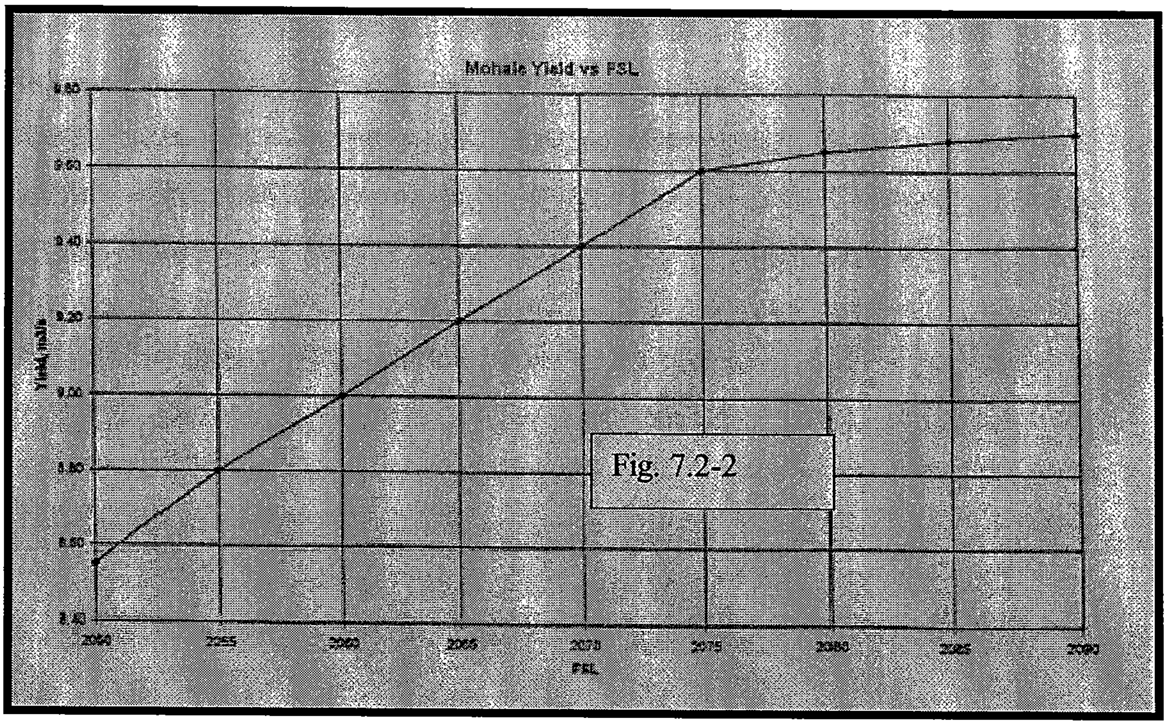


Figure 7.2-2: Mohale Yield vs. Full Supply Level

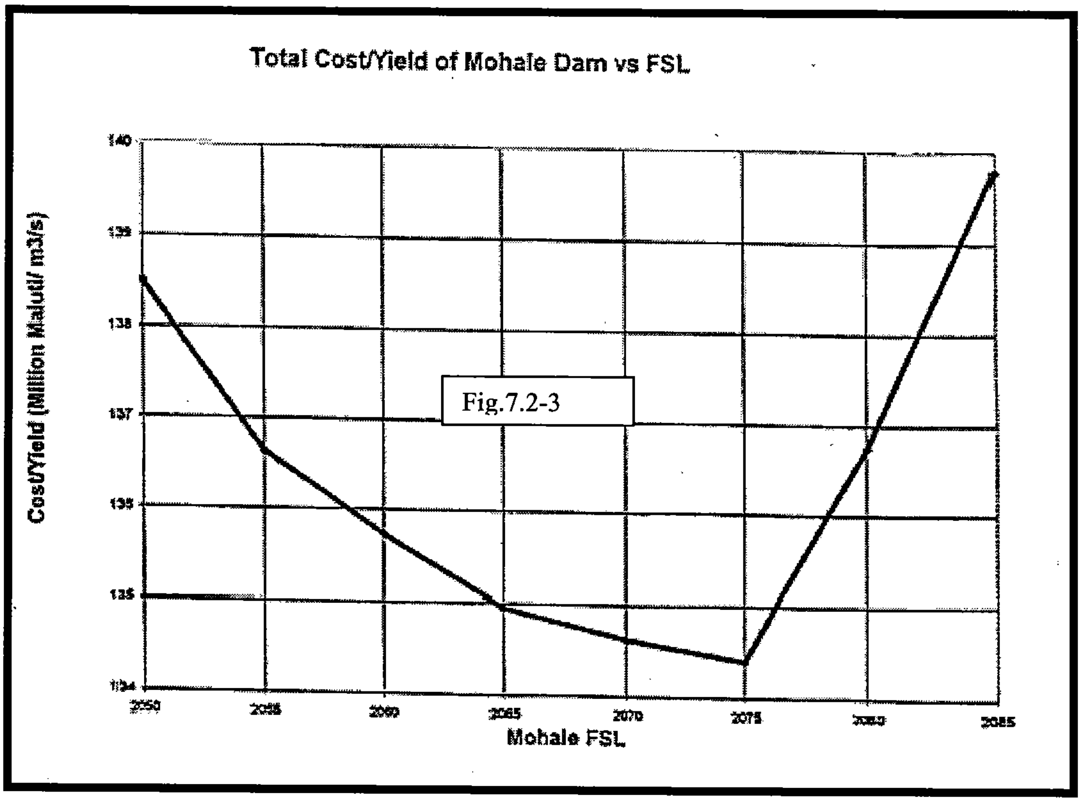


Figure 7.2-3: Total Cost/Yield of Mohale Dam vs. Full Supply Level

Fig. 7.2-1 shows the cost of the Mohale Dam and the transfer tunnel versus FSL, while Fig. 7.2-2 shows firm yield of the Mohale Dam versus FSL. Fig. 7.2-3 shows the cost of yield from the Mohale Dam per m^3/s versus FSL, including the cost of the transfer tunnel. Fig. 7.2-4 shows the incremental cost curve. Table 7.2-1 summarizes all the values applied in the analysis.

Table 7.2-1 Summaries of Costs and Yields

FSL	Mohale yield* (m/s)	Mohale** Costs (million M)	Incremental Mohale yield (m^3/s)	Incremental Cost (million M)	Mohale Cost/ Mohale yield (million $M/m^3/s$)	Incremental cost/incremental yield(millionM/m/s)	Mohale Dam Environ-mental Cost(million M)
2 050	8.55	1 184	0	0	138	0	45
2 055	8.8	1 202	0.25	18	137	73	47
2 060	9	1 222	0.2	19	136	96	49
2 065	9.2	1 242	0.2	20	135	100	52
2 070	9.4	1 266	0.2	24	135	120	54
2 075	9.6	1 290	0.2	25	134	124	56
2 080	9.65	1 321	0.05	31	137	512	59
2 085	9.7	1 356	0.05	35	140	875	60

* Mohale yield reflects the total Phase 1B yield minus the contributions from Katse and Matsoku weir.

** A Mohale cost includes capital for the Mohale Dam, Mohale Tunnel and Mohale environmental costs.

*** Cost Figures are based on data available in May 1997

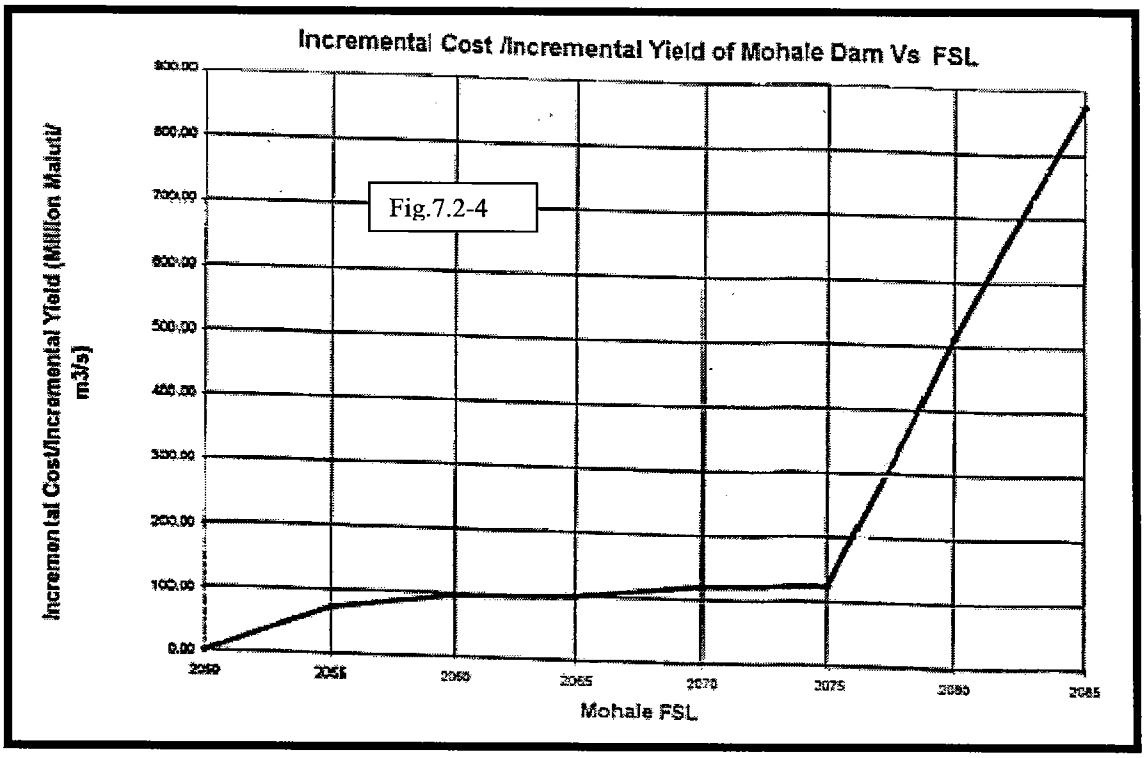


Figure 7.2-4: Incremental Cost/Incremental Yield vs. Full Supply Level

The values in the third column of Table 7.2-1 include the cost of the tunnel to the Katse Reservoir, the Mohale cost of relocation from LHDA, and the Mohale Dam implementation cost. Fig. 7.2-4 shows incremental cost of moving from one FSL to the next, plus environmental cost as listed in the last column, plus cost of the transfer tunnel from the Mohale Reservoir to the Katse Reservoir. The curves on Fig. 7.2-3 and Fig. 7.2-4 would be steep if fixed cost, e.g. infrastructure is added in the above assessment.

Applying the 1996 updated hydrology for the Katse, Mohale and Matsoku and relevant and available cost data including environmental and relocation cost as obtained from LHDA, the Mohale reservoir level at EL 2075 clearly represents the most economical FSL.

7.3 General Arrangement at the Mohale Dam

7.3.1 Location of Cofferdams, Mohale Dam, Spillway and Diversion Tunnels

Fig. 7.3-1 shows the location of the Mohale Dam Embankment with its spillway, the Cofferdams and the Diversion Tunnels.

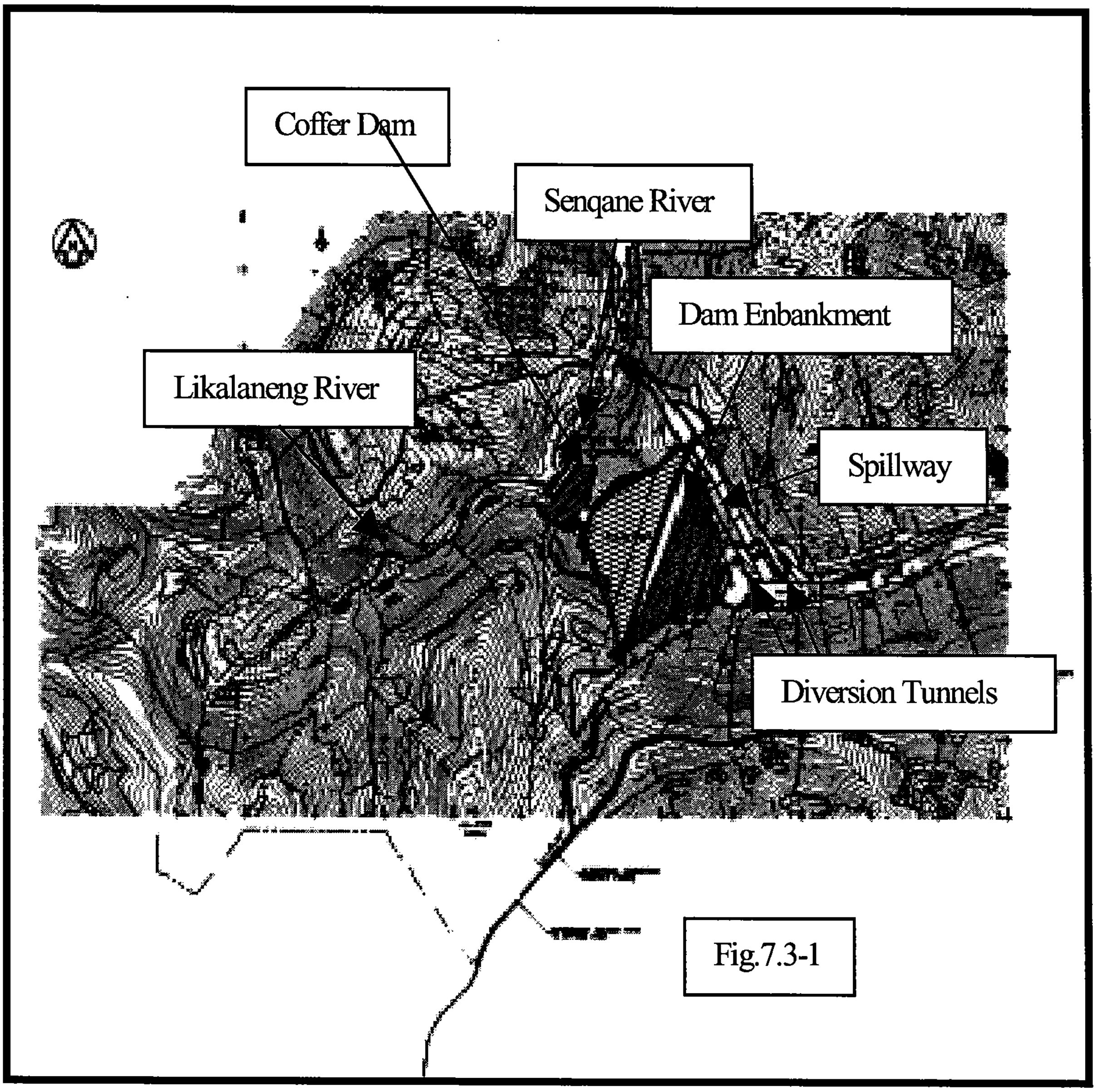


Figure 7.3-1: Location of Cofferdams. Mohale Dam, Spillway and Diversion Tunnels

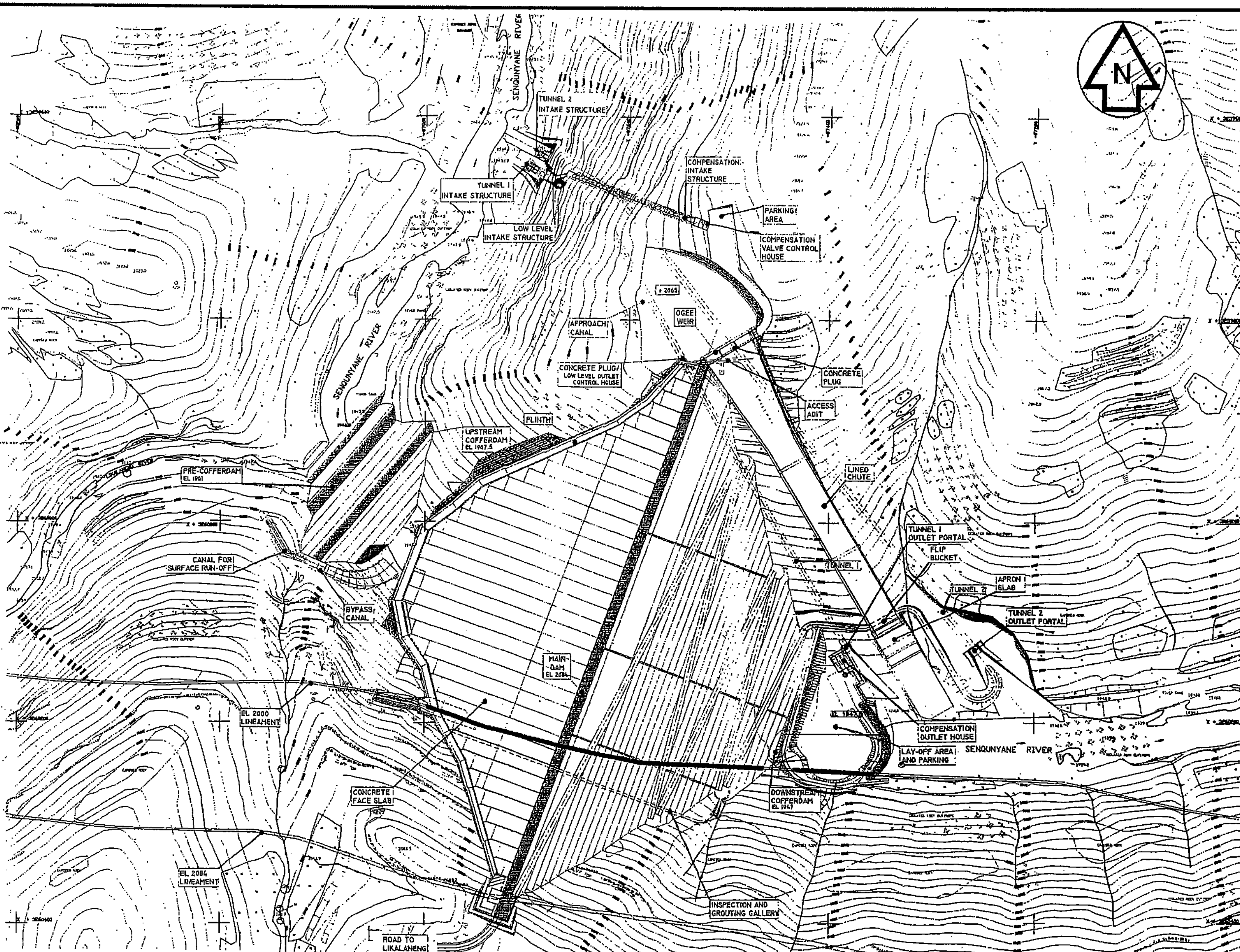


Figure7.3-2: Mohale Dam and Ancillary Structures

In the project layout optimisation of the following approach has been adopted, which provides the opportunity to locate the Mohale Dam Embankment further upstream:

- The Pre-cofferdam and the Upstream Cofferdam were combined and located at the furthest possible upstream position, letting the Pre-cofferdam to basically lie in the upper confluence area between the Likalaneng and Senqunyane Rivers.

This location is advantageous because it brought about a shortening of the diversion tunnels, a reduced excavation for the spillway and improved the plinth foundation conditions.

- A minimum clearance of 15 metres was provided between the downstream toe of the Upstream Cofferdam and the Mohale Dam Embankment to provide access for construction of the plinth.
- To reduce costs and to shorten the diversion tunnels, the Downstream Cofferdam is incorporated into the downstream toe from the Mohale Dam Embankment.
- Due to the construction sequence, tunnel 1 was closed in May 2001 to facilitate the installation of the low-level release facilities. The river then flowed through tunnel 2. The outlet of Tunnel 2 has been rotated downstream of the flip bucket for the following reasons:
  1. To provide space for construction of the flip bucket;
  2. To ease access to Tunnel 1 from downstream during lining of the tunnels and during installation of the Low Level Outlet Works; and
  3. To ease placing of the concrete slab below the flip bucket.

