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The uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study: Raw Water

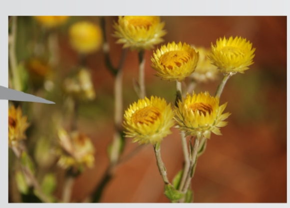
GEOTECHNICAL REPORT

SUPPORTING DOCUMENT 3:

SMITHFIELD DAM: MATERIALS AND GEOTECHNICAL INVESTIGATION
VOLUME 1 OF 3

FINAL

NOVEMBER 2014



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*Approved for **Consultants**:*

A van Schalkwyk	***DB Badenhorst***
Sub-Task Leader	*Task Leader*

DEPARTMENT OF WATER AFFAIRS (DWA): Directorate: Options Analysis

Approved for DWA:

K Bester	***LS Mabuda***
Chief Engineer: Options Analysis (East)	*Chief Director: Integrated Water Resource Planning*

- *BKS (Pty) Ltd was acquired by AECOM Technology Corporation on 1 November 2012*

Prepared by:

AECOM

AECOM SA (Pty) Ltd
PO Box 3173
Pretoria
0001

In association with:

Africa Geo-Environmental Services — AGES Purpose with Passion

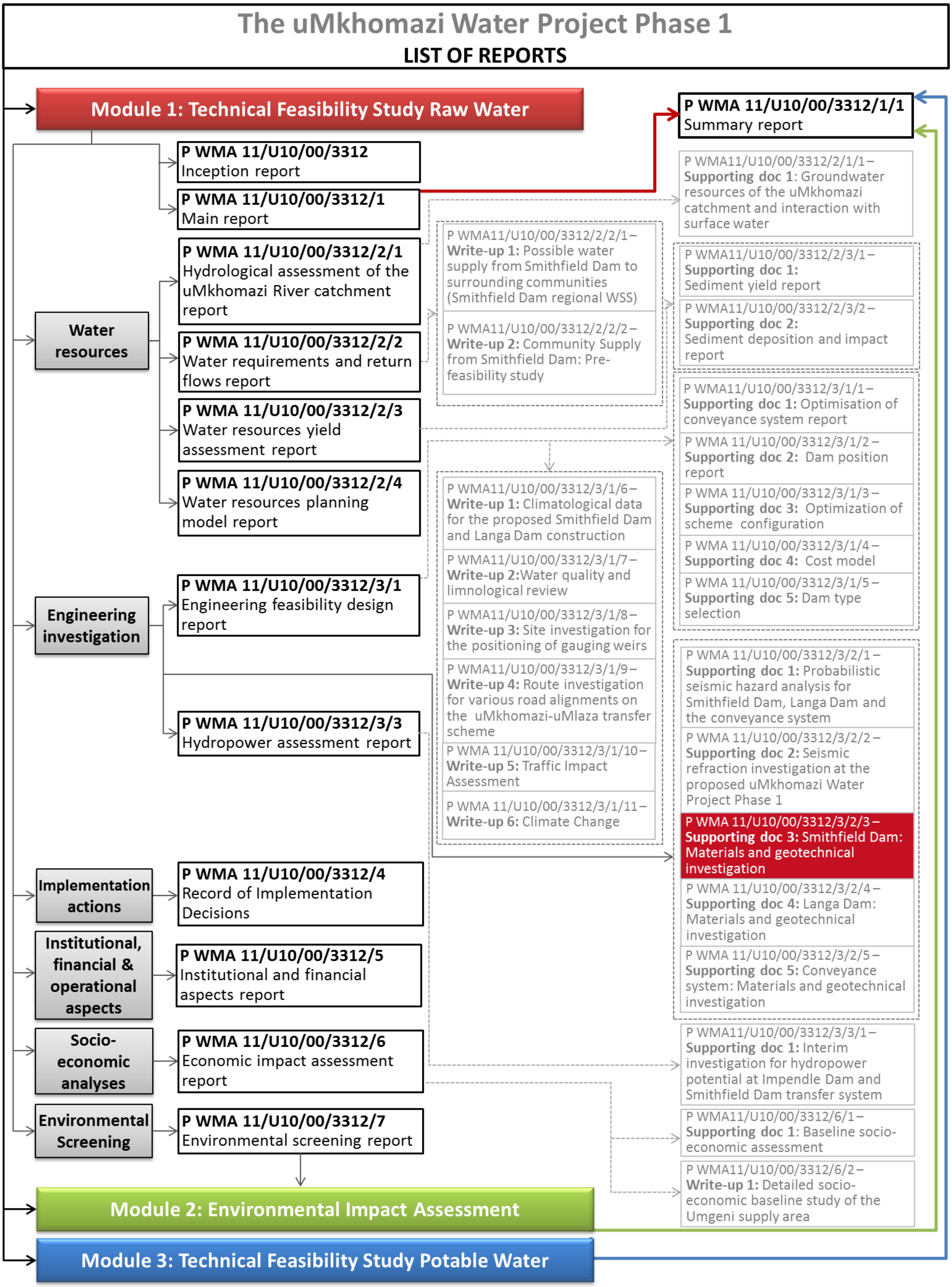
Mogoba Maphuthi and Associates — Mogoba Maphuthi & Associates (MMA)

Urban-Econ — URBAN-ECON Development Economists

PREAMBLE

In June 2014, two years after the commencement of the uMkhomazi Water Project Phase 1 Feasibility Study, a new Department of Water and Sanitation was formed by Cabinet, including the formerly known Department of Water Affairs.

In order to maintain consistent reporting, all reports emanating from Module 1 of the study will be published under the Department of Water Affairs name.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Executive summary

The objectives of this report are to describe the results of construction materials and geotechnical investigations for the following components of the uMkhomazi Water Project:

- *Smithfield Dam site B as defined in Supporting Report 1 on the Engineering Investigations.*
- *An earth core rockfill dam with approach channel and side spillway with chute and plunge pool on the left flank and diversion tunnels through the right flank.*
- *Alternatively, a composite dam with a central RCC Gravity spillway section.*
- *Alternatively, a zoned embankment dam with approach channel and side spillway with chute and plunge pool on the left flank and diversion tunnels through the right flank.*
- *Alternatively, a concrete-faced rockfill dam.*
- *An earth core rockfill dam or zoned embankment dam across the saddle on the left flank.*
- *Construction material sources for the above types of dams.*
- *Concrete aggregate for the inlet section of the conveyance tunnel.*

Information from published geological maps was used to describe the general geology of the area, while pre-feasibility geotechnical investigations provided valuable information on sources for impervious embankment material and also limited information on founding conditions for a lower dam.

The area around the site is underlain by rocks of the Volksrust Formation of the Ecca Group, comprising shales (mudrocks) with sub-ordinate sandstones. Three near-horizontal dolerite sills have intruded mainly concordantly into the sedimentary strata and are responsible for the narrow river valley at the dam site and the presence of good quality rock for concrete aggregate and rockfill.

Seismic refraction surveys have been conducted across the proposed quarry areas, the dam centre line and the diversion tunnels. Although the seismic velocities tended to over-estimate the depth of sound rock, they were extremely useful in showing the presence of dolerite sills below a cover of shale and also to identify the positions of faults.

A Probabilistic Seismic Hazard Analysis (PSHA) was conducted by Dr A Kijko of the Natural Hazards Assessment Consultancy and classified the site as of low seismic risk.

Two borrow areas that are located below FSL 930 masl can provide about 1.65 of the required 1.8 million m^3 impervious material required for a Earth Core Rockfill (ECR) dam or Zoned Embankment dam. The shortfall can be supplemented by using some of the completely and highly weathered shale (overburden) from the rock quarry or the soil overburden from the plunge pool excavation.

A quarry can produce about 2.6 million m^3 of hard dolerite suitable as rockfill or concrete aggregate. However, in order to mine this material, about 0.6 million m^3 of completely to highly weathered shale and 0.6 million m^3 of moderately weathered to unweathered shale (and in some places weathered dolerite), have to be removed. The shales are prone to rapid disintegration upon exposure and can be used in a zoned soft rock/hard rock embankment, provided it is used in the inner zones and is protected on the outside by durable dolerite.

Sufficient hard dolerite is available for construction of an RCC gravity dam.

If the shale overburden and underlying dolerite is combined and the floor of Quarry 1 is excavated to elevation 865 masl, sufficient rockfill material is available for a zoned embankment with soft rock inner zones and durable hard outer shells.

Founding conditions along the dam centre line are generally not very suitable because of deep weathering of shales along the higher flanks and the presence of a thick (14 m) layer of transported material on the right flank.

Due to the presence of deeply weathered shale on the upper left flank (14 m – 30+ m) and transported material and deep weathering of shales on the right flank (13 – 25 m), it is not considered feasible to construct a concrete dam along the full length of the centre line. However, in the central section a concrete dam can be founded on strong dolerite and strong indurated shale at depths between 2 m and 11 m.

The shells and plinth of a rockfill dam can be founded at depths between 3 m and 10 m on the left flank, 1.5 m and 5 m in the central section and 3 m to 15.0 m on the right flank. A large volume of soil will have to be excavated from below the dam wall and this might be suitable as construction material for the saddle dam.

The core trench for any embankment dam can be founded at between 4 m and 10.6 m on the left flank, 2 m and 11m in the central section and 3.5 and 15 m on the right flank.

Lugeon water tests generally showed very low permeabilities, but low gradients of the natural water table indicate the opposite. A grout curtain will have to be provided.

The control structure for a side spillway on the upper left flank can be founded on slightly weathered shale at depths ranging between 15 m and 20 m below ground surface and the concrete lined channel can be founded on moderately weathered shale at depths of between 10 and 12 m.

The clay core of an earthfill or rockfill dam across the saddle can be founded on moderately weathered shale that occurs at depths of between 2 m and 4 m.

The foundations along the saddle are generally impervious, but a grout curtain is nevertheless recommended.

Depending on their positions, the proposed 5 x 6 m diameter diversion tunnels can vary in length between about 300 m and 400 m. In every case about 200 m length of tunnel will be in rock that requires substantial support, while the remaining part is in sound dolerite where only local rockbolt support might be needed.

Excavations for tunnel portals will result in steep slopes in moderately weathered shale (that is prone to rapid deterioration) and moderately to highly weathered dolerite (corestones in a soil matrix). Flattening of these slopes cannot be done due to the steep topography of the river flanks. Provision will therefore have to be made for slope support, protection and drainage by means of rock anchors, shotcrete and drain holes.

The stability of the reservoir rim was assessed by means of field studies and calculations of heights and run-ups of impulse waves generated by landslides. It was concluded that there is a moderate (1:50 year) probability of a talus/gravel failure from two adjacent slope areas that will result in a run-up of up to about 1.4 m against the main dam wall. It also shows that there is an extremely low (1:10 000 year) probability of a large rock slide that will result in a run-up of about 4.4 m against the main dam wall. The available freeboard will prevent overtopping of the dam walls in the event of such failures.

Recommendations are made for additional geotechnical investigations during the design stage of the project.

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LIST OF ABBREVIATIONS

ACV	Aggregate Crushing Value
AEG	Association of Engineering Geologists
BKS	BKS (Pty) Ltd
CCR	Clay Core Rockfill
CFR	Concrete Faced Rockfill
CGS	Council for Geoscience
D:NWRP	Directorate: National Water Resource Planning
D:NWRP	Directorate: National Water Resource Planning
D10	Effective size (sieve size passing 10% of soil)
DM	District Municipality
DWA	Department of Water Affairs
ECRD	Earth Core Rockfill Dam
FSL	Full Supply Level
KZN	KwaZulu-Natal
LL	Liquid Limit
LM	Local Municipality
LS	Linear Shrinkage
MAR	Annual Run off
masl	Metres above sea level
MCE	Maximum Credible Earthquake
MDD	Maximum Dry Density
MDE	Maximum Design Earthquake
MMTS	Mooi Mgeni Transfer Scheme
OBE	Operating Basis Earthquake
OMC	Optimum Moisture Content
PE	Potential Expansiveness
PGA	Peak Ground Acceleration
PI	Plasticity Index
PSHA	Probabilistic Seismic Hazard Analysis
PSP	Professional Service Provider
RSA	Republic of South Africa
SAICE	South African Institution of Civil Engineering
SAIEG	South African Institute of Engineering and Environmental Geologists
TLB	Tractor Loader Backhoe
UCS	Unconfined Compressive Strength
uMWP	uMkhomazi Water Project
uMWP-1	uMkhomazi Water Project – Phase 1
uMWP-2	uMkhomazi Water Project – Phase 2