7.3.2 Site Works, Access Roads, Quarries and Borrow Areas

The principal quarries and borrow areas are located as shown in (fig. 7.3-2). The Contractor's Works Areas have been incorporated within the site.

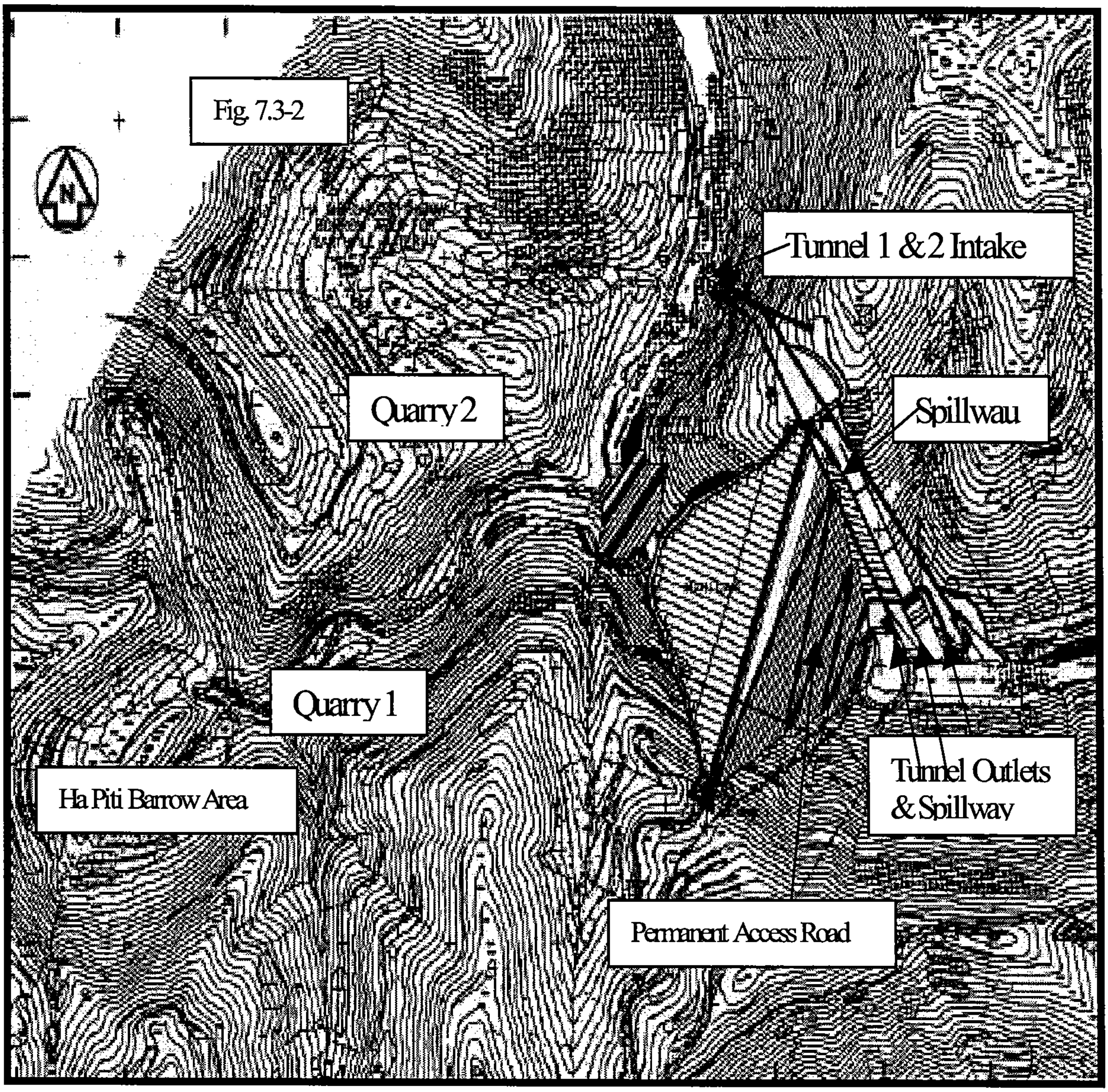


Figure 7.3-3: Location of Construction and Permanent Access Roads, Quarries and Borrow Areas

The arrangement of the layout is based on environmental requirements, namely, these areas are located below the full reservoir supply level, and available materials from within the reservoir area are used. This arrangement is very important in terms of costs and environmental disturbance reductions, because the distance from the quarries to the construction site has been reduced and the quarries will be buried under water. The required quarries and borrow areas are the following:

- Quarry 1 for rockfill, for the Mohale Dam Embankment and the cofferdam (see panorama of photos).

- Quarry 2 for concrete aggregate (see panorama of photos), drainage rock (Zone 3E), and crushed rock, (Zone 2A, 2B, and 3A, and quarry run rockfill (Zone 3C and 3D).
- In addition Quarry 2 provides rockfill for the top section of the Mohale Dam Embankment in case of inundation of Quarry 1.
- Borrow area for core material for the cofferdams and for fill upstream from the face slab. This material will also be used in the Upstream Cofferdam. Alternatively, semi-impervious materials from stockpile, originating from foundation excavation for the Mohale Dam Embankment may be used, which renders its construction more economical.

The effect on the environment has been taken into consideration as can be seen from the locations described below. The temporary working areas have been located below the full supply level while the permanent working area, including the crusher and screening plant, batching plant, workshops, offices and laboratories have been located above the full supply level on a plateau adjacent to Ha Piti Borrow Area. Work platforms have been formed in part by excavating into the hillside and in part by fill. Temporary access roads have been located below the full supply level. As the fill is hauled into the dam from upstream, several ramps are cut into the abutments. This is done in a manner minimizing the effect on the plinth layout and construction.

A permanent access road with a slope of 10% has been provided to the outlet facilities on the downstream slope of the Mohale Dam Embankment (Fig. 7.3-2). This road is an all-weather road. Zone 3C Rockfill has been placed in the river section, over the Downstream Cofferdam abutting against the Compensation Outlet House to serve as a working and parking area and to provide space to locate the access roads to the Low Level Outlet House and to Tunnel 1 (Fig.7.3-2).

7.4 Embankment Layout of the Mohale Dam

7.4.1 Introduction

Fig. 7.3-1 and Fig. 7.3-2 show the layout of the Mohale Dam Project. Fig. 7.1-1 shows the cross section of the dam at maximum height, a full supply level of EL 2075, and the various zones of the embankment, with a slope of 1.4 horizontal to 1 vertical on the upstream side and a slope of 1:25 horizontal to 1 vertical between roads on the downstream slope. The average downstream slope is 1:45H:IV. The water-retaining barrier is the concrete slab, also referred to as the face slab.

It is however, important to note that:

- Some 860 000 m^3 rockfill has to be placed on the left side of the river prior to the construction of the cofferdams in order to complete the placement of rockfill of the Mohale Dam before impoundment commences; and
- The Face Slab should have been constructed to EL 2040 at an early stage to enable early impoundment by 1 October 2001, a deadline that has not been met due to environmental issues, which has not been addressed.

7.4.2 Earthfill on top of Face Slab

Earthfill covering protects the plinth perimetric joint as an additional protection. This fill has been extended for the Mohale Dam to cover the lineament EL 2000 upstream of the plinth. This fill has been economically designed to be lower in the middle section to cover the plinth with a minimum soil depth of 8 m.

7.4.3 Downstream Slope

The layout of the downstream slope of the Mohale Dam is shown in Fig. 7.3-3. An access road to the downstream areas has been provided at a maximum 10% slope with hairpin bends as shown in the figure. Access is also provided from one hairpin bend to the Inspection and Grouting Gallery on the right abutment.

7.5 Foundation Preparation

This section discusses the various requirements for the foundation of a rockfill dam. These requirements have been specified and observed by the construction team of the Mohale Dam.

7.5.1 Rockfill

The following characteristics for the foundation were required:

- To be equal or better than moderately weathered, moderately fissured and fractured rock, with shear strength better than the rockfill placed above.

- At least 85% of the foundation surface will be exposed, and areas of unexposed foundation should be limited to 1 m in any direction.
- In general, excavation should continue until about 25% of the foundation is sound rock.

To achieve these characteristics the following was necessary:

- For a minimum distance of 15 m downstream from the plinth below EL 2040 and 10 m above EL 2040, the foundation must be excavated to the same level and quality as required for the plinth foundation, cleaned with air/water jet and treated with slush grout or shotcrete to lengthen the leakage path.

Within this area, joints or erodible seams extending from the plinth foundation are cleaned out to a depth at least equal to twice their width. Narrower seams are filled with mortar and the wider ones with backfill concrete. Where these defects are numerous and closely spaced, a continuous reinforced concrete blanket is being placed.

For a distance of half the hydraulic head downstream of the plinth and under any concrete slab downstream from the plinth, erodible infillings in seams and fractures below the general level of the foundation has been excavated, backfilled with slush grout or backfill concrete and covered with a filter consisting of Zones 2A, 2B and 3A materials, to prevent erosion of fine materials.

Over the remainder of the rockfill foundation, where the foundation in local areas consists of completely weathered rock, such rock is excavated to a depth of not less than 0.8 m and then covered with 400 mm thick layers of Zone 2B or 3A materials. The final foundation clean up is done with light mechanical tracked or rubber-tyred equipment not heavier than 7 tonnes.

7.5.2 Crushed Rock Zone 2B and Zone 3A

The foundation is specified to consist of slightly weathered moderately fissured and fractured rock or better. Declivities and overhangs are trimmed back to 0.25 H: IV.

Where large permeable or erodible features are present below the foundation of the plinth, such weak zones are removed as directed by the Engineer and backfilled with concrete. A reinforced concrete blanket covers marginal areas downstream from the plinth.

Where excavation results in the production of low areas in the foundation, either:

- The base level of the plinth will be lowered; or
- The low area will be backfilled with concrete to the general level of the adjoining foundation.

If any low area is brought up to the general level of the adjoining foundation with backfill concrete, the shape and width of backfill concrete is designed to suit local conditions, ensuring stability under all loading conditions. Backfill concrete is used to reduce the height between the perimeter joint and the foundation.

When the excavation has been completed to the approximate levels, the surface is cleaned off by barring, wedging, and picking with an air and/or water jet under high pressure for the purpose of inspection. If the foundation is found to be unsatisfactory, supplementary excavation is undertaken and the surface again cleaned for inspection. This procedure is repeated until an acceptable foundation has been obtained. A final clean up of the rock surface is performed just prior to placing the concrete. This takes into account the removal of all loose, shattered, or disintegrated material and the final surface cleaning with jets of air and/or water under high pressure. That is followed by the removal of all water from depressions. Immediately afterwards, the surface is wet before the concrete is placed.

Joints and crevasses with infillings are cleaned out to a minimum depth equal to twice their width and backfilled with mortar or concrete. Closely jointed rock is slush-grouted.

7.5.3 Treatment Below Zone 1A, 1B and 1C

A zone of impervious material Zone 1A had been placed against the lower part of the concrete slabs and covered the lineament at EL 2000. It is anticipated that with the materials readily available, this zone consists of decomposed basalt with maximum particle size of 75 mm.

Permeable material is removed from under the Zone 1A material. The surface, on which Zone 1B material is placed, levelled and compacted prior to placement. The air-water jet is used to clean thoroughly the foundation for Zone 1C after all loose rock is removed. Immediately after the cleaning process, the zone is covered by 100 mm of backfill concrete. The concrete is covered by Zone 1C material prior to the final set to avoid drying out and cracking.

7.5.4 Treatment of the Foundation at Lineaments

The lineaments located on the right abutment at EL 2000 and EL 2084 receives the following special treatment:

- Access needed after construction has been provided by the inclusion of an Inspection and Grouting Gallery to the Lineament EL 2000 plinth connection and a concrete block at the right Abutment Wall on Lineament EL 2084. Both lineaments are easily accessible. The lineament/plinth contact at El. 2084 is easily accessible. This does not apply to the lineament/ plinth crossing at EL 2000, and due to the much larger head, more cut-off and safety measures have been included for this feature.
- Two metre deep excavation of the area at the plinth/lineament contacts and backfilling with concrete to provide a slab or socle. The socle is required for providing mass over the lineament for providing in turn a longer seepage path and to act as a grout cap. At EL 2000/plinth contact the socle is extended by 15 m upstream and 30 m downstream from the grout curtain. At the EL 2084/plinth contact these dimensions are taken as 10 m and 8.45 m respectively from the grout curtain. Providing a 400 mm wide PVC waterstop includes a movement joint in the socle on the lineament. The plinth is located on the socle.

- Consolidation grouting has been performed to a depth of at least 30 m over the complete area of the socle and to a depth of 60 m in the vicinity of the lineament. Curtain grouting is extended to depths of 80 - 100 m near and in the lineaments.
- Extensometers and pressure meters have been installed over the lineament. They monitor water pressures and movements from the Inspection and Grouting Gallery.
- Placement on the lineament of a filter consisting of a geotextile as well as Zones 2A, 2B and 3A material after excavation downstream of the plinth to prevent erosion of particles from the lineament.
- In addition to several lines of waterstops (top stainless steel, middle uPVC and bottom copper waterstops, placement of six layers of hypalon (high density polyethylene HDPE)) over the lineament in the plinth and first 15 metres of face slab. Placement of fill, consisting of Zone 2B, Zone 3A and cohesionless fines on the lineament upstream of the plinth, on the plinth and on part of the slab (also covering the HDPE sheets) to serve as a "crack stopper" if the downstream cut-off measures as explained before are malfunctioning or damaged. This fill is connected to the Zone 1A and 1B fill, which has been placed on the perimeter joint below EL 1090 on the left abutment and EL 2010 on the right abutment.

7.5.5 Treatment at Bladed Doleritic Basalt Area

The foundation is treated at bladed dolerite basalt area as specified and in the detailed description of the procedure and measurements. The treatment is necessary for the stability and reduction of potential seepage. For the bladed doleritic basalt area at the plinth on the right bank between chainages 360 to 400, a special design as described below is provided for to reduce the height of excavation cuts, to increase stability and to minimize seepage through the plinth foundation rock:

- The plinth width has been reduced to 7 m to save excavation cost upstream of the plinth.
- However, the plinth has been extended downstream by at least 3 m to maintain a low hydraulic gradient.

- As an extension of the plinth a 100 mm shotcrete or backfill concrete layer with 1.59kg/m^2 wire mesh has been placed on the excavated and cleaned foundation for a distance equal to 0.25 times the hydraulic head downstream from the plinth.

7.6 Instrumentation

There are generally two categories of measuring instruments in geotechnical instrumentation for monitoring:

- The first category is used for in-situ determination of soil or rock properties e.g. strength, compressibility, and permeability, normally during the design phase of the project.
- The second category is used for monitoring performance, normally during the construction or operation phase of the project, and may involve measurement of groundwater pressure, total stress, deformation, load, or strain.

In the design of the dam the ability of the ground to support the construction must be considered. Engineering observations during geotechnical construction are often an integral part of the design process, and geotechnical instrumentation is a tool used to assist with the observations.

Human capabilities:

Basic capabilities required for instrumentation personnel are reliability and patience, perseverance, a background in the fundamentals of geotechnical engineering, mechanical and electrical ability, attention to detail, and a high degree of motivation.

7.6.1 General

In this section of the chapter, the discussions on the instrumentation are looked into as discussed in the reports and checked against what has been actually done, in terms of instrumentation installed in conformity with the specifications. A discussion is provided and conclusion is drawn at the end of this section based on the site facts and upon the discussions have been held with relevant site engineers and technicians.

The discussion on the Mohale Dam Instrumentation is well documented in several reports but most precisely in the final report. It is a crucial part of dam construction to have all necessary instrumentation timeously in place as required by specification. Instrumentation is required to monitor the behaviour, performance and safety of the dam during construction, initial reservoir filling and operation, and to check the design assumptions. The purpose of the instrumentation is to monitor the following:

- Settlements within the dam to check the adequacy of camber provided for and the assumed engineering properties of the rockfill;
- Surface movements for the same purpose as stated above;
- Strain measurements in the face slab;
- Joint opening in the perimetric and face slab joints;
- Seepage;
- Ground water levels; and
- Earthquake motion, including movements along the lineaments.

The design process rarely enables the designer to predict precisely the behaviour of the dam. Hence, design assumptions and hypotheses must be supplemented and confirmed by field observations.

7.6.2 Scope of Instrumentation

Three cross sections, one at the maximum height of the dam and one each on the left and right abutment, have been instrumented fully to monitor internal behaviour of the embankment. In addition, other instruments have been installed to monitor the surface behaviour of the embankment (fig. 7.6-1).

The instrumentation installed includes:

- Hydraulic settlement gauges and tubes;
- Load cells;
- Geodetic pillars;
- Crest and surface settlement points;
- Thermometers;
- Borehole standpipe piezometers;

- Vibrating wire piezometers;
- Seepage measuring weirs;
- Strain meters and contraction joint meters in the concrete face;
- Perimetric joint meters;
- Switchboxes in recesses in the parapet wall and a portable indicator for reading the installed meters e.g. joint meters;
- Inclinometers;
- Crack monitoring pins and crack meters;
- Pore pressure cells;
- Extensometers;
- Water level indicators;
- Convergence anchors;
- Earthquake and strong motion monitoring equipment;
- Readout units for vibrating wire piezometers, open standpipe piezometers and inclinometers; and
- Data loggers and software will also be provided.

7.6.3 Measuring the Face Slab Strain

Face slab strains are of interest in determining the areas of tensile strain and the potential for cracking. The strain meters installed immediately under the slab and the inclinometer installed on the face slab measure the strain in the concrete face slab. The strain meters provide strain data only at the location of the meters while the inclinometer on the face slab provides a continuous strain profile along the slab. Close to the perimeter joint the strain meters have been installed in groups of three at 45° rosette formation and further up on the face slab these have been installed in groups of two.

An inclinometer on the face slab has been installed at two locations in the valley where the maximum settlement/deflection of the face slab is expected to occur. Adequate protection of the inclinometer casing has been installed on the face slab and has been provided to prevent damage of the casing by debris in the reservoir.

7.6.4 Joint Opening

Joint opening may occur in the perimetric joints and in the vertical construction and contraction joints. The openings of these joints are monitored to confirm that the total displacement is within the range, which the waterstops can accommodate without rupture. Openings or movements are measured normal in relation to the opening or closing of the joint, normal to the slab settlement and parallel to the joint (shear in plane of face). All three types of joint meters are provided at the perimeter joint. Only two types of joint meters to measure movement normal in relation to the joint and movement normal in relation to the slab are installed at the contraction and construction joints. Close monitoring of all contraction joints in relation to the opening of the perimeter joint are to be made as required.

7.6.5 Lineaments Monitoring

Two prominent lineaments are located on the right abutment at EL 2000 and EL 2084. They are of concern with regard to development of potential cracks/openings of these lineaments during filling and operation of the reservoir. To monitor such potential opening, two systems have been provided. One of these instruments is placed at the surface while the other is placed at depth below ground surface.

Potential surface crack/opening of the lineaments will be monitored employing simple survey pins installed on both sides of the lineament or a crackmetre. Borehole extensometers at depth below ground surface will monitor potential opening of the lineaments. Monitoring of surface crack/opening of the lineament and the opening at depth will be at a location immediately downstream of the dam embankment. Crackmetre and extensometers locations are shown in Instrumentation General layout.

7.6.6 Piezometers (Vibrating Wire and Casagrande Open Pipe)

A series of standpipe piezometers monitor the effects of seepage through the abutments and foundations of the embankment (fig. 7.6-2). Groundwater levels higher than observed prior to filling will indicate seepage through the abutments and the foundation. Standpipe piezometers have been installed in the abutments just above full supply reservoir level (FSL) and downstream at selective locations to monitor the ground water level and thus assess the effect of the reservoir on the groundwater regime and seepage. In addition, standpipe piezometers have been installed within the lineaments downstream of the dam to monitor seepage through and change in the ground water level in the lineaments.