USCS	Unified Soil Classification System

LIST OF UNITS

a	annum
ha	hectare
hrs	hours
km	kilometre
km^2	square kilometre
kW/m^2	kilowatt per square metre
ℓ	litre
ℓ/cap/day	litre per capita per day
m	metre
m/s	metre per second
m^3/s	cubic metre per second
masl	metre above sea level
million m^3	million cubic metre
million m^3/a	million cubic metres per annum
Mℓ/day	mega litre per day
mm	millimetre
MW	megawatt
Ø	diameter in millimetres
s	second
kℓ	kilolitre
ℓ/c/d	litre per capita per day
m^3	cubic metre

1 INTRODUCTION

The Department of Water Affairs appointed **BKS (Pty) Ltd** in association with three sub-consultants **Africa Geo-Environmental Services, MM&A and Urban-Econ** with effect from 1 December 2011 to undertake the **uMkhomazi Water Project Phase 1: Module 1: Technical Feasibility Study Raw Water** study.

On 1 November 2012, BKS (Pty) Ltd was acquired by **AECOM Technology Corporation**. As a result of the change in name and ownership of the company during the study period, all the final study reports will be published under the AECOM name.

In 2010, the Department of Arts and Culture published a list of name changes in the Government Gazette (GG No 33584, 1 October 2010). In this list, the Mkomazi River's name was changed to the **uMkhomazi River**. *The published spelling will thus be used throughout this technical feasibility study.*

1.1 BACKGROUND TO THE PROJECT

The current water resources of the Mgeni system are insufficient to meet the long-term water requirements of the system. The Mgeni System is the main water source that supplies about five million people and industries in the eThekwini Municipality, uMgungundlovu District Municipality (DM) and Msunduzi Local Municipality (LM), all of which comprise the economic powerhouse of the KwaZulu-Natal Province.

The Mgeni System comprises the Midmar, Albert Falls, Nagle and Inanda Dams in KwaZulu-Natal, a water transfer scheme from the Mooi River and the newly constructed Spring Grove Dam. The current system (Midmar, Albert Falls, Nagle and Inanda Dams and the MMTS-1) has a stochastic yield of 334 million m³/annum (measured at Inanda Dam) at a 99% assurance of supply. The short-term augmentation measure, Phase 2 of the Mooi Mgeni Transfer Scheme (MMTS-2), currently being implemented with the construction of Spring Grove Dam, will increase water supply from the Mgeni system by 60 million m³/year. However, this will not be sufficient to meet the long-term water requirements of the system.

Pre-feasibility investigations indicated that Phase 1 of the uMkhomazi Water Project (uMWP-1), which entails the transfer of water from the undeveloped uMkhomazi River to the existing Mgeni system, is the scheme most likely to fulfil this requirement. The uMkhomazi River is the third-largest river in KwaZulu-Natal in terms of mean annual runoff (MAR).

Eight alternative schemes were initially identified as possible alternatives, and the Impendle and Smithfield scheme configurations have emerged as suitable for further investigation. The pre-feasibility investigation, concluded in 1998, recommended that the Smithfield scheme be taken to a detailed feasibility-level investigation as its transfer conveyances would be independent of the existing Mgeni System, thus reducing the risk of limited or non-supply to eThekwini and some areas of Pietermaritzburg, and providing a back-up to the Mgeni System.

The ***Mkomazi-Mgeni Transfer Pre-feasibility Study*** concluded that the first phase of the uMWP would comprise a new dam at Smithfield on the uMkhomazi River near Richmond, a multi-level intake tower and pump station, a water transfer pipeline/tunnel to a balancing dam at Baynesfield Dam or a similar in-stream dam, a water treatment works at Baynesfield in the uMlaza River valley and a gravity pipeline to the Umgeni bulk distribution reservoir system, below the reservoir at Umlaas Road. From here, water will be distributed under gravity to eThekwini and possibly low-lying areas of Pietermaritzburg. Phase two of the uMWP may be implemented when needed, and could comprise the construction of a large dam at Impendle further upstream on the uMkhomazi River to release water to the downstream Smithfield Dam. Together, these developments have been identified as having a 99% assured stochastic yield of about 388 million m^3/year.

The DWA aims to have this scheme implemented by 2022.

1.2 STUDY AREA

The study focus and key objective is related to the feasibility investigation of the Smithfield Dam and related raw water conveyance infrastructure. However, this is a multi-disciplinary project with the study area defined as the uMkhomazi River catchment, stretching to the north to include the uMngeni River catchment. (refer to **Figure A1.1**). The various tasks have specific focus area, defined as:

- Water Resources: uMkhomazi and Mgeni River catchments;
- Water requirements: Water users in the Mgeni System and the uMkhomazi River catchment;

- Engineering investigations: Proposed dams at Impendle (only for costing purposes) and Smithfield, and the raw water conveyance infrastructure corridor between Smithfield Dam and the Water Treatment Plant of Umgeni Water;
- Environmental screening as input for the Environmental Impact Assessment; and
- Socio-economic impact assessment: Regional, provincial (KwaZulu-Natal (KZN)) and national.

1.3 OBJECTIVE, SCOPE AND ORGANISATION OF THE STUDY

According to the Terms of Reference (November 2010), the objective of the study project is to undertake a feasibility study to finalise the planning of the proposed uMkhomazi Water Project (uMWP) at a very detailed level for the scheme to be accurately compared with other possible alternatives and be ready for implementation (detailed design and construction) on completion of the study.