7.6.7 Measuring Seepage at the Mohale Dam

Seepage through the face slab percolates through the embankment rockfill and exits at the downstream toe of the dam where it is measured. Seepage from the foundation and abutment and rainwater are mixed with seepage through the concrete face. V-notch weirs are used to monitor this combined flow on the downstream side of the dam. These are placed just above the normal operating tail-water level so that the flow can be measured at all times except during major floods. Seepage-measuring weirs have also been installed at the lineament El. 2000, downstream of the dam to monitor the potential leakage through the lineament. Furthermore, leakage through the Inspection and Grouting Gallery are also monitored at the entrance of the Inspection and Grouting Gallery.

7.6.8 Strong Motion Accelerograph

Two strong motion accelerographs (SMA), one on the crest of the dam at the maximum section and the other on a rock foundation have been installed to monitor the amplification through the dam embankment and the basic earthquake motion, respectively. The accelerographs are sensitive enough to record both the expected low and high range of acceleration at the site.

7.6.9 Microseismic Event Monitoring

Reservoir induced seismicity (RIS) is of concern at the Mohale Dam, particularly due to RIS events that have been experienced at the Katse Dam. Hence, five microseismic stations have been built around the reservoir area. The microseismic stations have been up and running before the impoundment to establish the base line seismic activity so that the effect of the reservoir on the seismic activity can be evaluated. The stations have been installed as planned and they are up and running. No events have been recorded that can be linked or correlated to the filling of the Mohale Reservoir.

7.6.10 Precision Bench Marks

Precision benchmarks have been installed at several locations around the dam site to serve as permanent benchmarks, which will be used as reference control points for all horizontal and vertical-survey controls. These benchmarks have fixed precision points to enable the quick setting up of the survey equipment.

All surface levels and movements will be measured using these benchmarks as control points. These benchmarks have been set up in conjunction with the national survey grid and benchmark system.

7.6.11 Monitoring in Diversion Tunnel Exploratory Chamber

The presence of stress relief joints has been observed during the geotechnical investigations conducted for the tender design. The planned depth of excavation at the Mohale Dam is relatively small compared to that at the Katse Dam where stress relief joints are of major concern. It is not expected that the existence of stress relief joints will have any significant influence on excavation and/or the performance of the structures at the Mohale Dam. However, it has been proposed to excavate a side chamber in the Access Adit between the diversion tunnels, during construction and to perform a dilatometer test, rock stress measurement with strain gauges, field point load tests on rock cores, laboratory unconfined compensation tests on rock cores and hydrofracturing tests. Additionally, extensometers have been installed in boreholes drilled from this chamber to monitor the potential movements in the rock mass caused by stress relief.

7.6.12 Training of Personnel

The contractor in compliance with the specifications has provided the services of the manufacturer's representative for the respective instruments to train the Contractor's, the Engineer's and Employer's personnel in the installation, calibration, testing, reading, operation and maintenance of the instruments, in conformity with normal international practice. Reading intervals are shown in Table7.6-1.

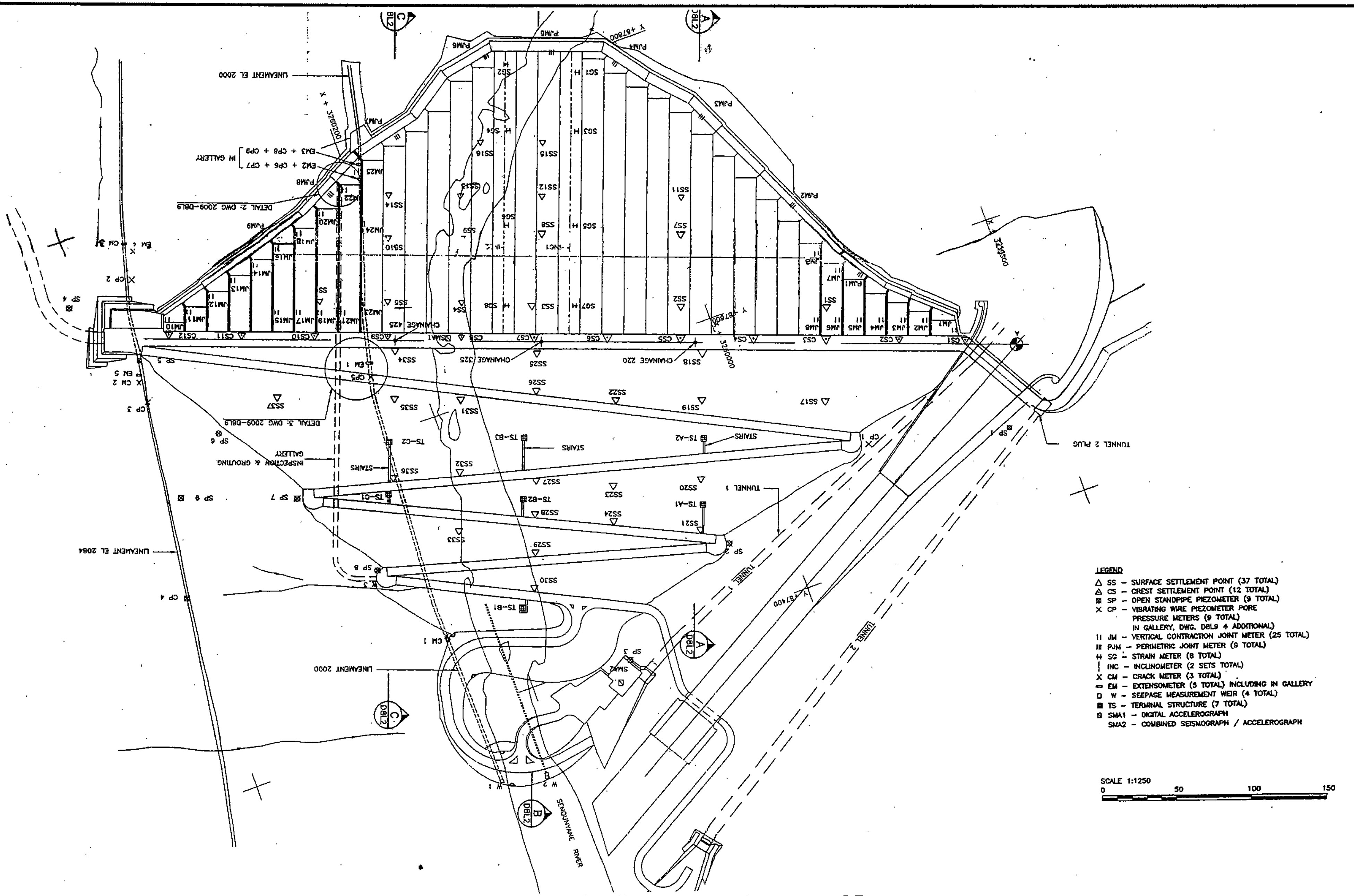
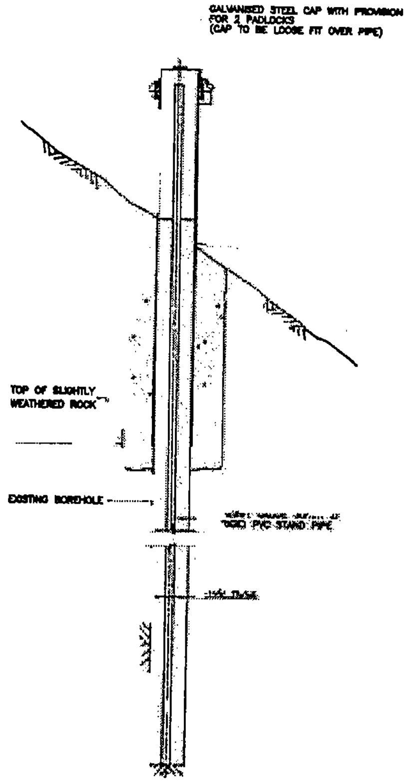


Figure 7.6-1: Instrumentation general Layout



STANDPIPE PIEZOMETER
SCALE 1:100

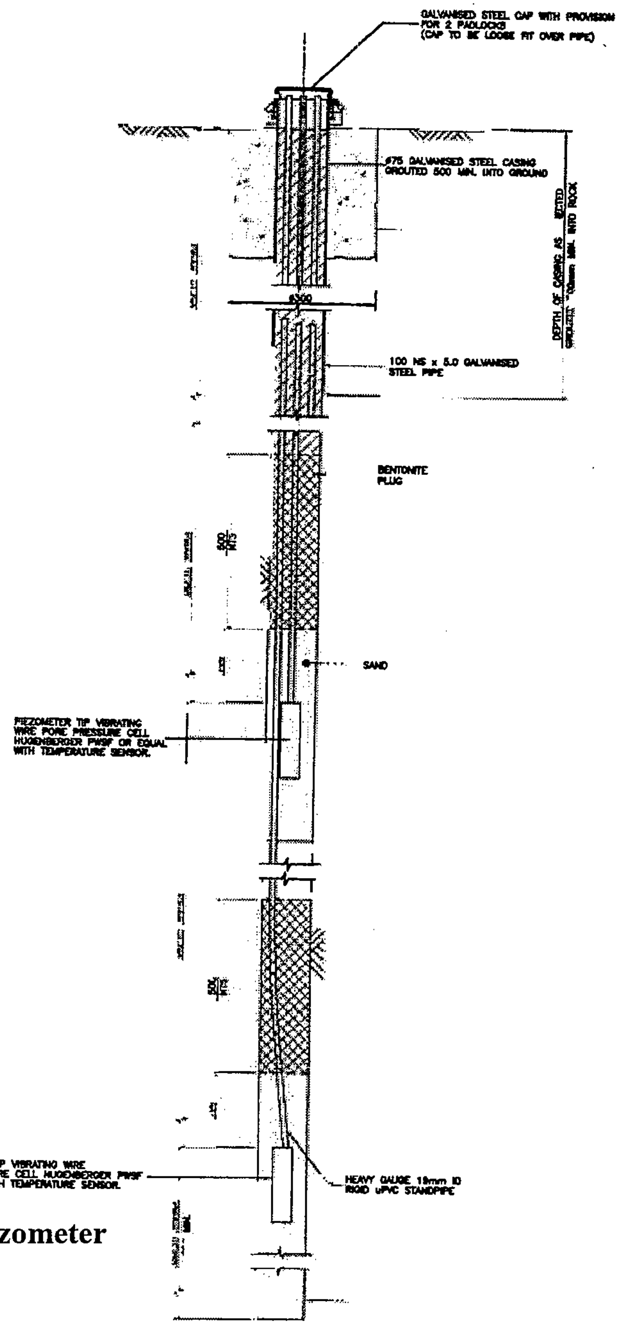


Figure 7.6-2: A Vibrating Wire Piezometer

Table 7.6-1: Instrumentation Reading Intervals

INSTRUMENTATION	READING INTERVAL
Hydraulic settlement gauges & load cells	weekly
Crest and surface settlement points	monthly
Thermometers	daily
Open standpipe piezometers	weekly
Vibrating wire piezometers	weekly
Pore pressure cells	weekly
Seepage measuring weirs	daily
Strain meters	weekly
No stress strain meters	weekly
Joint meters and crack monitoring pins	weekly
Perimetric joint meters	weekly
Inclinometers	weekly
Extensometers	weekly
Rainfall	daily
Reservoir level	daily

7.6.13 Water Level Recorders

Pressure type water level recorders have been required to measure the pressure at. Tunnel 1 plug. Information shall be relayed to the Compensation Outlet Control House. Reservoir water level recordings and gauge plates have been included in the Compensation Intake Structure.

7.6.14 Data Loggers

Data loggers for storing and reading of the following has been provided:

- Vibrating wire piezometers
- Perimetric joint meters
- Joint meters
- Strain meters
- Load cells
- Extensometers in tunnels
- Pore pressure

The data loggers have been provided for instruments, which are not accessible and for instrumentation in the tunnels for monitoring movement of the lineaments. Software has been provided to receive and record readings. It is necessary to provide continuous electric power to the data loggers to ensure constant storage of data. It has been arranged to provide data logger boxes below some of the switch boxes in the parapet wall on the crest of the dam and to join data logging of instruments to the capacity of the provided data loggers (say 40 instruments to one data logger).

7.7 Discussion and Conclusion

Several site visits were undertaken and discussions were held with the instrumentation team during the construction of the Mohale Dam. During these visits, all instrumentation sites were checked with the aim to record and monitor the installation progress. During the installation, engineering and client representatives were asked to attend so that they could monitor and witness the installation by the contractor. The instruments were checked according to specifications and tested accordingly. When all the parties have been satisfied, installation carried on as intended. Few delays have been experienced due to late deliveries from external suppliers. In spite of the delay problems the Mohale Dam has been well instrumented and its personnel is being trained on a continuous basis by experts. The instruments have been timeously installed and are up and running as required under the terms of the contract. Inspection of the whole setup has been done as required in the specification. Daily readings are taken from all installed instruments (Table 7.6-1).

In conformity with normal international practice the Specification required that the Contractor provides the services of the manufacturer's representative for the respective instruments to train the Contractor's, the Engineer's and Employer's personnel in the installation, calibration, testing, reading, operation and maintenance of the instruments.

8 DAMS AND GEOLOGY/GEOTECHNICAL INVESTIGATION PURPOSE

The investigations focus on the following very important issues:

- The geology of the dam site where the foundation will be sited
- The geology of the dam foundation
- The geology of the spillway, diversion tunnels and outlet works
- Treatment measures in general

The investigations are carried out to determine whether the dam foundation has sufficient strength and durability to support the type of dam to be designed and constructed. The question of water-tightness of the foundation plays a significant role also. The foundation is investigated to find out if it is watertight so that the amount of grout to be used is calculated. It is again very important to decide whether the spillway will need concrete lining or not.

The investigations also focus on the following:

- The geology of the basin to determine whether the basin is watertight.
- To determine geologically whether there is a possibility of landslides into the reservoir, that may cause overtopping of the dam
- Exploration for construction materials necessary to complete the building of the dam

In the design of embankment dams there are two major decisions, which have to be taken, both of which depend on geological factors:

- The extent to which it will be necessary to provide concrete lining and /or energy dissipation structures in the dam spillway, and
- The extent to which the spillway excavation will be able to supply fill for use in the construction of the dam embankment.

The geology of the spillway is therefore very important to the overall design and layout of the whole construction project (Dr. Woodward, 2002).

All these investigations are very important with a view to evaluating the possible environmental impact and in preparing an environmental management plan.

8.1 The Environmental Impact of Large Dams

There are, of course, negative and positive impacts when the relationship of the river and the land is broken. Usually the positives should outweigh the negatives by large margins. The critics of large dams consider only minor negatives without considering the impacts as a whole. They are perhaps not aware of the positives or on the other hand, they are interested in exaggerating the negatives so that they can pursue their agendas against the construction of large dams. However they do recognise that a river is the product of the land it inhabits, and the type of rock and soils as well as the shape of the land and the amount of vegetation are some of the factors that determine the river's shape, size and flow. This acknowledges the role of geological sciences in determining and perhaps predicting and providing lasting solutions for the mitigation of adverse impacts. This can also help in enhancing the positive impacts. This study makes use of all the geotechnical findings to address the environmental conditions.

The International Rivers Network (IRN) has written the following concerning the impacts brought about by world large dams:

- The IRN defines a large dam as one higher than 15 metres (taller than a four-storey building).
- Some 40 000 large dams, mostly built in the past 50 years, now obstruct the world's rivers. More than 300 are major dams in terms of height (=150 m), dam capacity and reservoir volume.
- More than 400 000 km^2, that is, an area larger than Zimbabwe and 13 times the size of Lesotho and almost the area of California have been inundated. This is ca. 0.3% of the world's land area.
- Volta Reservoir of Akasombo covers (8 500 km^2) in Ghana. It is the world largest man-made lake that is ca. 4% of Ghana's territory.
- The United States of America is the second most dammed country after China with 19 000 large dams, followed by former USSR, Japan, and India. Of the 5,500

large dams in the USA, around 50 of these are major dams, followed by the former USSR, Canada and Brazil with 16.

- The rate at which dams are completed has declined from around 1 000 a year from the 1950s to the mid-1970s to around 260 a year during the early 1990s. More than 1 000 large dams have been under construction at the beginning of 1994. In Lesotho, the construction of Phase 1 had commenced in 1994. China, Turkey, South Korea and Japan are currently countries with the largest dams under construction.
- The number of displaced people due to the construction of dams ranges between 30 to 60 million with the majority being in China. The estimated displacement rate is 2 million people a year.
- The majority of displaced people are usually poor farmers and indigenous people. They are made poorer and suffer cultural decline, high rates of sickness and death, and great psychological stress.
- In some cases people are not being compensated or receive meagre compensation, which does not match the value of their lost properties: Cash payments are too inadequate to compensate for loss of land, homes, jobs and business and replacement land for farmers is usually not as fertile as the appropriated land and is smaller in size than the original holdings.
- The statistics of the WCD reveals that roughly 200 dams outside China, which have collapsed or been overtopped during the 20th century, killed more than 13 500 people.
- The Chinese Province of Henan has been hit by a massive typhoon in August 1975 and as a result two dams burst and cost an estimated 80 000 to 230 000 lives.
- People have also died in earthquakes caused by the great weight of water in large reservoirs. A magnitude 6.3-event at Koyna Dam in India killed around 180 people in 1967.
- Large dams have provoked opposition for numerous social, environmental, economic and safety reasons. The main reason for this opposition is the huge numbers of people evicted from their land and homes to make way for reservoirs. People downstream from the dam also suffer in their millions because their livelihoods are adversely affected in the following ways: loss of fisheries, contaminated water, decreased amounts of water, and a reduction in the fertility of farmlands and forests due to the loss of natural fertilisers and irrigation in seasonal

floods. Dams also spread waterborne diseases such as malaria, leishmaniasis and schistosomiasis.

- Opponents of dam construction also believe that the benefits of dams have frequently been deliberately exaggerated and that other measures could provide the services which dams provide by other more efficient and sustainable means.
- The majority of large dams are built for irrigation; almost all major dams are built for hydropower. Dams generate nearly one-fifth of the world 's electricity. Dams also provide flood control, supply water to cities and can assist river navigation. Many dams are multipurpose, providing two or more of the above benefits. Hydroelectricity is cheap to produce once the dams are built.
- Critics of the dams do believe that dams should only be built after all relevant project information has been made public; the claims of project promoters about the economic, environmental and social benefits and costs of projects are verified by independent experts; and when affected people agree that the project should be undertaken.

Regarding the environmental impact in the case of Lesotho, the Transformation Resources Centre (TRC) has written:

- Rivers affected by the 5-dam Lesotho Highlands Water Project (LHWP) could deteriorate to "something akin to waste-drains" if Lesotho delivers as much water to the Republic of South Africa as stipulated in the original treaty, Metsi Consultants Report.
- Katse, Mohale, and Mashai dams will only allow floods on the average of once in a 20-year period (rather than the norm once a year). The Katse Dam affects the Malibamats'o and already the residents living immediately below the Katse Dam report increased numbers of aquatic insect pests, and skin rashes after crossing the low-flowing Malibamats'o River; Mohale affects the Senqunyane; and Mashai affects the Senqu. The Malibamatso and the Senqunyane flow into the Senqu.
- According to the report, the dams' impact "will manifest as strongly deteriorating physical and chemical conditions" and major biological changes. They predict dense algal growth throughout the system, which can be toxic to fish; encroachment of exotic plants (at the expense of native plants and the species that depend on them); moderate to critically severe increases of blackly and other pest

populations which prey on livestock; reductions in most fish populations, with some species like the maloti minnow and trout reaching the point of extinction; decline in waterfowl numbers, and an explosion in rodent populations, which could affect crops along the riverbanks.

- These changes to the ecosystem will have major social impacts. Many fish and wild vegetable species will be reduced by over 50%. Social studies have shown that when species decrease to this extent, communities living near the river no longer make the effort to harvest them (it is often a long, steep hike into and out of the river valley). Therefore, a reduction of 50% is effectively a loss of 100% of these resources to riparian villages. A serious situation given the already low levels of nutrition in these communities. The low flows will increase the level of pollutants in the river, causing a critically severe increase of diarrhoeal diseases such as giardia. The cost of cash compensation for lost resources and mitigation against public and animal health problems via provision of water supply systems, vaccinations, and VIP-latrines reach nearly $4.28 million annually.

The author would like to reproduce the response of the International commission on Irrigation and Drainage (ICID) to the above points from the IRN. The ICID made it clear that dams are very important in the sustainable development of countries of the world, especially the developing countries. Below are their invaluable comments on the issue of large dams:

- The analysis of existing dams is unbalanced with a strong implication that the majority of the world's 45,000 large dams are environmentally damaging or socially destructive. Very little attention is devoted to the many well-known benefits of carefully planned dams, and no feasible alternatives are suggested for meeting the future water, food and energy needs of the developing world;

- The conditions proposed in the 26 guidelines for the planning and implementations of future dams are, in many instances, idealistic but not realistic. They have no doubt been conceived with good intention but have not been verified on their applicability in practice. We feel they will have the effect of preventing or at least seriously delaying, future water, food and energy needs of the developing world;

- The conditions proposed in the 26 guidelines for the planning and implementations of future dams are, in many instances, idealistic but not realistic. They have no doubt been conceived with good intention but have not been verified on their applicability in practice. We feel they will have the effect of preventing, or at least seriously delaying, future water resources projects, which will be urgently needed during this century, particularly in developing countries. There is a serious danger of condemning some third-world countries to a future of "sustainable underdevelopment".

- Water scarcity is engulfing many countries of the world. Thousands of dams are still to be built to store water and make it available, during the first half of this century itself, on a worldwide basis, especially in the non-industrialized countries of Asia, Africa, Latin America and Eastern Europe.

- It would have been highly appreciated if the guidelines as developed by the World Council on Dams (WCD) could be in support of this process and therefore could contribute to the improvement of existing guidelines, standards and criteria in the concerned countries.

- Every human activity modifies the environment. Some changes are for the good, some are not always so, but awareness in societies about size and scope of adverse impacts plays an increasing role in decision-making. Effort is made to mitigate and compensate such effects, while increasing positive impacts so that sustainability of development is maintained and the natural resource base is not eroded. Water resources development is particularly environment friendly and dams mostly protect and enhance the environment. The challenge is to see that the enormous positive effects of dams on the environment far outweigh the negative effects. Concerned professionals have evolved mitigation and enhancement measures, over a period. ICOLD and ICID have developed detailed listings, criteria and guidelines for the study of environmental impacts and their mitigation. Many countries have developed appropriate policies and measures for compensating negative social and environmental impacts. While respecting the privilege of countries/Governments to develop their water resources plans and priorities, it will be only fair to expect that adequate compensatory packages are provided by them to adversely affected people and to ensure that these people are better off, after the project implementation, than before it.

The major general remarks that can be made on the WCD report itself are made in relation to the role of dams for irrigation, drainage and flood control as extracted from different international responses to the WCD report:

- In the WCD report, the benefits of dams are described in passing, almost without quantification, while the negative effects are outlined in detail. This especially plays a role in the analysis on the social issues where almost no attention is given to the role of dams for rural development, while hundreds of millions, if not billions of farmers have benefited tremendously from the revenues of irrigation that could only be realized in conjunction with reservoirs. The same holds true for the environmental benefits and protection against natural disasters, which have almost not been touched upon, while they are in many cases very significant. For example:

- About 1 billion people depend on food produced by reservoir-related irrigation. There is no suggestion of how this food could have been produced by other means;

- The WCD report paints a very negative picture of the health impacts of dams, while the positive aspects are not mentioned. These are, amongst others:

    - Huge benefit of food production;

    - Benefit of having better quality drinking water all year round, even during droughts;

    - Beneficial effect on women and children due to improved accessibility to a safe water source;

    - Reduction of the intensity of floods, which are a major threat to health;

- Even if we interpret the scanty data that is shown in the WCD report in a balanced way by paying proper attention to the advantages, disadvantages and actual results, then one could state that dams have in general very been beneficial so far, that it has been shown that lessons have been learned from the inadequacies of the past. These lessons have been incorporated into the adopted practices. The risks of future occurrences of inadequacies have been significantly reduced. This places the responsibility on those concerned to improve the situation, where improvements can be made;

- It is an illusion to suppose that decisions on development or infrastructure projects will get full and total support in society. They have to be politically acceptable;
- The WCD report advocates a "rights and risks" approach for affected people only. This approach as well as other proposals related to affected people has to extend to the "would-be beneficiaries" of a dam project as well. They normally number 100 times more than those affected and their "rights and risks" are similar to those affected people;
- The paragraph on irrigation in Chapter 2 of the report shows very positive results. These become even more positive when they are compared with the "no project" alternative or when determined against the lifespan of the concerned dam. The negative impression that is given in the WCD report results from the comparison between data in the feasibility studies with actual data, mainly valid for the initial period after construction of a new scheme. Under that condition the remarks are valid, but this is not so relevant when we look closely at the relatively high level of the actual performance figures;
- Most of the dam projects not only have the objective of providing economic benefits, but they also address the overall socio-economic development of the people of the region. The dam projects often help prevent the migration of rural people to the cities, while giving them a higher standard of living in their native areas. This is particularly true of areas with an arid, or semi-arid climate from where seasonal forced migration occurs;
- The developing nations of the world are mostly located in arid and semi-arid areas. Rapidly increasing populations and the relatively weak economies of these countries cause the degradation of ecosystems. In several cases, dams are instrumental in preventing, or at least reducing such degradation;
- The ratios of numbers of beneficiaries to those adversely affected are not identified in the examples in the WCD report. For example, in the case of the Sardar Sarovar Project in India - often quoted in the report - the Supreme Court of India has stated in its verdict that the direct ratio in this project is 100. If the multiplier effect is considered, it would rise to 200;

- If the WCD had examined the widespread and serious impact of drought on socio-economic life in rural areas, it would have comprehended the significance of drought proofing and employment generation in the farm sector through irrigation. The ravages brought about by repeated droughts on the rural economy also need to be acknowledged. The WCD should have recognized the widespread chronic under employment, and migration of rural labourers particularly of tribal and weaker sections to far away places, in search of a livelihood because of failure of the harvest and hence loss of employment. Had they recognized these maladies, then possibly the social dimensions of the positive impact of large dams in arid regions of the world would have been appreciated by the WCD;

- In the WCD report, it is stated that single purpose dams are more beneficial than multipurpose dams. This is generally simply incorrect when compared with the umpteen cases of multipurpose dams, which have proven to be necessary and optimal in water development for competing needs. Based on this erroneous comparison, effort is made to malign irrigation and drinking water sector dams. The consumable nature of irrigation water use in contrast to the generally non-consumable nature of other sectors is neither understood, nor recognized in the WCD report;

- Basically the WCD report proposes five options as alternatives to large dam irrigation:

    1. Improving performance and productivity of existing systems;
    2. Improving the productivity and livelihood opportunities offered;
    3. Alternative supply through enhancing rain-fed agriculture;
    4. Investing in conventional supply side measures to develop new irrigation areas based on direct abstraction from rivers and groundwater;
    5. Food importation

In this respect, it may be useful to clarify that most of the required duplication in agricultural production during the forthcoming 25 years will have to come from existing cultivated land, among others by improved irrigation and drainage. Support and incentives for rain-fed farming are considered to be of importance for poverty alleviation, but it is not correct to assume that rain-fed farming without an irrigated water management system will significantly contribute to the required increase in food production. As indicated before it will be unavoidable to increase storages. The remark on food import is an interesting one, especially relating to developing countries. One has to realize that the number of food exporting countries has significantly decreased during the past years. If this process continues as is expected, then it may be expected that food prices will eventually rise sharply. This will put a high burden on the concerned governments to assure sufficient food at affordable prices, especially for the poor in their countries. It would have been better if the WCD had studied this aspect in some detail before formulating this recommendation.

8.2 Summary

ICID consider the WCD report simply as a useful document to generate further discussion, but as inadequate, as it stands to find the required sustainable solutions. ICID do not accept the unbalanced judgment on the role of existing dams. ICID consider the 26 WCD guidelines, as they currently stand unrealistic and in applicable.

The ICID states: "As during the WCD process, our organizations are available to contribute to your work in refining the guidelines and criteria. However, we like to stress that this should also be done in close consultation with those who are really in charge of decision making on dams, and guidelines should not be imposed on them through indirect channels".

The comments and conclusions of ICID expose the weakness of the WCD arguments and showed how much the opponents of large dams ignore the very important issues and irreplaceable benefits of large dams. It is clear that the benefits outnumber the disadvantages created by large dams.

8.3 Dam Accidents

The failure of a large dam is a national catastrophe and it happens with fearful rapidity, and usually with little warning (Thomas 1976). Unfortunately, no one has yet come up with a design for a dam that would show clear signs of impending failure in the way that an elastic structure usually does (Thomas 1976). Many other authors also share this view about dam design. Thomas, 1976, says that we learn from our failures in all phases of life. We cannot afford to adopt this method in the construction of dams. He concludes that many ancient dams failed because of a lack of knowledge on the part of their builders, whereas omission has been a cause of failure in recent years. He emphasises that our task should be to visualise all the factors that could affect the

Below is a summary of Gruner's classification of risk:

45% hydraulic conditions (floods, seepage, piping, and uplift)
30% type of structure and construction (hydraulic fill, seepage through poor concrete, inadequate design)
7% geology
6% environment (frost, ice, earthquake, decay, hostile action)
6% consequences (decay, abandonment, induced earthquake).

Thomas, 1976 concludes that the above percentages have been purposely tabulated in order to emphasise the need for care, particularly in the investigation of hydrology and geology. Since 1965, the International Commission on Large Dams (ICLD) has been engaged in an ongoing study of failures and accidents to Large Dams, which has actually come up with a report prepared in 1973 on 'Lessons from Dam Incidents'. The historical section that has been dealing with the evolution in design and construction of the various types of dam has been taken note of, highlighting major dam failures and reporting dam incidents. In total 466 incidents have been studied, of which about 140 may be described as failures. This report is considered to be basic reading for all engineers.

The table below (table 8.3-1) quoted by Thomas, 1976 from the ICOLD Report highlights the preponderance of incidents with earthfill dams and the high proportion of incidents related to inadequate investigation and design.

Table 8.3-1: Number of Dam Incidents for Different Dam Types

	Number of incidents						
	Arch	Buttress	Gravity	Earthfill	Rockfill	Miscellaneous	Total
Exploration	9	5	6	49	2	1	72
Material	1		2	8			11
Layout		1	4	17	3		25
Design	4	6	13	48	3	2	76
Construction	1	1	2	37	5		41
Operation				5	1		6
Supervision	1	1		3			5
Total	16	14	27	162	14	3	236

The statistics of failures of dams are said to be somewhat alarming according to the studies done by several authors including Thomas, 1976, where a couple of examples have been extracted from the same author. There have often been errors of omission that have had serious consequences, i.e. insufficient study of possible floods, inadequate examination of foundations and inaccurate knowledge of the behaviour under load or saturation of materials of the dam or its foundation, according to Thomas, 1976; and Duncan, 1969. "The cost of failure is immeasurable. Damage to property may be assessable; damage to the environment may be less tangible but no less real, but loss of life is a matter of personal suffering. The engineer must be continually aware of possible consequences. Water can lubricate clay seams, it can adversely affect the physical properties of most rocks, it can act as a colossal hydraulic ram, it can erode and corrode, and it may induce earthquakes. If a dam should fail, water will devastate and destroy", Thomas (1976).

He emphasises that when so much attention is being given to environmental impact studies, it may be opportune to consider the overall risk to the community of any vast storage of water. Can any occurrence of earthquake, sabotage, or structural degeneration lead to disaster? He says if finance is not available for guaranteed security, there may be a case for rejection of the proposal. Cost is of vital importance to the engineer, but security must always be paramount (Thomas, 1976). Innovation in design and construction is to be commended but only after a thorough examination of the consequences of departing from established practice (Thomas, 1976).

The list below presents the statistics of failure extracted from Thomas, (1976) and others (unknown):

- Cedar Reservoir, near Seattle, USA, built in 1914, failed in 1918, due to a buried channel, the small town of Edgewick and other property were destroyed.
- Vaiont Dam, built in 1960 arch dam northern Italy, failed in 1963, huge landslide into reservoir causing overtopping (due to inadequate geological survey), everything in the path of the flood for kilometres downstream was destroyed, the five villages were destroyed and 3000 people were killed.

- Bouzey Dam, built in 1878, failed in 1895 killing 150 people, failed due to design and uplift
- Dale Dyke Dam, United Kingdom, built in 1858 with dam height of 29 m, failed on 11th March, 1864 killing 250 people and destroying 800 houses, failed due to inappropriate construction materials, inadequate compaction and spillway
- Baldwin Hills Reservoir, built in 1951, dam height of 50 m, failed on 14th December, 1963; failed due to geology (fault), foundation (piping) and land subsidence
- St. Francis Dam, USA; built in 1926; 60 m high gravity dam; failed on 12th March, 1928 killing 500 people and caused \$10 million property damage; failed due to foundations, rock weakening on saturation and geology
- Malpasset Dam, France; 60 m high; failed on 2nd December, 1959 killing 400 people and wiping out the town of Frejus; failed due to geology, inadequate investigation and instability of abutment

8.4 Lesotho Highlands Water Project and Environmental Impact

The objectives of the LHWP are listed in section 5.1. The project recognizes that the dams and tunnels to be constructed will be expensive and will occupy a large area. This will definitely interfere with the ecosystems in that area and along the Malibamatso and Senqunyane River and finally, the mighty Senqu River would be affected. The purpose has been to identify and separate the positives from the negatives and to improve on the positives while mitigating the negatives. The study of the environment continues vigorously throughout the construction of the LHWP phases' structures. There are local, international and specialist institutions carrying out their own research for different motives. The LHWP also have employed experts to carry out environmental impacts investigations.

The studies are carried out on the inhabitants of the Mohale catchment basin and downstream areas. The reservoir area and surroundings are put into perspective. The studies encompassed many areas and aspects, for example: biological and aquatic life, diseases, resettlement, and topography of the site, geology as well as the possible meteorological changes that may be caused by the filling of the dam.

For the Mohale Reservoir, the Katse Reservoir is a perfect model since the two sites are located in similar geological conditions. The study has been undertaken with the point of view that each site has its own unique characteristics. The model is still open to questioning because the Katse Dam is a relatively new dam (it is only seven years old). They both lie above 1500 m.a.s.l on average and are outside the tropics. The Mohale Dam is just about 30 km from the Katse Dam as the crow flies. A 32-km long tunnel, which will transfer water to the Katse Reservoir when the level of water at the Katse Reservoir is low, connects the two reservoirs. The tunnel is designed in such a way that the water can flow into the Katse Reservoir and back into the Mohale Reservoir when the Katse Reservoir is full.

The impact of the Katse Reservoir on the environment varies, according to the studies carried out. The amount of fish has increased significantly in the reservoir in terms of numbers and size. There are plenty of fish in the Malibamatso River, which very few people are interested in catching. Foreigners do some sporadic fishing. Clean, clear water exports to the RSA have commenced as well as the production of electricity for the whole of Lesotho. The reservoir is well supervised by the LHDA-Operation and Maintenance Group. Sampling and analysis of reservoir water is carried out regularly and is an on-going operation. The LHDA have hired specialists to monitor the aquatic life of flora and fauna on a day-to-day basis for this purpose.

The reservoir does not flood any mineral deposits as the geological studies have revealed. Sedimentation is not a threat since the valley is covered by thin soils. No climatic change has been observed around the reservoir except low temperatures at night and increased fog intensity during winter. Thanks to the location of the project, there are no tropical water-borne diseases envisaged in these areas. Many life improvements have been brought about in the area and for the country as a whole. The researcher likes to name a few examples:

- Improvement in health care through construction of clinics and introduction to modern medicine, health education and improved sanitation
- Job creation for the local communities, acquiring of skills making it easier for the locals to obtain jobs after the construction phase, improvement of the standard of

living, the mushrooming of modern houses in the area, general improvement of the village services

- Introduction of public transportation, good infrastructure (all weather roads, telecommunication, electricity etc.)
- Improved water supply and sanitation
- Improvement in education facilities (construction of new schools and upgrading of sport grounds)
- Addressing of gender issues (gender awareness)
- Improved tourism potential due to the dam structures and reservoirs
- An infusion of new ideas and skills in the area
- When the project is viewed in the nation's interest, it has more far reaching positives than negatives.

The resettlement of the local communities is undertaken in such a way that the communities themselves choose the places where they wish to be relocated. The relocation has improved in the case of Mohale because the project did not wish to repeat any mistakes that might have been made in the Katse era. However, the communities are still not 100 satisfied with the whole procedure. They feel the authorities are not doing enough to meet the relocation package and the package itself is considered inadequate. Those who are left behind feel that they have been robbed of their land and its natural riches.

Below are some of the lessons learned from the Katse Dam

- The necessity for timely and detailed information on the local communities, the natural environment and the need to ensure full public awareness of the project and participation of local communities in planning and implementation.
- Detailed and extensive studies during the planning phase ahead of the implementation period, which should be intensified during implementation.
- Collective decision making concerning whatever that involves the locals

8.5 Environmental Issues Relating to the Construction of the Mohale Dam

Prior to the construction of the Mohale Dam, Environmental Impact Assessment (EIA) studies have been undertaken in order to enable the LHWP to measure the magnitude of the social and physical impacts likely to occur during construction.

8.5.1 Physical Impact

8.5.1.1 Land Resources

The impacts on the land resources because of dam construction are considered minimal compared to impacts that may result from inundation of the dam. When land is lost, the result is obviously that pressure is put on grazing land and other resources and leads to changing of the whole ecosystem. The change can lead to either deterioration, prosperity or both.

8.5.1.2 Water Resources

Potential impacts will involve the pollution of water from construction activities. Pollution risks will be associated with cement spills, oil spills, and deposition of silt from spoil dumps and from the erosion of valley sides. This can lead to temporary degradation of the water quality through an increase in turbidity. On the other hand the project has through its construction activities brought water near to the communities. The water supplied is purified prior to distribution to the villages.

8.5.1.3 Biological Resources

There are no major impacts likely to occur on the biological resources other than impacts on aquatic biota related to incidences of turbidity in the river. Localised populations of plants will be lost and these will have significance to users. This impact is restricted to the site because water released from the works goes through some cleaning first.

8.5.2 Social Impact

The construction of the dam will have social impacts on the current lifestyles of communities resident in the area. On the other hand, public investment and private enterprise that accompanies a project of this magnitude will open up development opportunities. It seems at this juncture definitely the project will make the locals enjoy employment and other benefits. It is also noted from other projects experience elsewhere that social unhappiness will be experienced.

8.5.2.1 Cultivated Land

A significant extent of arable land and rangeland will be lost and inundated. Forty-four households will lose arable land. This loss has considerable implications given the fact that the people depend on subsistence farming for their existence. Apart from the food and survival value that the land provides, and the accompanying security against possible hardship, agriculture strengthens socio-economic co-operation between households. Villagers form work parties to assist each other in cultivation, they share and exchange crops, and they loan out oxen or labour in return for payment in cash and kind.

8.5.2.2 Livestock

Livestock is an important cultural and economic resource that provides security in the form of wealth (usually sold in a time of emergency); a food source; income from the sale of wool, mohair and other by-products; fuel from dung; draught power in agricultural activities and transport.

Livestock is generally grazed in remote mountain areas in summer (from about November to April) and are brought down to the lower-lying areas close to the villages in winter. The construction of the dam will disrupt free movement of cattle over the summer periods to their place of grazing. Animals are also grazed along riverbanks where the grass is more plentiful and more succulent and with reduced flow in the river during construction, vegetation will be reduced.