The feasibility study has been divided into the following modules, which will run concurrently:

- Module 1: Technical Feasibility Raw Water (DWA) (*defined below*);
- Module 2: Environmental Impact Assessment (DWA); and
- Module 3: Technical Feasibility Potable Water (Umgeni Water) (*ranging from the Water Treatment Plant to the tie-in point with the eThekwini distribution system*).

This module, the raw water technical feasibility study, considers water resources aspects, engineering investigations and project planning and scheduling and implementation tasks, as well as an environmental screening and assessment of socio-economic impacts of the proposed project.

Some specific objectives for this study, recommended in the Mkomazi-Mgeni Transfer Scheme Pre-feasibility are listed below:

- Smithfield Dam (Phase 1) to be investigated to a detailed feasibility level;
- Investigate the availability of water from Impendle Dam (Phase 2) as a future resource to release to Smithfield Dam, and refine the phasing of the selected schemes;
- Optimise the conveyance system between Smithfield Dam and the proposed Baynesfield Water Treatment Plant;
- Undertake a water resources assessment of the uMkhomazi River catchment, including water availability to the lower uMkhomazi;

- Evaluate the use of Baynesfield Dam as a balancing dam; and
- Investigate the social and economic impact of the uMWP.

This one of three studies was undertaken in close collaboration with the DWA, Umgeni Water and the Professional Services Providers (PSPs) of the other modules.

1.4 Scope of this Report

This report deals exclusively with the construction materials and geotechnical investigations conducted at the Smithfield Dam site.

The activities specific to the ***construction materials and geotechnical task*** included:

- Review the available geotechnical information.
- Describe the general geology of the area.
- Conduct seismic refraction surveys along and adjacent to the centre line of the Smithfield Dam site, the saddle dam site, the diversion tunnels and across the potential quarry site.
- Conduct a site specific probabilistic seismic risk analysis for the Smithfield Dam site area.
- Conduct additional investigations for sources of dam construction materials by means of test pitting rotary core drilling and laboratory testing,
- Undertake additional geotechnical investigations for the foundations of the dams and spillway structures by means of rotary core drilling and Lugeon water pressure testing.
- Assess the stability of slopes around the reservoir rim.

1.5 Purpose of the Report

The objectives of this report are to describe the results of construction materials and geotechnical investigations for the following components of the uMkhomazi Water Project:

- Smithfield Dam site B as defined in ***Supporting Document 1*** on the Engineering Investigations (AECOM, et al., 2014).
- An earth core rockfill dam with approach channel and side spillway with chute and plunge pool on the left flank and diversion tunnels through the right flank.
- Alternatively, a composite dam with a central RCC gravity spillway section.

- Alternatively, a zoned embankment dam with approach channel and side spillway with chute and plunge pool on the left flank and diversion tunnels through the right flank.
- Alternatively, a concrete-faced rockfill dam.
- An earth core rockfill dam or zoned embankment dam across the saddle on the left flank.
- Construction material sources for the above types of dams.
- Concrete aggregate for the inlet section of the conveyance tunnel.

The report presents the site conditions, methodology and results of the geological and materials investigation task of the study. The information is based on a desktop study and field investigations conducted between January and April 2013. The seismic survey report was completed in April 2013 and laboratory testing was completed in May 2013.

The information from this report is required input into the engineering, financial and institutional investigations tasks, and this report is thus a supporting report to the main study report.

1.6 Approach of the Investigation

The approach of the investigation was as follows:

- To investigate all sources of natural dam construction materials located within an area of about 3 km from the dam and below the proposed FSL of 930 masl.
- To investigate foundation conditions along the dam centre line taking into account the requirements for various types of dams.
- Once it was established that there was not sufficient semi-pervious soil for construction of a zoned embankment, but sufficient impervious material for a clay core, the investigation was focussed on the materials and foundation requirements for a clay core rockfill dam and alternatively, a composite dam with rockfill flanks and a central RCC gravity spillway section.

1.7 Contents of the Report

This report is the ***Supporting Document 3*** for the main Geotechnical Report and refers to ***Supporting Document 1*** on the Seismic Hazard Assessment (AECOM, et al., 2014) and ***Supporting Document 2*** on the Seismic Refraction Survey (AECOM, et al., 2014).

This report comprises of the following three volumes:

- Volume 1: Text, as well as **Annexure B** containing the test pit profiles and the results of laboratory tests and **Annexure D** that includes photographs of the Smithfield reservoir rim.
- Volume 2: **Annexure A** containing all the figures that are referred to in the text of Volume 1.
- Volume 3: **Annexure C** containing the drilling information, i.e. driller's journals, water tests results, core logs and core photographs.

2 PREVIOUS INVESTIGATIONS

2.1 GENERAL

Since 1997, various options for the augmentation of the Mngeni River system from the uMkhomazi River had been considered by DWAF and Umgeni Water. These included a dam at Impendle with a 35 km tunnel to Midmar or a dam at Smithfield with a 25 km tunnel to Richmond or a 32 km tunnel to Baynesfield.

Previous geotechnical investigations for the Smithfield Dam and the Smithfield-Baynesfield Tunnel were conducted by the Council for Geoscience and the following reports are available:

- Davis, G.N. (1997) ***Upper Mkomazi-Mgeni Transfer Scheme: Smithfield Dam Sites & Transfer Tunnel Alignments: Impendle and Polela Districts.*** First Engineering Geological Reconnaissance Report. Unpublished Council for Geoscience Report 1997-0118 (Davis, GN, 1997).

- Davis, G.N. (1998) ***Mkomazi-Mgeni Transfer Scheme: Smithfield Dam Site: Impendle and Polela Districts.*** First Engineering Geological Pre-Feasibility Report. Unpublished Council for Geoscience Report 1998-0037 (Davis, GN, 1998).