8.5.2.3 Natural Resource Use

The basins of the rivers feeding into the dam provide a vast reserve of natural resources. Rivers are used for watering of animals; washing of heavy items of clothing such as blankets, bucket irrigation of vegetable gardens, and recreation (swimming). Rivers are also used for religious purposes such as baptisms.

Other natural resources available include:

- Fuel, such as wood, dung and shrubs/bushes
- Wood for making agricultural equipment
- Building material such as stone, water and sand for brick making and thatching grass
- Medicinal plants
- Reeds for basket making
- Food sources such as fishing, wild vegetables, fruit, berries, and cooking herbs

8.5.3 Impact Management

8.5.3.1 Social

8.5.3.1.1 Population Pressure

There is increased pressure due to large numbers of people moving into the area, namely, work seekers, the labour force and those taking advantage of entrepreneurial opportunities in the short term with possible long-term settlement. The ratio of people to land due to the influx of people and the loss of land from construction works will be high.

The LHDA, in consultation with the Regional Development Sector and the Government and involved communities, will implement and monitor development plans.

8.5.3.1.2 Resettlement

Resettlement of villages because of inundation is inevitable. Implications for host villages should be taken seriously. This includes monitoring closely the social disruptions in a new environment. This also constitutes psychological counseling of both the host villagers and the resettler. Consultation with the community regarding impacts and options to enable informed decision-making and implementation of resettlement plans is in progress.

8.5.3.1.3 Social Characteristics

The influx of relatively young and single males transforms the population profile. Particularly as many will gain high status, as money, earners resulting in greater promiscuity manifested in the increased transmission of sexually transmitted diseases, STDs, a growing demand for commercial sex workers, and the higher potential for unwanted pregnancies. There will be changes in the family unit structure in the long-term with more single parents resulting from the transient nature of the situation and men increasingly neglecting their responsibilities. The LHDA, in consultation with sociologists and epidemiologists have studied these issues.

The provision of health services, adequate recreational facilities, adequate accommodation and regular transport to and from home has been implemented.

8.5.3.2 Physical

8.5.3.2.1 Land Resources

The loss of natural resources to the communities will occur in stages, as the construction work progresses. The loss of arable and grazing land will result in the loss of food, security, destruction of crop production, grazing pressure on existing land particularly in the winter season, disruption of livestock movement patterns and natural depletion of stock in response to limited feed.

The change in land-use patterns resulting in a decrease in subsistence agricultural activities, residential stands evacuated for resettlement and development of formal and informal settlements will occur. Rural Development Plans in association with broader Government regional development plans as well as range management plans will have to be integrated and implemented.

8.5.3.2.2 Water Resources

Even though after inundation communities will have access to a large volume of water, but access to existing springs, streams and rivers will affect usage such as watering of livestock, washing of heavy items of clothing and bucket irrigation of vegetable gardens.

Water quality will also be affected by construction with the possibility of polluting rivers downstream. Specified environmental mitigation measures identified during the environmental impact assessment will be implemented.

8.5.3.2.3 Natural Resources

There will be a loss of natural food products such as herbs, fuel/energy sources such as wood, brushwood, medicinal plants, fruit trees, orchards, building, fencing and craft material such as grasses, wood and stones, and fish in the river stream base. Compensation of resources as per the Client's compensation policy will be offered after identifying the positioning of these resources within the project area with local input. Removal, transplantation, breeding and reintroduction programmes are being implemented for endangered species.

8.5.3.2.4 Cultural Resources

Gravesites and historical sites such as cattle posts will be relocated through consultation with communities. Compensation will be implemented according to the Client's compensation policy.

8.5.4 Conclusion

The project commenced with the carrying out of the environmental impact assessment (EIA). As part of EIA, environmental management plans and environmental specifications have been formulated for implementation during the construction of the dam and appurtenant structures. This has ensured adherence to specified environmental standards, and the minimisation, and mitigation of environmental impacts.

The creation of the Mohale Reservoir will regulate floodwaters and bring about changes in the natural environment, in plants, animals, insects, humans and other living things, in the atmosphere and in the biophysical chemistry of the water. These environmental changes will create negative as well as positive impacts on the lives of the communities living around and downstream from the reservoir.

The environmental data gathered before and during the construction made it possible to plan mitigation for these impacts.

9 DISCUSSION

Geological factors have obviously determined the location of the engineering works, the design, and construction, and subsequently determined the maintenance procedures of the Mohale Dam. The details of the geological-geotechnical investigations for the Mohale Dam obtained during the site exploration programme, have been given in the tender and other geotechnical documents. It is well known that the construction of a dam involves a great degree of risk, in terms of geological conditions and their environmental impact.

There are unknown factors to be encountered; hence, every effort is made to reduce such unknowns through sufficient empirically based information. That is why the location and planning of the Mohale Dam have involved massive site investigation that starts with the feasibility, geotechnical and planning stages. It involves extensive geological mapping of the catchment area using aerial photos, and field observations to find and prove observation from aerial photos. Geophysical surveys are also carried out to locate subsurface features in the area, especially along the foundation and plinth. A diamond drilling programme during which over a thousand meters of core drilling have been completed, is carried out to determine the subsurface geology of the dam foundation and to locate aggregate material for dam and appurtenant construction. Borehole tests are conducted to determine the fractured condition of the rock and lugeon tests are conducted. The investigations continue throughout the construction stage. They are continuing into the operation and maintenance stage.

This work addresses geological conditions of the LHWP in order to map out the environmental impact of the Mohale Dam. It attempts to demonstrate the invaluable importance of geological investigations in dam construction, in terms of site selection, investigation, construction materials, building, and environmental impact. It also touches upon the topic of dam failures because that topic is of particular concern in site investigations. The failure of a large dam is known to have more far-reaching, disastrous consequences than any other man-made structure. This is because flood waves have enormous destructive power when there is a collapse of the dam wall. In the event of the Mohale Dam failure caused by an earthquake or any other activity, the downstream settlements could be devastated by floods from the Mohale Reservoir.

The effects of failure would be experienced across the borders into the Atlantic Ocean. Loss of income, environmental destruction and infrastructure and loss of life would be the consequences. From the feasibility study to the design and completion, the hazard potential of the dam has been scrutinized in detail.

The most common causes of dam failures addressed in this thesis are:

- Embankment dams overtopping due to inadequate spillway discharge capacity to give way to flood waters. The problem here is believed to lie solely with designers and not with geology. This is one of the common failures and any embankment dam will definitely fail if the spillway cannot cope with the amount of water flowing through it.
- Construction errors such as inadequate compaction of fill or use of contaminated materials. Materials should therefore be tested continually during the construction phase. Finding and extraction of quality materials is very important. As for the Mohale Dam two quarries have been used; quarry 1 to produce aggregate for all concrete works and quarry 2 to produce rockfill.
- Geology of the dam foundation. The foundation should be adequately mapped and then assigned the correct treatment. The foundation of Mohale Dam has been treated by drilling and grouting. It is found through investigations that the basalt rock of the Lesotho Formation forms excellent foundations due to its strength and durability. Two lineaments on the left bank have been mapped and treated. They have been also instrumented to monitor their behavior.
- The reservoir slopes have been mapped in order to identify places of possible instability. It is found that the basalt of Lesotho formation forms excellent storage because of few faults and non-continual connected joints. The strength of basalt is within acceptable limits.

Earthquakes are being investigated and a total of about 100-recorded earthquakes have occurred within a 150-km radius of the Mohale dam site. One of these has an M 5.5 event; five events have M 4.5 to M 5.0 and the remainder range from M 2.0 to M 4.5.

- The largest earthquake recorded within 150 km of the dam site has the M 5.5 Zastron earthquake, which has occurred about 143 km south-west of the project area in 1957. Large events of M 6.0 and M 5.9 have occurred further west at Koffiefontein in 1912 and 1976 respectively. These latter events may have been mining induced, but they are indicative of crustal stresses and earthquake capability in the region.

- Three recorded earthquake epicentres and those at Katse Dam have occurred within a 30 km radius of the Mohale Dam site:
    1.) M 3.8 on February 10, 1958; 23 km NE of the site,
    2.) M 4.2 on February 5, 1971; 13 km S of the site,
    3.) M 3.6 on May 3, 1976; 24 km from the site.
    4.) Seismic situation from The Katse Dam forms a credible model for the Mohale dam.

Earthquake focal depths vary throughout the region from 0 km to more than 60 km. The majority of focal depths are, however, less than 33 km in depth. These are termed shallow earthquakes whilst focal depths greater than 33 km are referred to as deep earthquakes.

The distribution of historical earthquake epicentres in Lesotho indicates that the majority of earthquakes occur in the southwest half of the country, including the Mohale site area. Future ongoing studies are necessary to delineate precise seismic zoning. To date, however, no clear pattern of earthquakes has been proven to exist in the Mohale area.

Reservoir induced earthquakes have happened at the Katse Reservoir within the first three months of filling possibly due to the fact that there has been high rainfall, which resulted in a rapid rate of filling.

The environmental issues are discussed and comparison has been drawn with other projects elsewhere of a similar magnitude. The environmental management plan (EMP) has been discussed and is in place.

Data-gathering methods are as follows:

- Reading and scanning of available documentation followed by extracting of relevant data,
- Site visits to check on and verify the assembled geological information, with the aim of identifying geological-information defects or malfunctions that could affect the safety of the dam,
- Compiling all the findings stated above on geological conditions and described features, which could impact on the safety of the dam and the environment.
- Site discussion with the technical staff of the engineer and the contractor, as well as conducting interviews with the affected communities.
- The long service involvement in deferent activities of the LHWP, especially in the construction of the Katse Dam, geotechnical supervision of the Mohale Dam and engineering geological activities in the construction of the Mohale Tunnel.

10 SUMMARY AND CONCLUSION

The Mohale Area is located south-east of the Front Range of the Maloti Mountains which are a natural barrier up to 3000 masl. This range can be traced from the southwest to the northeast of Lesotho. Basaltic rocks of the Lesotho Formation underlie the area. The Senqunyane River has deeply incised valleys into the plateaux that extend southwards .from the Malotis at about 2500 masl. The valley sides are steep, but are interrupted by pediments, terraces and cliffs at various elevations. The uppermost geology at Mohale is basalt from the Lesotho Formation, which consists almost entirely of thoeliitic and olivine basalt flows. The basalts are fine to medium grained and range from highly amygdaloidal to doleritic. The linear features such as dolerite dykes and lineaments have been identified as the only water-bearing structures. In general, the weathering of the basalt is shallow. Localised deeper weathering can be observed at and along the lineation and dolerite dykes.

With the exception of the Great Wall of China, dams are the largest structures ever built. Throughout history, big dams have prevented flooding; have irrigated farmland, and generated tremendous amounts of electricity. Without dams, it is commonly viewed that modern life, as we know it, would simply not be the same. Since the first large-scale dam was built in Egypt more than 5,000 years ago, engineers have devised various types of dams to withstand the forces of a raging river.

The main objective of this study is to map out the geological conditions of the Mohale Dam and its environmental impact. The study attempted to show how geological conditions of the Mohale Site have influenced the decision on construction site selection, materials and dam type selection, construction methods and possible environmental influence of the Mohale Dam.

The influence of the geological site conditions on the planning, design and construction, and environmental impact, and hence the significant cost of these structures has been discussed throughout this study. The most important influencing factors include the topography or landform of the region, which is controlled by the rock type and geological structures, the geotechnical properties of the geological materials and the availability of construction materials. The study has concentrated on the geological mapping of the dam site and adjacent potential rock quarries and soil borrow areas as well as the treatment of the dam foundation. Airphoto interpretation has been employed and so has been the detailed field mapping of hard rock exposures of the predominantly horizontally bedded basalt flows. The design parameters have been looked into closely as is demonstrated in Chapter 7. Chapter 5 has dealt closely with the hydrological parameters, which are of paramount importance in the design of large dams for safety and environmental reasons.

Dams have made an important and significant contribution to human development, and the benefits derived from them have been considerable. Dams have been in the past associated with high food and hydropower production. As much as there are opponents of dams, there are engineers who are for dam construction. Dams have also caused devastating damages in history and continually posing a thread to the human lives and natural environment.

The researcher is of the opinion that people should not pray for good rains and when those rains have fallen standby happily watching the water running into the sea. When geological conditions are favourable for building a dam and the environmental investigations yield positive findings, and the capital is available. The construction project should go ahead. Dams are also recreational and tourist centers of increasing importance. Dams somehow have an element of beauty, which is the case with the Mohale Dam. It has added some beauty to the landscape with its massive concrete face rockfill wall.

In conclusion, the geotechnical studies, which have been undertaken during the various stages of the feasibility studies, actually give a general account of the nature and properties of the basalt formation in Lesotho and identify a dam site. The geological mapping has made extensive use of aerial photographs to make interpretations and a detailed geology of hard rock exposures. The studies of the Mohale Dam have concentrated on the geological mapping of the dam site and adjacent potential rock quarries and soil borrow areas. The basalt of Lesotho has been proven to provide excellent materials for construction of dams and the valleys they form are suitable for dam construction. The dam wall has been constructed on sound foundations. Construction parameters have been calculated in order to build the dam on sound foundations and make the necessary foundation treatments. The dam has been well instrumented according to specification. Natural construction materials have been sourced within a short distance, which have ensured very little environmental disturbance, in terms of air pollution in general. There is very little possibility of breaching of the dam walls that may happen due to geology, as the geological conditions are good. The study has demonstrated the cardinal role geology played in determining all the vital parameters necessary for the sound construction of a dam. Two lineaments have been geologically identified and investigated and then accordingly treated.

The environmental impact is being dealt with throughout all the project stages up to completion. This has permitted the orderly development of the project phases. The investigations have shown that some negative impacts will occur. The impact has been accordingly well documented and the project has been equipped to put suitable structures in place for mitigation. Local people have been somehow appropriately compensated. The area has been furnished with medical centres, roads and bridges; new schools, shopping centers, clean water, agricultural centres and new skills have been introduced. The communities have been infected with new diseases. Grazing land has been reduced or occupied by the Dam wall and the reservoir behind it. Crime and prostitution have been introduced. Although deaths are on the increase due to acquired immune deficiency syndrome (AIDS), there is a population growth due to the high influx of people and therefore high pregnancy rates among teenagers. Some families have broken down while new ones have been formed. Some working areas around the reservoir have encouraged increase in soil erosion.

In order to obtain a structure that actually meets the specified criteria it is of cardinal importance to integrate the components of the project, that is the need, the planning, the effects, finance, investigation, design, construction, operation, surveillance, maintenance, and future alteration. The geological and environmental conditions have been demonstrated to be suitable, hence fulfilling the specified criteria.

11 CONCLUSION

To design, construct, and operate and maintain safely and economically, it is important to know the geology of the site thoroughly. In this study a quantitative and qualitative approach to assist in the use of the geological conditions to predict or to map out the environmental impact of the Mohale Dam is presented. It employs the construction procedure as quantitative characterization factors. The approach is built on the linkage between descriptive geological terms and measurable field parameters such as rock strengths and others. Dams constructions make use of geological materials, which could be in the form of sound rock or loose soil. The influence of the geological site conditions on the planning, design and construction, and hence the cost, of these structures is considerable. The most important influencing factors include the topography or landform of the region, which is controlled by the rock type and geological structure, the geotechnical properties of the geological materials and the availability of construction materials.

Site investigation expertise lies in the following:

- Dam sites and related works, such as pumpstations
- Dam-safety investigations
- Tunnel alignments
- Construction-material investigations (coarse aggregate, rip-rap)
- Stability of rock slopes
- Environmental Impacts

The geological conditions of the Mohale Dam have been to a large degree thoroughly investigated in the different geotechnical studies of the Mohale Dam and Tunnel. The studies have covered intensively a wide range of geological issues involving the construction site, reservoir site and dam foundation with a view to mapping out the environmental impact. Based on all geological and geotechnical results, and design parameters discussed in this work, it is concluded that the foundation is sound. The two lineaments crossing on the left flank of the dam have been treated adequately. Seismicity is on the basis of the discussions in this study negligible. The valley with its steep-sided slopes satisfy the requirements, therefore is concluded to be suitable for this type of dam. The construction materials used have been easily obtainable and are of good construction quality. The dam is considered well monitored on the basis of the state of the art instrumentation. Instrumentation is required to monitor the behaviour, performance and safety of the dam during construction, initial reservoir filling and operation, and to check the design assumptions. The environmental management plan is in place. Therefore, the Mohale Dam is deemed safe (CFRD are safest to build; there is no incidence of failure of CFRDs that has ever been reported anywhere in the world). The flow contacts between flows are closed or filled with clayey material, which render the basalt impervious. The basalt forms, as the result of its strength and imperviousness excellent reservoir and foundation sites. There is no significant leakage anticipated at the dam and reservoir.

The Katse Dam has been an excellent model for the Mohale Dam predictions. The Katse Dam has been well documented in terms of RIS and the researcher presents data that are more recent and other observations relevant to RIS. The main shock of Magnitude 3.3 at Katse is not among the largest in the world such as the 6.1 at Xingjiang Reservoir China, Kariba and Koyna India. Prior to the construction of the Katse Dam, Malibamats'o and Bokong, valleys were considered aseismic. There has never been a seismic event ever experienced by the communities in and around the valley. Thus, a sudden spurt of seismic activity in the areas around the reservoir following impoundment is considered to be due to the water load of the lake. The Mohale Dam has been entirely constructed in the Lesotho Formation, as is the case with the Katse. The area is also regarded as seismically quiet. The experience from Katse suggests that moderate RIS can be anticipated in the Mohale Reservoir area, especially upstream from the dam wall. According to Skemtons Effect, the temporal association of RIS with filling showed that in some cases shallow, small earthquakes are associated with reservoir impoundment and the linear elements as has been the case in the Katse Reservoir. Certainly all reservoirs discussed in this work have triggered seismic activity during impoundment. Some seismic events have been induced immediately after filling and others have been delayed. The following mechanisms are believed to play a major role in triggering seismic activity:

- Geology, in particular structural geology of the area
- Filling of the reservoir particularly the rate of filling
- Drawdown rates

- The column of water in the reservoir

Phase 2 reflects the continuity of the LHWP, which is based on the economics of Lesotho. The project has displayed sound social and environmental principles, which are highly influenced by the geology of the area. The environmental issues are dealt with throughout the Phase 1 project. The project recognizes the environmental impact and puts some mitigative measures in place. However, rescue management plans are still lacking. The reservoir has separated villages and families. The provision of bridges does not meet the required access needs to cross the reservoirs. To this effect, self-propelled boats are required to transport the affected communities and the cost of running that service is to be borne by the LHWP. The experience and lessons from managing the environmental impact of Phase 1 is an invaluable asset to guide the LHWP in dealing with further development phases. This will ensure that the project is implemented in an environmentally sensitive and sustainable manner. Most important in this regard is the necessity for well detailed and timeously released information to the local communities about the communities themselves, their physical environment, and their participation and public awareness of projects of this nature and magnitude.

From a rock engineering and slope stability point of view, the rock mass consists of sub-horizontally layered rock cut by steeply inclined to vertical tectonic joints and fracture lineaments. Three tectonic joint sets with orientations of E-W, NE-SW and NNW-SSE have been consistently found across the site. Joints are of medium to high persistence (3 to 20m), widely to very widely spaced and sub-vertical to vertical. Every geological/structural feature on its own represents a body that may impact on slope stability. Even though their presence most likely reduces the stability of the slopes, they are often infilled with zeolite, calcite or quartz and features are generally healed. Infilling thickness generally varies from 1 to 5 mm. The right bank at the dam site is traversed by a series of E-W trending veined fractured lineaments which are generally very closely jointed and cemented crushed zones with zeolite infilling and occasional calcite and silica infilling. The risk of huge slope failure similar to the Vaiont dam case in Italy is considered minimal however, where two groups of thrusts intersect each other, water seepage and intensely fractured/jointed wedges of blocks are present, and then this may increase the risk of localised instability in such areas the slope.

There is no consistent hydrogeological pattern for the site. This is evident from the spring flow data and the Mohale Tunnel water inflows as have been observed from km 1 to 16. The ground water regime as can be concluded from the Mohale tunnel inflows and the spring flows, is influenced by the sub-vertical to vertical joints. The ground water flow regime appears to consist of a number of aquifers, that is a shallow aquifer in weathered, open surface joints and, at depth, a series of perched water tables associated with specific discontinuities, such as open flow contacts, veined lineaments and contacts.

Boreholes drilled in the river area (LDR 103, LDR 205, LDR 206, LDR 217) have encountered artesian flow, associated with partially open and zeolite filled vuggy joints or fracture zones. The artesian flow from LDR 103 (drilled during the Planning Study) appeared to be interrupted after the drilling of borehole LDR 206 drilled during the Tender Design, indicating possible connection between joints and fracture zones. The flows do not appear to be affected by the rise and fall of the river and this has been the case with the spring on the Katse reservoir shoreline. The piezometric measurements at Katse Dam do not show a significant correlation with the reservoir levels. At Mohale the water level in the raised alluvial terrace deposits approximates to that of the river level. This does not rule out the possibility of some linear features to conduct water from the reservoir for significant distances. The water levels in boreholes drilled through the structural features and the lineaments are dependent on the hydrogeological conditions surrounding these features and were found to be variable. For example in the structural lineament feature at EL 2000, artesian water has been found which increases after heavy rains. In the lineament at EL 2080, the water level has been recorded at 11 m in borehole LDR 07 during the Feasibility Study and has been found at a depth of about 31 m in borehole LDR 210 during the Tender Design. An intermittent spring occurs where this lineament intersects a gully near the downstream toe of the dam.

The rock core from boreholes and mapping of the rock indicate that the basaltic rock is generally tight and treatable by normal cement grouting. Water acceptance testing in the plinth and dam area during the various investigations has shown a consistent low water intake on the upper left abutment. Some secondary permeability resulting from joints in the doleritic basalt may result in water seepage and inflows. The ground water levels in boreholes drilled above the tunnel are variable ranging from a depth of 51 m in boreholes LDR 101 to 5.3 m in LSR 202. Perched water levels are present in fracture zones.

The study describes geologically and environmentally impact assessment methodology while carrying out the impact assessment and discussions associated with the Mohale Dam. Finally this impact assessment at the Mohale Dam is considered not sufficient due to time constrains and lack of resources to carry out tests as well as the difficult terrain of the study area. The geological and environmental impact study should be a continuous exercise.

Each dam must be individually viewed, not as one size fits all. In that, way one gets to understand the pros and the cons of the dams. A balance must be struck between the engineering and the environment and geology is there to make this balance possible. The success then of a dam is determined by how well it serves the purposes for which it was constructed without causing undue environmental harm.

12 RECOMMENDATIONS

The following topics are considered to be very important and need to be investigated further so that the status of all structures is well known to avoid disaster and that all early warning systems are kept in check continuously:

- Evaluation of hydraulic structures of LHWP for stability and serviceability.
- Evaluation of a structure's life expectancy and maintenance programmes.
- Evaluation of environmental impact after the completion of the project and continuity.
- Early warning systems are the seismic networks of the LHWP, other networks, animals and people's unusual behaviour and change in known geological structures.

It is of utmost importance to put in place a good early warning system. The system will allow appropriate steps to be implemented timeously should any hazard be eminent. To have such a system the LHWP is bound to have a good understanding of the structural geology and water movement, monitoring and regular inspection along the shoreline, good history about the slope failure in basalt and a continuous slope and dam surveillance. It is acknowledged, in this work, that some of these systems are already in place.

Intensification of environmental studies must be put in place during implementation of the project through gathering information during construction and impoundment. This will assist in formulating mitigative management plans and help in immediately identifying changes in the ecosystem.

There is unhappiness amongst the relocated communities and those who have been left behind in the Katse and Mohale areas. They feel they have been robbed of their land and belongings and that little is being done to correct the situation. Hence, authorities should improve the quality of life and speed up service delivery to these communities. The client should also educate its officials in terms of dealing with relocation prior to the commencement of construction.

Since the dam/reservoir continually/permanently poses a thread to the environment and communities around and downstream, the dam wall should be monitored around the clock. The communities should be made aware of the natural catastrophic situation that they may be exposed to. They should be given lessons in first aid, learn how to swim and rescue drowning people. They should be taught how to use the reservoir for their benefit and at the same time be made aware of protecting it from pollution.

Sustainable poverty alleviation schemes can be put in place such as fisheries, whereby fish is systematical being planted and like wise be harvested firstly for domestic consumption and surpluses for inland market. Quite a number of sustainable jobs can be created through tourism that is building of holiday resorts and sport facilities in the vicinity of the reservoir. The communities should be taught hospitality skills to deal with the tourists who would be visiting the surrounding areas. The communities should be made to feel that the project is theirs, so that they become proactive in originating sustainable projects. They should be aware that the LHWP is for their benefit and it actually belongs to them as much as it belongs to every citizen of Lesotho. Continuing education about the project is very important and it should include positive and negative impacts brought by the construction of the dam. Reservoir patrols may be put in place and if so, should be made up of the members of the communities. Such patrols should be trained in rescue procedures, laws of the country regarding environmental management and laws dealing with transgressors. Finally, a continuous environmental impact study is of utmost importance to the success of the project and everyone involved.

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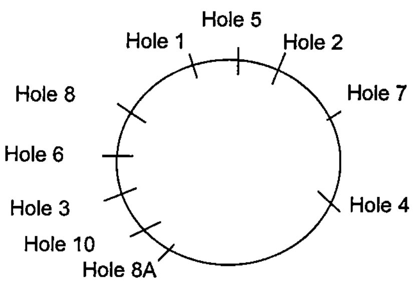
Feature Summary of the Mohale Tunnel from the Intake up to Ch. 16 000

Face Chainage	Feature No.	Water Noticed at Rod No.	Chanage		Feature Length (meters)	Event Duration (days)	Date Range		Distance To next Feature (m)	Time To next Feature(days)	Quantities and Durations				
			Start	End			Start	End			OPC (tonnes)	RHPC (tonnes)	Bentonite (tonnes)	Drilling (hours)	Grouting (hours)
5858.2	1	7	5862	5880	18	13	08/05/2000	21/05/2000	38	2	50.30	150.75	5.37	21.50	132.59
5885.0	2	19	5918	5930	12	6	23/05/2000	29/05/2000	3	1	8.40	74.80	2.37	19.02	49.03
5910.0	3	15	5933	5963	30	5	30/05/2000	04/06/2000	1786	101	0.00	49.95	1.50	23.68	30.73
7740.0	4	9	7749	7764	15	3	13/09/2000	16/09/2000	26	4	2.50	71.15	2.14	9.00	40.62
7789.3	5	5	7790	7813	23	5	20/09/2000	25/09/2000	2267	129	0.00	91.00	2.45	26.10	51.08
10004.6	6a	37	10080	10105	25	1	01/02/2001	02/02/2001	0	5	2.3	10.1	0.372	9.38	8.43
10057.9	6b	25	10105	10108	3	6	07/02/2001	13/02/2001	0	1	67.2	84.1	4.539	16.00	92.12
10084.6	6c	15	10108	10138	30	8	14/02/2001	22/02/2001	0	12	66.4	138.5	6.1935	11.58	133.72
10125.4	7a	10	10138	10140	2	0	06/03/2001	06/03/2001	0	1	0	132.5	0.336	4.433333	81.35
10125.4	7b	11	10140	10153	13	12	07/03/2001	19/03/2001	-8	1	48.8	282.5	0	9.03	180.15
10130.4	7c	11	10145	10167	22	8	20/03/2001	28/03/2001	16	1	2.3	207.3	0	13.08	191.90
10131.8	8	26	10183	10183	0	6	29/03/2001	04/04/2001	2	2	14.95	0.00	0.00	8.87	7.87
10160.4	9	15	10185	10190	5	3	06/04/2001	09/04/2001	21	1	0.00	9.00	0.01	13.25	7.00
10199.4	10	no water: plugged	10211	10211	0	1	10/04/2001	11/04/2001	245	11	0.00	1.30	0.00	7.53	0.50
10330.4	11	57	10456	10456	0	1	22/04/2001	23/04/2001	3	4	0.00	2.90	0.00	4.30	1.17
10430.0	12	17	10459	10478	19	10	27/04/2001	07/05/2001	310	13	0.00	86.70	0.34	10.87	49.77
10788.1	13a	tunnel flooding	10788	10788	0	26	20/05/2001	15/06/2001	72	1					
10788.1	13b	35	10860	10872	12	2	16/06/2001	18/06/2001	528	34	0.00	2.25	0.00	11.88	1.67
11377.6	14	14	11400	11402	2	3	22/07/2001	25/07/2001	1486	74	43.50	8.10	0.00	12.40	24.53
12848.4	15	22	12888	12895	7	6	07/10/2001	13/10/2001	6	3	24.30	0.00	0.91	16.17	28.63
12866.7	16	19	12901	12908	7	5	16/10/2001	21/10/2001			32.25	0.00	1.28	8.25	25.08
					tot.:	130					Totals:				
											363.20	*1402.90*	*27.80*	*256.33*	*1137.94*
											Maxima:				
											67.20	*282.50*	*6.19*	*26.10*	*191.90*

LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B
MOHALE INTAKE FISSURE GROUTING AT FROM FACE CH 5858 (feature No. 1)

Summary of ground treatment activites for lineament feature No.1 at approximate Ch 5870																			
Holes Drilled											Water Struck		Holes Grouted						
Hole	Hole type	Position	Date	shift	Collaring	Drilling*	Rods	End Ch	length	R/cover	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
1-P1	Probe 1	11 o'clock	08-May	m/s	35	25	10	5871.18	13	1.96	1000	14 bar	08-May	25.82	27.4	14.15	0.9895	flow 600-1000l/min
2-P3	additional	1 o'clock	08-May	a/s	14	35	12	5875.98	17.8	2.43	1000	14 bar	10-May	10.25	7	3.6	0.318	flow 600-1000l/min
3-G11	primary	8 o'clock	11-May	a/s	30	32	7	5863.98	5.8	1.18	600	12 bar	16/17 May	22.59	8.6	26.8	0.708	34+regrout 1.4 on 18-May
4-G4	primary	4 o'clock	11-May	n/s	31	239	30	5919.18	61	6.47	220	12 bar	12-May	65.43	5	97	2.76	cement shortage on 14 May - 7 hours lost
5-P2	secondary	12 o'clock	15/17-May	n/s	35	136	24	5904.78	46.6	5.19	dry/333 l/m	-	18-May	3.68	1.4	6.8	0.538	extend from 20 rods on 17 May,drilling time 31mins
6-G12	secondary	9 o'clock	15-May	n/s	75	43	9	5868.78	10.6	1.64	323	13 bar	17-May	No take	0	0	0	No take due to connection to hole 3
7-G2	Sec/Check	2 o'clock	16-May	m/s	40	176	25	5907.18	49	9.16	30	0	17-May	3.18	0	0.6	0.018	Hole plugged - as it is part of 2nd feature
8A-G9	Check	6-7 o'clock	17-May	n/s	-	cannot	0	0	0	0.00	0	0	No need	-	-	-	-	Abandoned due to collaring problems - 50 mins lost
8-G13	Check	10 o'clock	18-May	m/s	35	109	20	5895.18	37	4.17	300	14 bar	19-May	1.19	0.9	1.2	0.03	Hole plugged - as it is part of 2nd feature
9-G12	Check/redrill	9 o'clock	19-May	m/s	9	62	15	5883.18	25	5.20	0	0	19-May	0.22	0	0.3	0.006	Hole plugged - as it is part of 2nd feature
10-G10	Check	7 o'clock	19-May	n/s	25	104	18	5890.38	32.2	6.39	0	0	20-May	0.23	0	0.3	0.006	Hole plugged - as it is part of 2nd feature
																		Assumed distance of probe drill boom from face = 11m
					5.48	16.02	170.00		298.00	43.77	3473.00			132.59	50.30	150.75	5.37	* drilling time until water is struck
					hrs	hrs	No.		m	m	l/min			hrs	tonnes	tonnes	tonnes	



Total grout take = 50.3t + 150.75t = 201.05 tonnes
TBM was halted for this feature on 8 May at 11h00
TBM restarted excavation on 21 May at 14h15
Total TBM downtime = 315.25 hours
Actual drilling and fissure grouting time = 154.09 hours
Total time for packer insertion = 21.58 hours

LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B
MOHALE INTAKE FISSURE GROUTING FROM TBM FACE CH 5885 (feature No. 2)

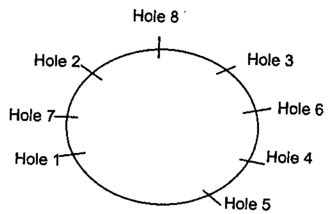
Summary of ground treatment activites for lineament feature No. 2 at approximate Ch.5900

	Holes Drilled										Water Struck		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling*	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
1-G10	primary	7 o'clock	22-May	n/s	40	50	11	5900.4	15.4	2.10	320	12	24-May n/s	557	0	18.6	0.558	connection to hole 2 found
2-G14	primary	10 o'clock	23-May	m/s	60	112	23	5929.2	44.2	8.53	8	0	25-May m/	0	0	0	0	Connection to hole 1 as above -hence no take
3-G1	primary	2 o'clock	23-May	a/s	30	75	18	5916	31	6.35	196	9	25 May n/s	1422	2.8	37.6	1.164	Considerable time lost due to difficulty in packer insertion
4A-G5	primary	4-5 o'clock	24-May	m/s	0	0	0	-	-	-	-	-	-	-	-	-	-	Hole abandoned - due to problems of drill bit being stuck
4-G4	primary	4 o'clock	24-May	a/s	60	121	20	5922	37	4.17	100	10	25-May m/	540	5.6	9	0.438	Water struck at 17th Rod.
5-G7	Sec/Check	5 o'clock	27-May	m/s	11	145	20	5922	37	4.17	dry	-	-	-	-	-	-	Not plugged - reason to be established
6-G2	Sec/Check	2.5 o'clock	27-May	m/s	40	128	25	5934	49	9.32	dry	-	28-May m/	35	-	0.2	0.006	Hole plugged
7-G12	Sec/Check	9 o'clock	27-May	a/s	38	103	19	5919.6	34.6	3.94	440	11	28-May a/s	188	-	6.15	0.132	Hole plugged
8-P2	Sec/Check	11 o'clock	27-May	a/s	30	98	22	5926.8	41.8	4.73	252	10	28-May m/	200	-	3.25	0.072	Hole plugged
					5.15	13.87	158.00		290.00	43.31	1316.00			49.03	8.40	74.80	2.37	
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

* distance ahead of the face " radial cover from tunnel

Assumed distance of probe drill boom from face=11

* drilling time until water is struc



Salient Points

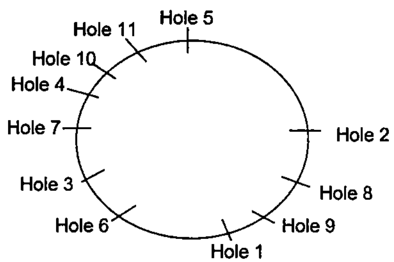
Total Grout take = 8.4t OPC+74.8t RHPC=83.2 tonnes
TBM was halted for this feature on 23-May at 00h30
TBM restarted excavation on 29-May at 13h00
Total TBM downtime = 156.5 hours
Total drilling and fissure grouting time = 68.05 hours
Total time for packer insertion = 27.16 hours
Total time spent on setting up drill rig = 11.93 hours

LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B
MOHALE INTAKE FISSURE GROUTING FROM TBM FACE CH 5910.044 (feature No.3 @ Ch 5926)

Summary of ground treatment activites for lineament feature No.3 at approximate Ch.5926

Holes Drilled											Water Struck		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
1-G7	primary	5.5 o'clock	30-May	a/s	6	164	15	5933.84	23.8	2.90	320	10	31-May a/s	572	0	17.4	0.536	Collaring without sleeve, communicated with hole 2
2-G3	primary	3 o'clock	30-May	n/s	81	80	16	5936.24	26.2	3.13	230	No gauge	01-June m/s	1225	0	31.6	0.948	Packer left open, Time lost for packer insertion without sleeve
3-G11	primary	8 o'clock	30-May	n/s	70	80	11	5925.44	15.4	2.10	300	No gauge	Connected to hole G7					Packer left open, Time lost for packer insertion without sleeve
4-G13	primary	10 o'clock	31-May	m/s	25	37	10	5922.24	12.2	3.25	400	No gauge	Connected to hole G7					Packer left open. Time lost for packer insertion without sleeve
5-P2	primary	12 o'clock	31-May	a/s	10	232	29	5968.64	58.6	6.34	0	0	2-June m/s	8	0	0.2	0.006	Water loss/void at 5th rod=Ch. 5911, plugged, connect to G3
6-G9	Sec/Check	7 o'clock	02-Jun	a/s	4	107	11	5925.44	15.4	2.10	0	-	-					collaring only 4 mins critical time, at rod 11 - rods struck.
7-G12	Sec/Check	9 o'clock	03-Jun	m/s	60	101	20	5947.04	37	4.17	dry	-	3-June n/s	10	0	0.2	0	
8-G4	Sec/Check	4 o'clock	03-Jun	a/s	33	113	19	5944.64	34.6	3.94	dry	-	3-June n/s	25	0	0.2	0	
9-G6	Sec/Check	5 o'clock	03-Jun	n/s	?	38	5	5911.04	1	0.72	dry	-						reason for drilling & stop unknown
10-G14	Sec/Check	10.5 o'clock	03-Jun	n/s	19	63	15	5935.04	25	5.36	dry	-						
11-P1	Sec/Check	11 o'clock	03-Jun	n/s	0	98	27	5963.84	53.8	5.88	dry	-	4-June n/s	4	0	0.35	0.006	reinsert rods-previous hole & extd/abandon @ rod 27/struck
					5.13	18.55	178.00		303.00	39.89	1250.00			30.73	0.00	49.95	1.50	* distance ahead of the face " radial cover from tunnel
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	assumed distance of probe drill boom from face=11m

*drilling time until water is struck



Salient Points

Total Grout take = 8.4t OPC+74.8t RHPC=83.2 tonnes

TBM was halted for this feature (No. 3) on 30-May at 11h30

TBM restarted excavation on 4-June at 16h55

Total TBM downtime = 125.42 hours (including public Holiday - 1June)

Total drilling and fissure grouting time =20. 68 hours

Total time for packer insertion = 27.16 hours

Total time spent on setting up drill rig = 11.93 hours

LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B

MOHALE INTAKE FISSURE GROUTING FROM TBM FACE CH 7750 (feature No. 4)

Summary of ground treatment activites for lineament feature at approximate Ch.7750																		
			Holes Drilled								Water Struck		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
Normal	Probe MI/103	1 o'clock	09-Sep	a/n/s	49	158	34	7756.83	68.6	7.29	500	12	11-09 m/s	186	2.5	4.75	0.149	Grouting to plug the hole, had to regrout in a/s with 1250kg
1-P1	primary	11 o'clock	13-Sep	m/s	20	22	10	7750.98	11	1.77	380	12	15-09 m/s	151	-	5.8	0.174	
2-P3	primary	1 o'clock	13-Sep	a/s	25	27	11	7753.38	13.4	2.00		9.5	15-09 m/s	860	-	24.8	0.744	
3A-G4	primary	4o'clock	13-Sep	n/s	0	0	0	-	-	-	-	-	14-09 n/s	-	-	-	-	Hole abandoned-due to problems of drilling difficulties/rods obstructed
3-G5	primary	4-5 o'clock	3-14/09/20(	n/m/s	33	72	11	7754.78	14.8	2.04	-	8	14-09 n/s	790	-	26.2	0.786	
4-G10	Sec/Check	7-8 o'clock	14-Sep	a/s	62	72	15	7764.38	24.4	2.96	-	7.5	14-09 n/s	450	-	9.6	0.288	
																		* distance ahead of the face " radial cover from tunnel
					3.15	5.85	81.00		132.20	16.07	880.00			40.62	2.50	71.15	2.14	
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

Salient Points

Total Grout take = 2.5t OPC+ 71.15t RHPC=73.65 tonnes

TBM was halted for this feature on 07h30 13-September 2000.