In the reconnaissance report, four alternative centrelines for the Smithfield Dam, namely A, B, C and the Upper Site have been considered, while the pre-feasibility investigation deals with the favoured Upper Site only. The Upper Site is the same site as the one dealt with in this report.

The general geology of the area is described in the previous reports as follows:

The area of interest is underlain by sedimentary strata of the Karoo Supergroup which were subsequently intruded by younger dolerites in the form of sills and dykes.

More specifically, the dam sites are underlain by rocks of the Volksrust Formation of the Ecca Group, comprising siltstones (mudrocks) with this, subordinate sandstones. The sedimentary strata are essentially horizontal, and largely undisturbed. Regional dips of 3 – 7 degrees are recorded however; while locally-disturbed horizons are recognised in places and are ascribed to the intrusion of dolerites.

Although faults occur, no major faults have been mapped in the vicinity of the dam sites. Several prominent lineations are recognised in the vicinity of the dam sites. The pronounced NW-striking orientation is common to the preferred orientation exhibited on a regional scale by the dolerite dykes, considered to represent weakness zones in the earth’s crust.

Neither the rocks of the Volksrust Formation nor the intrusive dolerites which underlie the dam basin are typically associated with economically-important mineral deposits.

2.2 GEOTECHNICAL INVESTIGATIONS

During the pre-feasibility investigations, four cored boreholes (total 93.88m) were drilled along the Upper Site centreline where a dam of about 50 m high (FSL at 910 masl) was considered. The results of the pre-feasibility investigation have been summarised as follows (Davis, GN, 1998):

The dam site is roughly symmetrical. On either side of the 60 m wide river section, the flanks rise relatively steeply to a height roughly 25 – 30 m above river level; above which the gradient is much flatter.

The steeper slopes on the respective flanks are underlain by a 25 m thick dolerite sill. The upper slopes are underlain by siltstone; as in the river section.

On the left flank the upper slopes are covered by shallow colluvial soils (1 – 2 m). The underlying siltstone is weathered, and jointing is closely to very closely spaced. The medium to widely jointed dolerite sill comprises unweathered high strength rock.

Unconsolidated overburden within the river section is negligible. The indurated siltstone within the river section is generally unweathered, and joints are either tight or contain occasional calcite fill. A minor dolerite sill (width 5.5 m) is intrusive into the siltstone, occurring at a depth of 21 m.

The dolerite sill underlying the steeper slopes of the right flank is slightly weathered and widely to very widely jointed. Soil and boulder cover here may vary in thickness up to 1 – 4 m. On the upper right flank the transported soil cover attains a thickness in excess of 12 m, and comprises clayey colluvium overlying alluvium of an earlier river terrace. The underlying bedrock comprises weathered siltstone.

An embankment dam may be considered for this site, either with a central concrete spillway or a side-channel spillway on the left flank. Another option could be a mass concrete wall extending from the left flank to the steeper slopes of the right flank. Geological conditions on the upper right flank do not favour the construction of a concrete wall. For river diversion works during construction, of an embankment dam, either a culvert or a diversion tunnel may be considered.

For a concrete structure, an excavation depth of 7,5 m may be assumed within the river section. On the steep flanks underlain by the dolerite sill depths of 2 – 5 m may be assumed. Excavation depths up to 9 m may be assumed for the upper left flank.

For an embankment dam, a cut-off to depths of 2.5 – 3 m may generally be envisaged. On the steeper slopes however, the boulder horizon may require removal to depths of up to 2 – 5 m. Within the river section excavation depths of 3 – 4 m would allow founding beneath a weathered siltstone horizon. Attention is drawn here to the upper right flank where a presumably pervious alluvial horizon occurs at a depth of 9 m.

Water pressure (Lugeon) tests conducted in the exploratory boreholes indicate almost impermeable founding conditions within the bedrock. However, the dolerite sill contacts are typically open and occasional stained. Foundation grouting should, at the very least, intersect these contact zones. Indications are that the 3 m thick alluvial horizon on the upper right flank is relatively pervious.

Properties of the founding materials have not been determined as yet. Typically UCS values >200 MPa may be expected for the dolerite, while UCS values between 50 – 100 MPa may be expected for unweathered siltstones. Sub-horizontal discontinuities within the siltstone represent zones of w weakness. The siltstones must be expected to be susceptible to slaking.

2.3 MATERIALS INVESTIGATIONS

No previous investigations for sources of concrete aggregate, rip-rap and rockfill had been conducted. The report by (Davis, GN, 1998) mentions that the 25 m thick dolerite sill represents a potential source of these materials.

No suitable sources of sand for fine aggregate or filters were identified in the vicinity of the dam site.

Two potential borrow areas for impervious core materials were briefly investigated, primarily in order to confirm the availability of suitable materials. These areas are located respectively roughly 1000 m and 2000 m upstream of the proposed dam site. The results of these investigations are summarised as follows (Davis, GN, 1998):

At the closer of the areas (Area A) the materials comprise residual dolerite soils, basic laboratory testing reveals high clay contents and Atterberg constants in the upper boundary of acceptable limits. At Borrow Area B, the available materials predominantly comprise alluvial clays, aside from occasional high clay contents; these materials satisfy the requirements in terms of grading and Atterberg constants. More detailed testing would however be required at a later stage. Volume calculations indicate sufficient material is available.

3 PRESENT INVESTIGATIONS

3.1 NEED FOR ADDITIONAL GEOTECHNICAL INVESTIGATIONS

Based on a study of the available information, AECOM identified the need for the following additional geotechnical investigations:

3.1.1 Geological mapping

Geological mapping of the Smithfield Dam site based on the observation of surface outcrops and the results of core drilling and test pitting must be done.

3.1.2 Seismic risk analysis

A site-specific probabilistic seismic hazard analysis must be undertaken.

3.1.3 Seismic refraction survey

During the previous investigation of the Smithfield site no seismic refraction surveys had been conducted, and the gaps between boreholes along the dam centre line are too large to enable reliable interpretation of founding depths. It was considered necessary to conduct a seismic line along the proposed dam centre line to interpret founding conditions between previous and additional boreholes. The seismic survey will also ensure that further boreholes are drilled at the most appropriate locations. Seismic must also be conducted along the proposed alignments for the saddle embankment, the diversion tunnels and along the proposed spillway chute on the left flank.

Before drilling was undertaken to investigate the proposed quarry site upstream of the dam on the left side of the river, it was necessary to conduct seismic surveys to establish the geological conditions (overburden thickness and shale/dolerite contact) and to ensure that boreholes are located at appropriate locations.

3.1.4 Test pitting, soil sampling and laboratory testing

The previous soil surveys did not provide conclusive information on the availability of embankment materials, particularly for impervious core material. AECOM recommended that a 20 ton traxcavator be used to dig deeper (up to 5 m) test pits within the dam basin. The test pits has to be geotechnically logged and representative samples taken for additional laboratory testing. Laboratory tests must include grading, Atterberg Limits, Proctor compaction, permeability and dispersivity testing.

Samples of rock must be subjected to tests for rockfill.

3.1.5 Core drilling

Only a limited amount of information was available from previous core drilling along the Smithfield Dam centre line, and it was considered necessary to drill additional boreholes and conduct water pressure testing to assess the necessary excavation depths, the permeability and the need for grouting. The drilling of additional cored holes on the left flank was required to investigate foundation conditions for a spillway structure, while drilling at the portals for the proposed diversion tunnels and along the centre line of the proposed saddle dam was also necessary. Boreholes within the proposed approach channel and in the stilling basin were required to investigate the suitability of excavated material for dam construction.

Core drilling was also required to investigate the quantity and quality of material in the proposed quarry on the left side of the river and along a proposed alternative spillway channel on the left side of the saddle embankment.

The investigation also considered the need for concrete aggregates for lining the inlet portion of the conveyance tunnel.

Drilling information from the tunnel inlet portal area can be used to assess the suitability of excavated material for dam construction.

3.1.6 Reservoir slope stability

Slopes within the dam basin must be studied in order to determine the risk of failures and the effect of such failures on the dam structure.

3.2 GEOLOGICAL MAPPING

A regional geological map of the area around the site for the proposed Smithfield Dam was compiled from information on the published Geological Map, while the geology of the dam site and quarry areas was based on surface geological mapping, the seismic survey, test pitting and core drilling.

3.3 SEISMIC HAZARD ANALYSIS

A Probabilistic Seismic Hazard Analysis (PSHA) was conducted by Dr A Kijko of the Natural Hazards Assessment Consultancy in Centurion (***Report no P WMA 11/U10/00/3312/3/2/1: Supporting document 1: Probabilistic hazard analysis for Smithfield Dam, Langa Balancing Dam and the conveyance system***).

The PHSA was performed using conventional Cornell-McGuire procedures where the integration across the uncertainty in the peak ground acceleration (PGA) prediction equation is an integral part of the methodology.

In accordance with current seismic regulations provided in Bulletin 72 of ICOLD (1989), Eurocode 8 (2004) and ASCE (2005), three seismic designated levels were considered namely the Operating Basis Earthquake (OBE), Maximum Design Earthquake (MDE) and Maximum Credible Earthquake (MCE).

Results for the horizontal component of earthquake acceleration are as follows:

- Operating Basis Earthquake (Return period 144 years) =0.016 g
- Maximum Design Earthquake (Return Period 475 years) =0.021 g
- Maximum Credible Earthquake (Return period 10 000 years) =0.113 g

The above results classify the site as of low seismic risk.

3.4 SEISMIC REFRACTION SURVEY

Seismic refraction surveys were undertaken by Open Ground Resources to determine the succession of seismic velocity layers and depth to sound bedrock at the following locations (***Report no P WMA 11/U10/00/3312/3/2/2: Supporting document 2: Seismic refraction investigation at the proposed uMkhomazi Water Project Phase 1***):

- Smithfield Dam site left flank: 415 m
- Smithfield Dam site right flank: 785 m
- Smithfield Dam site left spillway: 595 m
- Smithfield Diversion Tunnel line 1: 235 m
- Smithfield Diversion Tunnel line 2: 335 m
- Smithfield Saddle embankment site: 595 m
- Smithfield Saddle spillway site: 595 m

- Smithfield Quarry Site Upper line: 715 m
- Smithfield Quarry Site Lower line: 595 m
- Smithfield Quarry Site Cross line: 355 m
- Tunnel inlet portal site: 355 m

3.5 TEST PITS

Test pits were excavated to investigate the subsurface conditions to a maximum depth of about 5 m. A 20-ton excavator was used (instead of a TLB) to speed up the work and to penetrate possible zones with boulders and hardpan ferricrete.

Table 3.1 provides details of 44 test pits that were excavated in 3 proposed borrow areas during the present materials investigation.

Table 3.1: Summary of test pit excavations

Project area	Number of test pits	Test pit numbers
Smithfield Dam Borrow Area A	21	TPA 6 – TPA27
Smithfield Dam Borrow Area B	17	TPB5 –TPB11, TPB14-TPB16, TPE1 – TPE6
Smithfield Dam Borrow Area C	6	TPC1 –TPC6

* *Note: Borrow area C was later incorporated in Quarry I*

The test pits were excavated to a maximum reach of machine or to partial refusal (caused by dense/stiff material), whichever occurred first. Test pits were profiled by an engineering geologist according to the current standards and practice in South Africa (SAIEG-AEG-SAICE (Geotechnical Division), 1990). Representative samples were taken for laboratory testing.

Immediately after completing the profiling and sampling of each test pit, the excavated material was placed back into the hole and compacted in layers. Material from the upper fertile soil layer was kept separate and placed on top of the backfilled material. The ground surface was restored as near as possible to its original condition.

Each test pit was positioned using a hand-held GPS. **Figure A5.1** shows the locations of the borrow areas and test pits.

3.6 ROTARY CORE DRILLING

Rotary core drilling was undertaken to obtain relatively undisturbed samples of soil and rock to depths of 20 m - 40 m below ground surface and to conduct packer permeability tests to these depths along the dam centre line.