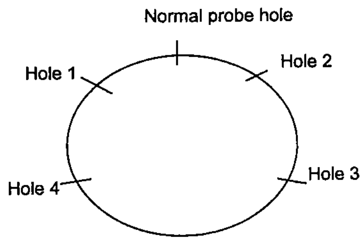
TBM restarted excavation at 11h30 on 18-September, (final grout set by 13h00 on 16 September 2000)

Total TBM downtime = 124 hours

Total drilling and fissure grouting time = 3.15 + 5.85 + 40.62 = 49.62 hours

Total drilling distance ahead of the face = 132.2 (excluding normal probe hole = 63.6m)

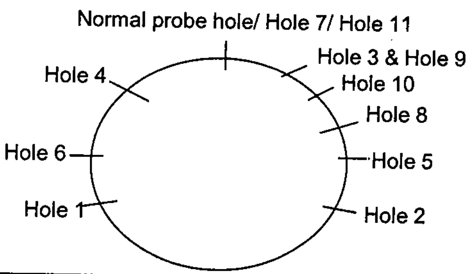
Normal probe Hole



LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B
MOHALE INTAKE FISSURE GROUTING FROM TBM FACE CH 7799 (feature No. 5)

Summary of ground treatment activites for lineament feature at approximate Ch.7799 (TBM Face at Ch 7789.338

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
P1	heck hole MI/104	12 o'clock	16-Sep	a/s	26	138	30	7848.3	59	6.37	***830***	9.2	17-Sep	30	-	0.9	0.018	Water inferred from pressure, plug hole to advance TBM.
1-G10	primary/MI105A	8 o'clock	20-Sep	m/s	20	17	6	7792.1	2.8	0.99			20-Sep	36	-	***0.70***	0.018	Grouting stopped after seepage at face(.7t) - see note 1
2-G5	primary/MI105B	4 o'clock	20-Sep	m/s	54	67	15	7813.7	24.4	3.06	205	10	20-21/sep	260	-	***4.6***	0.138	Water @ Ch 7793 (8 rod)/after 3.6t-seepage @ face-see note 2
3-P3	primary/MI105C	1 o'clock	20-Sep	a/s	34	43	11	7802.7	13.4	1.91	36		22-Sep	377	-	11.3	0.33	Water initially @ Ch 7790(7th rod)
4-P1	primary/MI105D	11 o'clock	20-Sep	n/s	50	74	18	7819.5	30.2	3.52	0	0	23-Sep	197	-	3.6	0.102	First attempt unseccessful, hole dry
G10	Redrill	8 o'clock	21-Sep	a/s	20	20	6	7792.1	2.8	0.99	No readings		21-Sep	1444		49.65	1.332	Only shift report recording - problem with packer lead to redrill
G5	Redrill	4 o'clock	21-Sep	a/s	20	70	15	7813.7	24.4	3.06	No readings		21-Sep	No take				Only shift report recording - problem with packer lead to redrill
5-G3	primary/MI105E	3 o'clock	22-Sep	n/s	24	18	7	7794.5	5.2	1.22	80	7.5	24-Sep	138	-	6.65	0.102	Water initially @ Ch 7790(7th rod), LHTP(M) grout record=4.05t
6-G12	primary/MI105F	9 o'clock	23-Sep	n/s	22	136	25	7837.7	48.4	5.36	8		24-Sep	9	-	0.2	0.006	low water after rod extraction
7-P2	primary/MI105G	12 o'clock	24-Sep	d/s	30	123	28	7843.5	54.2	5.82	0	0	24-Sep	35	-	0.6	0.018	Dry
8-G2	secndry/MI105H	2.5 o'clock	25-Sep	m/s	35	56	15	7813.7	24.4	2.96		8.5	25-Sep	538	-	12.6	0.378	Water initially @ Ch 7795.5
9-P3	secndry/MI105I	1 o'clock	25-Sep	m/s	redrill	76	15	7812.3	23	2.83	0	0	25-Sep	1	-	0.2	0.006	Increased water return at 9th rod reported
10-G1	secndry/MI105J	1.5 o'clock	26-Sep	a/s	20	119	15	7813.7	24.4	2.96	18	0	Grouting not required					Water initially @ 9th rod
11-P2	secndry/MI105K	12 o'clock	26-Sep	a/s	2	252	56	7910.7	121.4	12.26	0	0	Grouting not required					dry - normal probe
																		* distance ahead of the face " radial cover from tunnel
					5.95	20.15	262.00		458.00	53.29	1177.00			51.08	0.00	91.00	2.45	Distance of boom of probe rig fom face = 13 m.
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	



Salient Points

Total Grout take = 91tonnes of RHPC + 2.45 tonnes of Bentonite

TBM was halted for this feature at 21h45 (a/shift) on19-September 2000.

TBM restarted excavation at 12h55 on 27-September (final grout set by 13h15 on 26 September 2000)

Total TBM downtime = 183.17 hours

Total drilling and fissure grouting time = 5.95 + 20.15 + 51.08 = 77.18 hours (excluding last hole = 72.95hrs)

Total drilling distance ahead of face = 458m (excluding normal probe holes = 336.6m)

Chrysofluid added to certain mixes. Total chrysofluid used = 61 litres

Note 1 - Thicken mix & add chrysofluid(32.5l for 11 batches/again 5l for two batches/23.5l for 8 batches)

Note 2 - thicken mix 1:1.25 w/c-1000kg(2hrs)

Normal probe hole

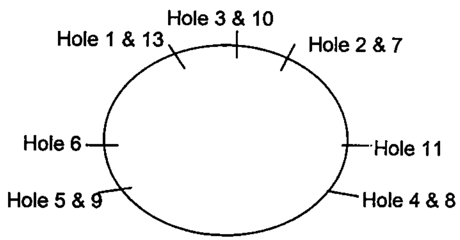
LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B
MOHALE INTAKE FISSURE GROUTING FROM TBM FACE CH 10004, 10057 and 10085 (feature No. 6 Ch 10084 - 10111)

Summary of ground treatment activites for lineament feature at approximate Ch.10085 to 10105 (TBM Face at Ch 10004, 10057 and 10085)

	Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
	Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
Stage 1	P1	probe/Stage 1	11 o'clock	01-Feb	m/s	25	359	47.5	10105.6	40.6	4.52	1016	18	01-Feb n/s	506	2.3	10.1	0.372	Hole plugged to enable TBM to advance to Ch 10065
	P3	Stage 1	1 o'clock	02-Feb	a/s	30	149	32	10068.43	63.8	6.83	Dry		No Grouting required					Check hole to provide confidence for TBM advance to 10065
Stage 2	P2	Stage 2	12 o'clock	07-Feb	d/s	35	143	26	10107.33	49.4	5.45	880	13.5	10-Feb a/s	45	-	0.8	0.024	Water intesected @ Ch 10105.5
	G4	Stage 2	4 o'clock	07-Feb	n/s	20	155	26	10108.73	50.8	5.49	847	12.5	9-Feb a/s	1119	-	29.4	0.882	Water intesected @ Ch 10107.5
	G11	Stage 2	8 o'clock	08-Feb	m/s	28	218	24	10103.93	46	5.03	928	15	8-Feb n/s	2668	34.8	43.8	2.358	Water intersected @ Ch 10102.7
	G12	Stage 2/Check	9 o'clock	11-Feb	m/s	32	168	27	10111.13	53.2	5.72	293	1.5	12-Feb d/s	1041	21.8	3.5	0.759	Holes plugged as agreed to enable TBM advance to Ch. 10085
	P3	Stage 2/Check	1 o'clock	11-Feb	a/s	35	·126	26	10108.73	50.8	5.49	864	13	12-Feb a/s	654	10.6	6.6	0.516	Holes plugged as agreed to enable TBM advance to Ch 10085
Stage 3	G4	Stage 3	4 o'clock	14-Feb	a/s	28	68	14.5	10107.8	23.2	2.94	812	11.5	15-Feb-a/s	3507	36.35	53	2.6805	Grouting interrupted due to seepage of grout through lining
	G11	Stage 3	8 o'clock	14-Feb	n/s	27	81	16	10111.4	26.8	3.29	-	-	15-Feb-a/s	2602	7.85	49.5	1.7205	Grouting interrupted due to seepage of grout through lining
	P2	Stage 3	12 o'clock	15-Feb	m/s	32	49	14.5	10107.8	23.2	2.94	812	11.5	17-Feb-a/s	1814	22.2	36	1.746	Grouting interrupted due to seepage of grout through lining
	G12	Stage 3/Check	9 o'clock	22-Feb	m/s	25	98	20	10121	36.4	4.11	Dry		No Grouting required				0	Collaringdone with no sleeve-space restriction from ring
	G3	Stage 3/Check	3 o'clock	22-Feb	m/s	25	106	20	10121	36.4	4.11	Dry		No Grouting required				0	Collaringdone with no sleeve-space restriction from ring
	P1	Stage 3/Check	11 o'clock	22-Feb	a/s	21	135	27	10137.8	53.2	5.72	830	12	22-Feb n/s	100	-	1.55	0.0465	Hole plugged to enable TBM advance
																			* distance ahead of the face " radial cover from tunnel
						6.05	30.92	320.50		553.83	61.66	7282.00			234.27	135.9	234.25	11.10	Dist. of boom of rig from face=13 m/11.6m from erector mount.
						hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

Grout take Overall : 370.15

Packer used in this feature was hydraulic packers



	Stage 1 - Ch10004	Stage 2 - 10057	Stage 3 - 10085
TBM stop / *FG start*	07h55/***17h14 (1-Feb-2001)***	19h10/***08h05 (7-Feb-2001)***	18h05/***18h05(14 Feb 2001)***
FG end /TBM restart	***00h30*** /02h55, ***(3 Feb 2001)***	***12h30*** /19h55 ***(13-Feb-2001)***	***07h30(23-Feb-2001)*** /15h50 (3-March-2001)

Total

Total drilling and fissure grouting time = 6.05 + 30.92 + 234.27 = 271.24 hours
Total drilling distance ahead of face = 553.83m (incl normal probe = 614.196m)
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 593.50 hrs

LESOTHO HIGHLANDS WATER PROJECT - PHASE 1B

MOHALE INTAKE FISSURE GROUTING FROM TBM FACE CH 10138, 10143 and 10153 (feature No. 7 from Ch 10135 - 10153)

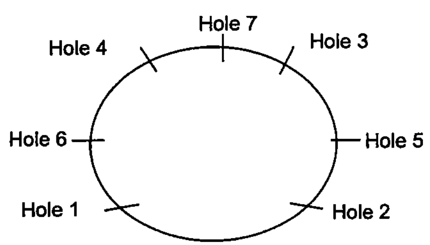
Summary of ground treatment activites for lineament feature No. 7 at approximate Ch.10138/10143/10153 (TBM Face at Ch 10125 & Ch 10130)

	Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
	Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC'	RHPC	Bent'nite	Remarks
Stage 1-Ch 10125	G11	Primary	8 o'clock	06-Mar	a/s	25	47	10	10137.8	12.4	1.81	420	11	08-Mar	3912	-	107.2	0.246	Intersected water Ch 10 135
	G4	Primary	4 o'clock	06-Mar	n/s	18	55	11	10140.2	13.4	1.91	420	10	07-Mar	270	-	6.8	0.09	
	P3	Primary	1 o'clock	07-Mar	a/s	25	37	12	10142.6	15.8	2.23	225	10	09-Mar	699	-	18.5		
	P1	Primary	11 o'clock	07-Mar	n/s	22	37	11	10140.2	14.8	2.14	185	10	Connected to P3/G11-grouting not warranted					
Stage 2-Ch10125	G3	secondary check	4 o'clock	11-Mar	a/s	34	96	20	10161.8	36.4	4.11	Dry	0	07-Mar		-	-		
	G12	secondary check	8 o'clock	11-Mar	n/s	37	126	20	10161.8	36.4	4.11	Dry	0	12-Mar	2499	9.3	47.4		
	P2	secondary check	12 o'clock	11-Mar	n/s	19	65	16	10152.2	26.8	3.19	180	9	12-Mar	8310	39.5	235.1		On 15 March, hole appear to refuse-later found water/grt until 19 March
	G4	Tertiary/check	4 o'clock	15-Mar	a/s	20	145	25	10173.8	48.4	5.36	Dry	0	Grouting not warranted, because hole dry					
Stage 3 - Ch 10130	G11	Primary	10 o'clock	19-Mar	n/s	21	46	11	10145.2	14.8	2.14	140	10	21-Mar	5915	1.1	163.25		Hole connect to P1, hole plugged eventually on 27-March m/s
	G4	Primary	4 o'clock	20-Mar	m/s	22	44	13	10150.0	19.6	2.60	Dry		27-Mar	37		0.8		G4 abandoned at 11th rod due entry into previous ungrouted hole
	G3	Primary	3 o'clock	20-Mar	m/s	18	99	20	10166.8	36.4	4.11	Dry							
	P1	Primary	11 o'clock	20-Mar	m/s	31	73	13	10148.8	19.6	2.50	120	10	23-Mar	3817	5.40	127.70		
	P3	Primary	1 o'clock	20-Mar	n/s	29	102	20	10166.8	36.4	4.11	90	10	22-Mar	520	1.20	6.00		Hole intially dry, but later showed water.
	G14	Primary	10.5 o'clock	21-Mar	d/s	66	53	11	10145.2	14.8	3.51		9	21-Mar	1138		37.25		
	P2	Check	12 o'clock	27-Mar	a/s	35	60	20	10166.8	36.4	7.08	Dry		28-Mar	50	0.65			Hole plugged
	G12	Check	9 o'clock	27-Mar	n/s	30	56	20	10166.8	36.4	7.08	Dry		28-Mar	37	0.45			Hole plugged
																			* distance ahead of the face " radial cover from tunnel
						7.53	19.02	253.00		418.80	58.00	1780.00			453.40	57.6	750.00	0.34	Dist. of boom of rig from face=13 m/11.6m from erector mount.
						hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

Grout take Overall : 807.60

G11: 5711

50



	Stage 1 - Ch10125	Stage 2 - 10125	Stage 3 - 10130
TBM stop / *FG start*	17h40/*17h50 (6-Mar-2001)*	???/*16h15(7-Mar-2001)*	02h10/*02h20(20 Mar 2001)*
FG end /TBM restart	*07h00* /???, *(7 Mar 2001)*	*18h00* /20h05 *(19-Mar-2001)*	*15h00* /20h30 (28-March-2001)

Total drilling and fissure grouting time = 479.95 hrs

Overall Grout take = 807.60 tonnes

Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.

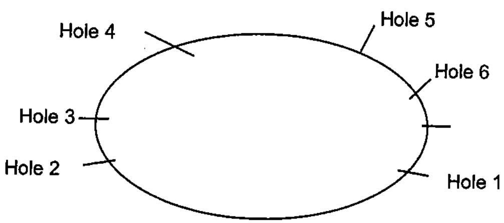
Total TBM downtime = 524.75 hrs

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. 8 *At Approx Chainage:* 10183
Face Ch. 10131.798 *Boom Position: Extended* 11.6

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
G4	Primary	4 o'clock	28-Mar	n/s	28	85	26	10182.6	50.8	5.59	225	11	28-Mar	339	11.95			
G11	Primary	8 o'clock	29-Mar	m/s	27	10	5	10132.2	0.4	0.76								
G12	Primary	9 o'clock	29-Mar	m/s	18	80	25	10180.2	48.4	5.36								Rods stuck due to lost packer; hole abandoned
P1	Primary	11 o'clock	29-Mar	m/s	41	83	25	10180.2	48.4	5.36	Dry			133	3			
P3	Primary	1 o'clock	29-Mar	a/s	43	6	5	10132.2	0.4	0.76	Dry							
G1	Primary	2 o'clock	29-Mar	a/s	40	71	25	10180.2	48.4	9.06	Dry							Rods stuck; hole abandoned
																		* distance ahead of the face " radial cover from tunnel periphery
					3.28	5.58	111.00		196.80	26.88	225.00			7.87	14.95	0.00	0.00	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

TBM Stop	FG Start	FG Stop	TBM Restart
23h50 (28Mar01)	00h00 (29Mar01)	17h10 (04Apr01)	21h05 (04Apr01)



Total drilling and fissure grouting time = 16.73 hours
Overall Grout take = 14.95 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 165.25 hrs

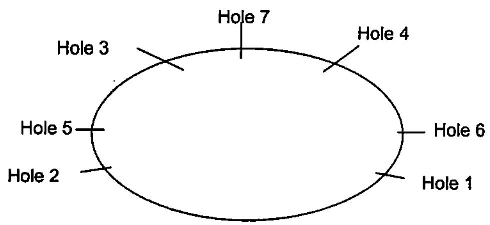
Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5°
All others: distance 7.5m, angle 5.5°

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. ***9*** ***At Approx Chainage:*** ***10185***
Face Ch. ***10160.376*** ***Boom Position:*** ***Extended*** ***11.6***

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
G4	Primary	4 o'clock	06-Apr	n/s	17	80	25	10208.8	48.4	5.36	140	1.5	07-Apr	206		4.35	0.006	Struck water at Ch 10184.8
G11	Primary	8 o'clock	06-Apr	n/s	15	73	25	10208.8	48.4	5.36	119	2	07-Apr	74		1.05		Struck water at Ch 10184.8
P1	Primary	11 o'clock	07-Apr	m/s	40	86	25	10208.8	48.4	5.36	150	1	07-Apr	120		3.3		Struck water at Ch 10190.1
P3	Primary	1 o'clock	07-Apr	m/s	26	75	25	10208.8	48.4	5.36	173	4						Struck water Ch 10189.6
G12	Secondary	9 o'clock	08-Apr	a/s	34	73	25	10208.8	48.4	5.36								No water intersected
G3	Secondary	3 o'clock	08-Apr	a/s	35	77	25	10208.8	48.4	5.36								No water intersected
P2	Secondary	12 o'clock	08-Apr	n/s	23	141	40	10244.8	84.4	8.81			09-Apr	20		0.3		Hole plugged
																		* distance ahead of the face " radial cover from tunnel periphery
					3.17	10.08	190.00		374.80	40.96	581.47			7.00	0.00	9.00	0.01	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

TBM Stop	FG Start	FG Stop	TBM Restart
23h10 (06Apr01)	23h10 (06Apr01)	10h40 (09Apr01)	14h00 (09Apr01)



Total drilling and fissure grouting time = 20.25 hours

Overall Grout take = 9.00 tonnes

Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.

Total TBM downtime = 62.83 hrs

Radial cover: function of position and angle of grout hole

G1, G2, G13 and G14: distance 6.5m, angle 9.5o

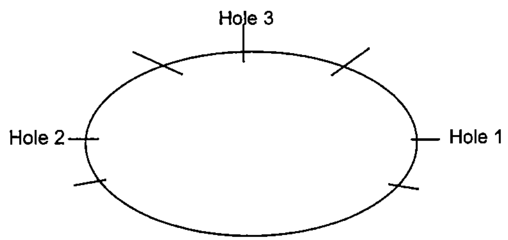
All others: distance 7.5m, angle 5.5o

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. *10* ***At Approx Chainage:*** *10211.76*

Face Ch. *10199.362* ***Boom Position:*** *Extended* *11.6*

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
G3	Primary	3 o'clock	10-Apr	n/s	22	87	25	10247.8	48.4	5.36			11-Apr	13		0.4		No water intersected - hole later plugged
G12	Primary	9 o'clock	11-Apr	m/s	19	78	25	10247.8	48.4	5.36			11-Apr	17		0.9		No water intersected - hole later plugged
P2	Secondary	12 o'clock	11-Apr	n/s	34	212	53	10315.0	115.6	11.80								Normal Probe
																		* distance ahead of the face . " radial cover from tunnel periphery
					1.25	6.28	103.00		212.40	22.52	0.00			0.50	0.00	1.30	0.00	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	



TBM Stop	FG Start	FG Stop	TBM Restart
01h45 (11Apr01)	01h45 (11Apr01)	10h20 (11Apr01)	22h30 (11Apr01)

Total drilling and fissure grouting time = 8.03 hours

Overall Grout take = 1.30 tonnes

Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.