Borehole cores were profiled by an engineering geologist according to the current standards and practice in South Africa (SAIEG-AEG-SAICE (Geotechnical Division), 1990).

Water pressure testing was carried out within the boreholes along the dam centreline to assess the permeability of the foundation materials. Special provision was made in the drilling contract for water pressure (Lugeon) testing in weathered rock conditions where conventional packers cannot be used. In these materials the packers were seated within a tight-fitted (drilled-in) casing so that packer tests could be conducted from about 1.5 m depth.

Samples comprising completely, highly, moderately and slightly weathered shale from boreholes on the proposed Smithfield quarry were taken for laboratory tests to determine their properties with respect to use as soft rockfill.

During the investigation by (Davis, GN, 1998) four boreholes were drilled along the dam centre line. Detailed core logs and core photographs of these boreholes are available and the results have been incorporated with those from the present investigation.

Table 3.2 summarises the distribution of the core-drilled boreholes. Positions of boreholes were surveyed to an accuracy of 200 mm both horizontally and vertically. The positions of previously drilled boreholes as well as boreholes drilled for the present investigation are shown on **Figure A5.2**.

Table 3.2: Summary of cored boreholes

Project area	Number of boreholes	Borehole no's
Smithfield Dam centre line left flank	6	DLS 1-DLS 3, DL 1 , DL 3, DL 4
Smithfield Dam centre line right flank	7	DRS 1-DRS 3,DR 1-DR 4
Smithfield Dam left spillway chute	4	DSS 1-DSS 3, DS 5
Smithfield Dam left stilling pool	4	DS 4, DS 6 – DS 8
Smithfield Dam left approach area	2	DS 1-DS 2
Smithfield left bank quarry	20	QLS 1-QLS 3, QLS 5-QLS 9, QL 1-QL 12
Smithfield saddle dam centre line	3	SES 1-SES 3
Smithfield saddle spillway	4	SSS 1-SSS 4
Smithfield diversion tunnels	5	DT1, 2 & 5, DTS1-DTS2

* *Note: Boreholes with suffix "S" (E.G. DLS 1) in borehole number denotes hole that was set out based on the results of the seismic survey.*

Borehole cores were profiled by an engineering geologist according to the current standards and practice in South Africa (SAIEG-AEG-SAICE (Geotechnical Division), 1990).

At the end of the drilling investigation, all the borehole cores were transported to the DWA offices at Midmar Dam where the core boxes were stacked in a dedicated carport.

The results of the drilling (Driller's Journals, Borehole Logs, Core Photographs and Water Test Results) are included in **Annexure C**.

3.7 Laboratory Testing

Samples comprising completely, highly, moderately and slightly weathered shale from boreholes at the Smithfield site were taken to Geostrada for the following laboratory tests to determine their properties with respect to their use as soft rockfill or alternatively as semi-pervious fill (the completely and highly weathered shale). The following tests were conducted:

- Primary crushing to minus 19 mm;
- Grading analysis, including hydrometer determination of the clay fraction;
- Determination of the Atterberg Limits, where possible;
- Aggregate crushing values (wet and dry);
- Secondary crushing to minus 0,425 mm; and
- Standard Proctor Compaction and Triaxial Shear Testing.

Samples of slightly weathered and unweathered shale and dolerite from the tunnel line investigation were tested for Unconfined Compressive Strength and various E-Moduli by Rocklab.

The results of the laboratory tests are contained in **Annexure B** of this report.

3.8 RESERVOIR SLOPE STABILITY STUDY

See **Section 7** for details on the analysis of the reservoir rim stability.

4 GEOLOGY

The area of interest is underlain by sedimentary strata of the Karoo Supergroup which were subsequently intruded by younger dolerites in the form of sills and dykes (**Figure A4.1**).

The dam sites are underlain by rocks of the Volksrust Formation of the Ecca Group, comprising shale (termed siltstones or mudrocks by (Davis, GN, 1998) with sub-ordinate sandstones. The sedimentary strata are essentially horizontal, and largely undisturbed. Regional dips of 3 – 7 degrees are recorded however; while locally-disturbed horizons are recognised in places and are ascribed to the intrusion of dolerites.

Several prominent lineations are recognised in the vicinity of the dam sites. The pronounced NW-striking orientation is common to the preferred orientation exhibited on a regional scale by the dolerite dykes, considered to represent weakness zones in the earth's crust.

Neither the rocks of the Volksrust Formation nor the intrusive dolerites which underlie the dam basin are typically associated with economically-important mineral deposits.

At the dam site the shales had been intruded by at least three near-horizontal dolerite sills (**Figure A4.2**). The upper sill occurs between elevations 946 m and 920 m and is separated from the main sill by about 20 m of shale. It occurs in the ridge above the left flank and also at the top of the right flank. The main sill varies in thickness between about 18 m and 28 m and occurs along the dam centre line and in the main quarry area where it is largely covered by shale. Upstream of the dam on the left flank, the sill had been displaced downwards along a fault by about 10 m. This fault possibly intersects the dam centre line along the lower right flank. On the right flank the main sill bifurcates in the area of the tunnel inlet portals and thin layer of dolerite crops out in the slope. The main sill is covered by shale over most of the right flank. The third sill is only about 6 m thick and occurs between elevations 820 masl and 828 masl below the centre line.

The shale had been indurated (baked) by the very hot dolerite intrusions. The effect of induration was to strengthen the shale and make it more durable, while the extent from the contact with the dolerite is variable from one or two metres to many metres.

Above elevation 890 m, the right flank is covered by remnants of an old high level river terrace deposited when the river ran straight across the present nose of the right flank. The bottom part of these transported materials is a mixture of alluvial boulders, clay and silt, while the upper part comprises of colluvial clay and silt.

5 CONSTRUCTION MATERIALS INVESTIGATION

During the present feasibility stage of the project, various types of dams are being considered, and therefore the available quantities and properties of different types of construction materials were investigated. The aim of the construction materials investigation was to locate sources of natural construction materials (soil and rock) in environmentally suitable locations, capable of providing the volumes as shown in **Table 5.1**. These quantities include the normal “safety factor” whereby twice the volume of material required for construction, is proved during the site investigation. They apply to a dam with FSL at 930 masl.