Total TBM downtime = 20.75 hrs

Radial cover: function of position and angle of grout hole

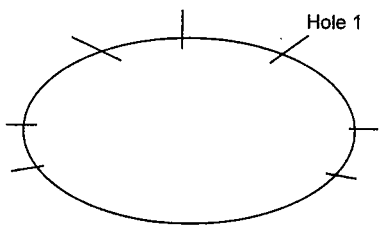
G1, G2, G13 and G14: distance 6.5m, angle 9.5o

All others: distance 7.5m, angle 5.5o

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. ***11*** ***At Approx Chainage:*** ***10455***
Face Ch. ***10330.414*** ***Boom Position: Extended*** ***11.6***

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
P3	Primary	1 o'clock	22-Apr	n/s	20	238	60	10462.8	132.4	13.41	175	7	23-Apr	70		2.9		Struck water Ch 10455 - hole later plugged
																		* distance ahead of the face " radial cover from tunnel periphery
					0.33	3.97	60.00		132.40	13.41	175.00			1.17	0.00	2.90	0.00	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	



TBM Stop	FG Start	FG Stop	TBM Restart
03h45 (23Apr01)	03h45 (23Apr01)	15h35 (23Apr01)	16h10 (23Apr01)

Total drilling and fissure grouting time = 5.47 hours
Overall Grout take = 2.90 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 12.42 hrs

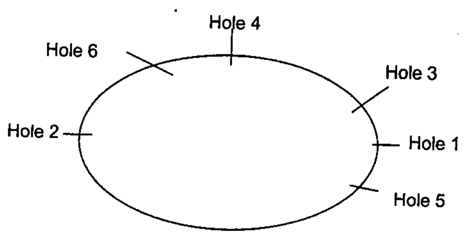
Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5°
All others: distance 7.5m, angle 5.5°

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. 12 *At Approx Chainage:* 10459 to 10478
Face Ch. 10430.007 *Boom Position: Extended* 11.6

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					Remarks
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	
G3	Primary	3 o'clock	27-Apr	a/s	6	37	8	10437.6	7.6	1.45								Packer problems - hole abandoned
G12	Primary	9 o'clock	27-Apr	a/s	39	75	25	10478.4	48.4	5.36	109	7	03-May	155		4		Water intersected Ch 10459
G2	Primary	2 o'clock	27-Apr	n/s	14	50	18	10461.6	31.6	6.29	9		03-May	5		0.2		Void/Fractured zone Ch 10456
P2	Primary	12 o'clock	02-May	n/s	29	78	25	10478.4	48.4	5.36	100	5	03-May	2715		77.85	0.34	Water intersected Ch 10464
G4	Secondary	4 o'clock	03-May	n/s	24	80	9	10440.0	10.0	1.68			07-May	10		0.55		Hole abandoned - later revisited & plugged
P1	Secondary	11 o'clock	06-May	n/s	39	181	46	10528.8	98.8	10.19			07-May	101		4.1		Hole plugged
					2.52	8.35	131.00		244.80	30.32	217.98			49.77	0.00	86.70	0.34	* distance ahead of the face " radial cover from tunnel periphery
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)

4100.00



TBM Stop	FG Start	FG Stop	TBM Restart
12h20 (27Apr01)	12h40 (27Apr01)	17h45 (07May01)	18h15 (07May01)

Total drilling and fissure grouting time = 60.63 hours
Overall Grout take = 86.70 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 245.92 hrs

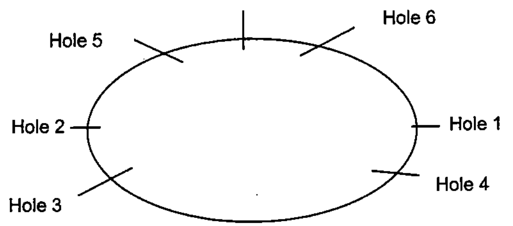
Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5 °
All others: distance 7.5m, angle 5.5 °

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. *13* *At Approx Chainage:* *10860* *to* *10872*
Face Ch. *10788.084* *Boom Position:* *Extended* *11.6*

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
Water intersected by TBM head			19/May	*n/s*	-	-	-	-	-	-								*Dewatering undertaken up to14Jun01*
G3	Primary	3 o'clock	15-Jun	a/s	22	70	25	10836.5	48.4	5.36								No water intersected
G12	Primary	9 o'clock	15-Jun	n/s	33	35	15	10812.5	24.4	3.06								No water intersected
G11	Primary	8 o'clock	15-Jun	n/s	21	39	15	10812.5	24.4	5.10								No water intersected
G4	Primary	4 o'clock	16-Jun	m/s	18	83	25	10836.5	48.4	5.36								No water intersected
P1	Primary	11 o'clock	16-Jun	m/s	39	158	43	10879.7	91.6	9.50	70	2	18-Jun	50		1.35		Water intersected Ch 10860 - hole plugged
P3	Primary	1 o'clock	16-Jun	a/s	30	165	50	10896.5	108.4	11.11			18-Jun	50		0.9		Water intersected Ch 10872 - hole plugged
																		* distance ahead of the face " radial cover from tunnel periphery
					2.72	9.17	173.00		345.60	39.48	70.00			1.67	0.00	2.25	0.00	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

TBM Stop	FG Start	FG Stop	TBM Restart
03h40 (20May01)	23h00 (16Jun01)	12h30 (18Jun01)	13h05 (18Jun01)



Total drilling and fissure grouting time = 13.55 hours

Overall Grout take = 2.25 tonnes

Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.

Total TBM downtime = 705.42 hrs

Radial cover: function of position and angle of grout hole

G1, G2, G13 and G14: distance 6.5m, angle 9.5o

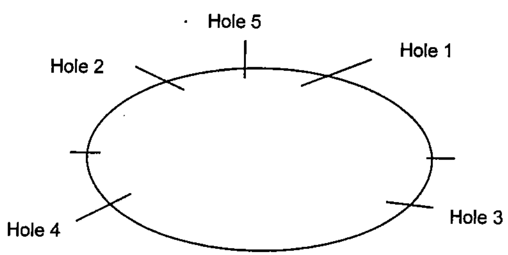
All others: distance 7.5m, angle 5.5o

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. ***14*** ***At Approx Chainage:*** ***11400***
Face Ch. ***11377.584*** ***Boom Position:*** ***Extended*** ***11.6***

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
P3	Primary	1 o'clock	22-Jun	a/s	22	86	25	11426.0	48.4	5.36	500		23-Jul	1205	43.5			Water intersected Ch 11400
P1	Primary	11 o'clock	22-Jul	n/s	25	93	25	11426.0	48.4	5.36	300	4	23-Jul	267		8.1		Water intersected Ch 11400
G4	Secondary	4 o'clock	24-Jul	n/s	37	59	20	11414.0	36.4	4.21	78	3						Water intersected Ch 11394
G10	Secondary	8 o'clock	24-Jul	n/s	28	56	20	11414.0	36.4	4.21	120	3						Water intersected Ch 11402
P2	Secondary	12 o'clock	25-Jul	m/s	51	287	55	11498.0	120.4	12.26	300							Water intersected Ch 11401
																		* distance ahead of the face " radial cover from tunnel periphery
					2.72	9.68	145.00		290.00	31.39	1298.00			24.53	43.50	8.10	0.00	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

TBM Stop	FG Start	FG Stop	TBM Restart
03h30 (21Jul01)	08h25 (23Jul01)	19h00 (25Jul01)	02h20 (26Jul01)



Total drilling and fissure grouting time = 36.93 hours
Overall Grout take = 51.60 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 118.83 hrs

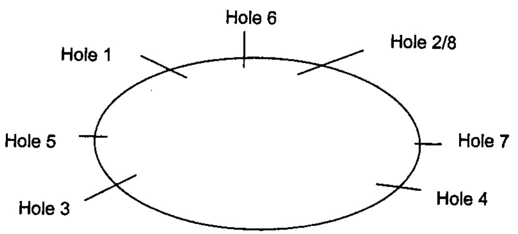
Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5 °
All others: distance 7.5m, angle 5.5 °

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. *15* ***At Approx Chainage:*** *12888 to 12895*
Face Ch. *12848.428* ***Boom Position:*** *Extended 11.6*

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
P1	Primary	11 o'clock	06-Oct	n/s	30	68	25	12896.8	48.4	5.36	315	33	09-Oct	536	10.65		0.317	Water intersected Ch 12895
P3	Primary	1 o'clock	07-Oct	m/s	20	52	22	12889.6	41.2	4.67	394	34	09-Oct	532	8.15		0.256	Water intersected Ch 12888
G11	Primary	8 o'clock	07-Oct	n/s	14	68	25	12896.8	48.4	5.36	84	22	08-Oct	50	1.1		0.033	Water intersected Ch 12894
G4	Primary	4 o'clock	07-Oct	n/s	38	74	30	12908.8	60.4	6.51	53	10	08-Oct	186	1.5		0.174	Water intersected Ch 12890
G12	Secondary	9 o'clock	11-Oct	a/s	37	94	30	12908.8	60.4	6.51	60	34	12-Oct	170	1.45		0.0535	Water observed when installing packer
P2	Secondary	12 o'clock	11-Oct	a/s	32	81	26	12899.2	50.8	5.59	78	34	12-Oct	15	0.1		0.003	Water intersected Ch 12894
G3	Secondary	3 o'clock	11-Oct	n/s	23	90	30	12908.8	60.4	6.51	67	30	11-Oct	161	0.85		0.0295	Water intersected Ch 12889
P3	Tertiary	1 o'clock	12-Oct	n/s	-	249	60	12980.8	132.4	13.41	112	18	13-Oct	68	0.5		0.04	Water intersected Ch 12901
																		* distance ahead of the face " radial cover from tunnel periphery
					3.23	12.93	248.00		502.40	53.91	1161.42			28.63	24.30	0.00	0.91	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	

TBM Stop	FG Start	FG Stop	TBM Restart
02h45 (07Oct01)	04h20 (07Oct01)	14h55 (13Oct01)	02h20 (14Oct01)



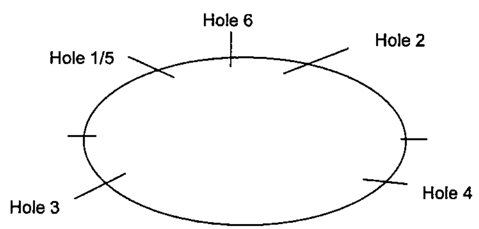
Total drilling and fissure grouting time = 44.80 hours
Overall Grout take = 24.30 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 167.58 hrs

Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5o
All others: distance 7.5m, angle 5.5o

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

Feature No. *16* *At Approx Chainage:* *12900* *to* *12908*
Face Ch. *12866.68* *Boom Position:* *Extended* *11.6*

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
P1	Primary	11 o'clock	16-Oct	n/s	20	57	23	12910.3	43.6	4.90	342	28	18-Oct	591	11.9		0.3525	Water intersected Ch 12908
P3	Primary	1 o'clock	17-Oct	m/s	35	68	25	12915.1	48.4	5.36	84	8	17-Oct	290	3.65		0.245	Water intersected Ch 12900
G11	Primary	8 o'clock	17-Oct	m/s	26	67	25	12915.1	48.4	5.36			17-Oct	24	0.05		0.0115	No water intersected
G4	Primary	4 o'clock	17-Oct	a/s	37	63	25	12915.1	48.4	5.36			17-Oct	20	0.05		0.0115	No water intersected
P1	Secondary	11 o'clock	19-Jan	a/s	3	37	20	12903.1	36.4	4.21	307	30	19-Oct	455	11.25		0.3405	Water intersected Ch 12903
P2	Secondary	12 o'clock	20-Oct	a/s	30	52	20	12903.1	36.4	4.21	120	20	20-Oct	125	5.35		0.315	Water intersected Ch 12902
																		* distance ahead of the face " radial cover from tunnel periphery
					2.52	5.73	138.00		261.60	29.39	853.32			25.08	32.25	0.00	1.28	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	



TBM Stop	FG Start	FG Stop	TBM Restart
23h20 (16Oct01)	05h00 (17Oct01)	22h10 (21Oct01)	13h15 (22Oct01)

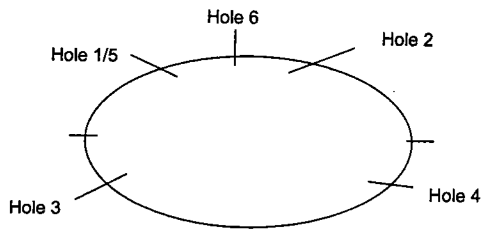
Total drilling and fissure grouting time = 33.33 hours
Overall Grout take = 32.25 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.
Total TBM downtime = 133.92 hrs

Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5o
All others: distance 7.5m, angle 5.5o

Lesotho Highlands Water Project - Phase IB
Contract LHDA 2007: Mohale Tunnel
Fissure Grouting in Intake TBM Drive

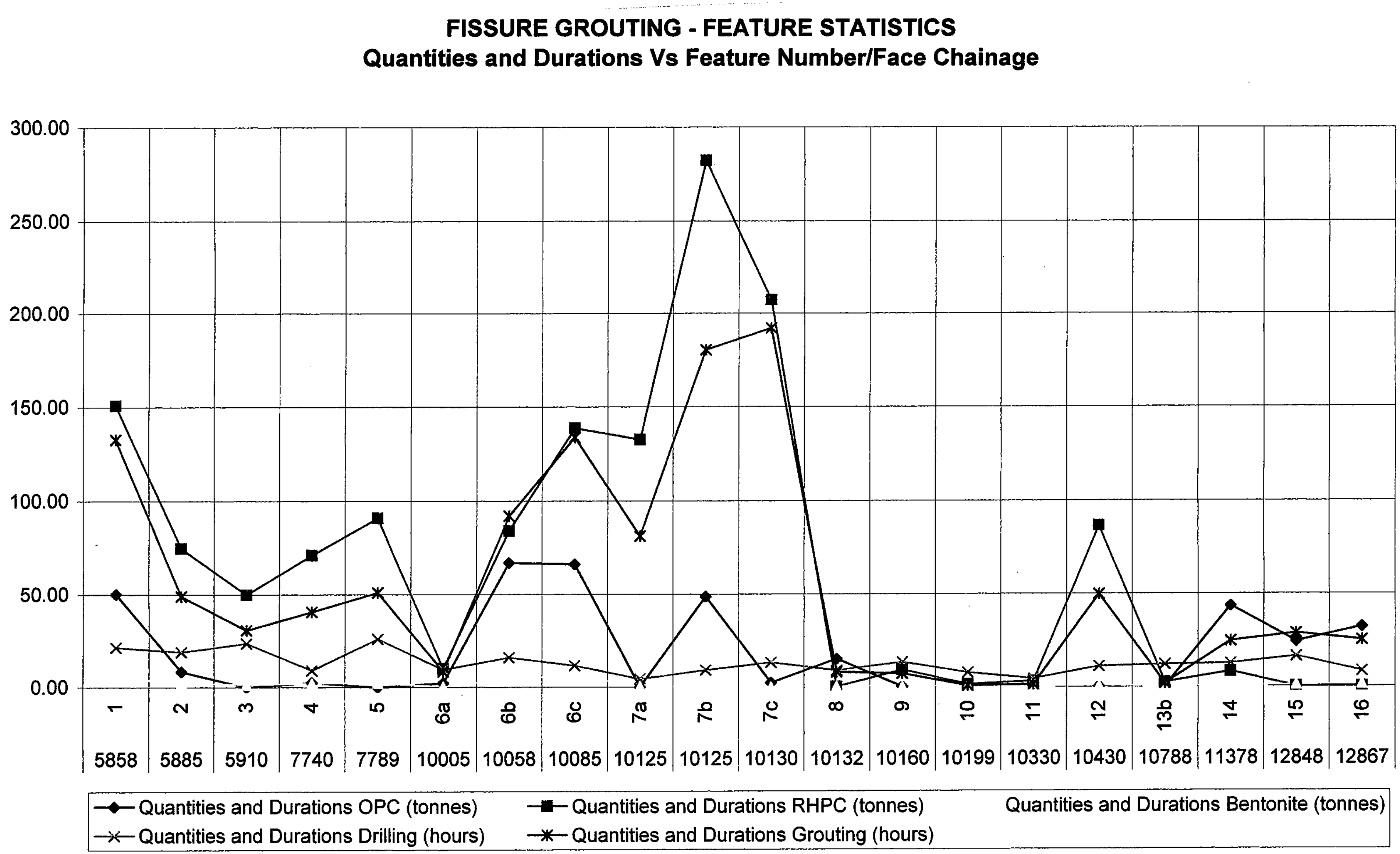
Feature No. ***16*** ***At Approx Chainage:*** ***12900 to 12908***
Face Ch. ***12866.68*** ***Boom Position:*** ***Extended*** ***11.6***

Holes Drilled					Durations		Coverage details				Water details		Holes Grouted					
Hole	Hole type	Position	Date	shift	Collaring	Driling	Rods	End Ch	length*	R/cover"	flow l/min	Pressure	Date start	Duration	OPC	RHPC	Bent'nite	Remarks
P1	Primary	11 o'clock	16-Oct	n/s	20	57	23	12910.3	43.6	4.90	342	28	18-Oct	591	11.9		0.3525	Water intersected Ch 12908
P3	Primary	1 o'clock	17-Oct	m/s	35	68	25	12915.1	48.4	5.36	84	8	17-Oct	290	3.65		0.245	Water intersected Ch 12900
G11	Primary	8 o'clock	17-Oct	m/s	26	67	25	12915.1	48.4	5.36			17-Oct	24	0.05		0.0115	No water intersected
G4	Primary	4 o'clock	17-Oct	a/s	37	63	25	12915.1	48.4	5.36			17-Oct	20	0.05		0.0115	No water intersected
P1	Secondary	11 o'clock	19-Jan	a/s	3	37	20	12903.1	36.4	4.21	307	30	19-Oct	455	11.25		0.3405	Water intersected Ch 12903
P2	Secondary	12 o'clock	20-Oct	a/s	30	52	20	12903.1	36.4	4.21	120	20	20-Oct	125	5.35		0.315	Water intersected Ch 12902
																		* distance ahead of the face " radial cover from tunnel periphery
					2.52	5.73	138.00		261.60	29.39	853.32			25.08	32.25	0.00	1.28	Dist. of boom from face=13/11.6m from erector mount.(retrc./ext. resp.)
					hrs	hrs	No.		m					hrs	tonnes	tonnes	tonnes	



Total drilling and fissure grouting time = 33.33 hours
Overall Grout take = 32.25 tonnes
Chrysofluid added to certain mixes to enable thicker mixes to be pumped to stop grout ingress through lining.

Radial cover: function of position and angle of grout hole
G1, G2, G13 and G14: distance 6.5m, angle 9.5o
All others: distance 7.5m, angle 5.5o


FISSURE GROUTING - FEATURE STATISTICS
Quantities and Durations Vs Feature Number/Face Chainage
300.00
250.00
200.00
150.00
100.00
50.00
0.00
1
2
3
4
5
6a
6b
6c
7a
7b
7c
8
9
10
11
12
13b
14
15
16
5858
5885
5910
7740
7789
10005
10058
10085
10125
10125
10130
10132
10160
10199
10330
10430
10788
11378
12848
12867
Quantities and Durations OPC (tonnes)
Quantities and Durations RHPC (tonnes)
Quantities and Durations Bentonite (tonnes)
Quantities and Durations Drilling (hours)
Quantities and Durations Grouting (hours)