Table 5.1: Approximate volumes of construction material required

	TYPE OF MATERIAL (m^3)					
Structure	**Impervious core**	**Semi-pervious**	**Soft rockfill**	**Hard Rockfill**	**Rip-rap**	**Rock for aggregate, filters, drains**
Main zoned embankment dam	1 800 000	8 800 000	0	0	320 000	800 000
Main rockfill dam	1 800 000	0	0	8 000 000	0	422 000
Main earth-rock dam*	1 800 000	0	4 000 000	4 000 000	0	422 000
Main concrete gravity dam	0	0	0	0	0	1 600 000
Saddle zoned embankment dam	570 000	1 600 000	0	0	0	0
Saddle rockfill dam	415 000	0	0	1 250 000	50 000	100 000
Saddle earth-rock dam*	415 000	0	600 000	650 000		100 000
Tunnel lining and Intake structure	0	0	0	0	0	44 000

* *Actual volumes will depend on the design and method of quarry development.*

A typical specification for earth embankment materials is given in **Table 5.2**.

Table 5.2: Specification for earth fill materials (Badenhorst, DB, 1988)

Property	Embankment zones		
	Impervious	Semi-pervious	Pervious
Clay content (%)	10-30	<25	<10
PI (%)	12-35	<12	<5
LL (%)	30-60	<30	<20
LS (%)	6-10	<7	<2
Standard Proctor MDD (kg/m^3)	1450-1880	1750-2000	1700-2100
Standard Proctor OMC (%)	12-25	10-15	6-12
Cohesion (kPa)	12-30	8-15	<10
Friction angle (^{0})	18-30	28-38	>35
Permeability (m/sec)	$<1x10^{-8}$	$1x10^{-7} – 1x10^{-5}$	$>1x10^{-5}$

5.1 EARTH FILL MATERIALS

5.1.1 Previous investigations

Two potential borrow areas (Area A and Area B) for impervious core materials had briefly been investigated during the pre-feasibility investigation (Davis, GN, 1998). These areas are located respectively about 1 000 m and 2 000 m upstream of the proposed dam site on the left side of the river, and are below the presently proposed (early 2013) FSL of 930 masl.

Five TLB test pits (A1 – A5) were dug in Area A and three test pits (B1 – B3) in Area B. Eleven samples were tested for Grading and Atterberg Limits.

It was concluded that about 1.5 million m^3 of usable impervious core material was available in Borrow Areas A and B.

The results of these test pits are included in the following sections.

5.1.2 Test pits

During the present investigation, a number of additional test pits were excavated in Area A (21 pits), Area B (10 pits plus 6 pits at the tunnel inlet portal), and Area C (6 pits) located to the south of Area A. These pits were dug by means of a 20 ton tracked excavator with a maximum reach of about 5 m depth. The purpose of this investigation was to confirm the previous findings in terms of the quantity and quality of the materials, and to prove an additional volume of about 300 000 m^3 required for the larger dam now under consideration. The positions of all test pits are shown on **Figure A5.1**.

The detailed test pit logs are provided in **Annexure B**, and a summary of the results is presented in **Table 5.3**, **5.4** and **5.5**.

Table 5.3: Summary of test pits in Borrow Area A

Test Pit No.	LAYER THICKNESS (m)				COMMENTS EOR = End of Reach of excavator
	Organic soil	Clayey sand/silt A = Alluvium C = Colluvium P = Pedogenic ferricrete RD = Residual dolerite RS = Residual shale	Boulders and clayey silt C = Colluvium A = Alluvium	Soft rock P = pedogenic S = Shale D = Dolerite	
A1	0.0 – 0.4	0.4 – 2.4 (RD)			EOR
A2	0.0 – 0.4	0.0 – 1.0 (P) 1.0 – 2.4 (RD)			EOR
A3	0.0 – 0.4	0.4 – 0.9 (P) 0.9 – 2.15 (RD)			EOR
A4	0.0 – 0.65	0.65 – 2.75 (A)			EOR
A5	0.0 – 1.0	1.0 – 2.6 (A)			EOR
A6	0.0 – 0.3	0.3 – 2.0 (C) 2.0- 5.0 (RS)			EOR
A7	0.0 – 0.4	0.4 - 3.4 (C) 3.4 – 5.0 (RS)			EOR
A8	0.0 – 0.4		0.4 – 5 0 (C)		EOR
A9	0.0 – 0.4		0.4 – 4.9 (C)		EOR
A10	0.0 – 0.3	0.3 – 1.0 (C) 1.0 – 4.9 (RS)			EOR
A11	0.0 – 0.3	0.3 – 5.0 (C)			EOR
A12	0.0 – 1.1		1.1 – 5.0 (C)		EOR
A13	0.0 – 0.6		0.6 – 4.0 (C)	4+ (S)	EOR
A14	0.0 – 0.5	0.5 -0.9 (P) 0.90 – 5.0 (RS)			EOR
A15	0.0 – 0.3	0.3 – 1.4 (P) 1.4 – 4.4 (RS)		4.4+ (P)	Seepage 1.0m Refusal (P)
A16	0.0 – 0.3	0.3 – 1.0 (P) 1.0 – 5.1 (C)			Seepage 1.0m EOR
A17	0.0 – 1.0		1.0 – 5.0 (C)		EOR
A18	0.0 – 0.6	0.6 – 1.5 (P) 1.5 – 2.6 (RS)		2.6 – 4.0 (S)	EOR
A19	0.0 – 0.5	0.5 – 5.0 (RS)			EOR
A20	0.0 – 0.5		0.5 – 4.5 (A)		Refusal (D)
A21	0.0 – 0.3	0.3 – 1.2 (A)	1.2 – 4.0 (A)		Refusal (D)
A22	0.0 – 1.0	1.0 – 5.0 (RS)			EOR
A23	0.0 – 0.6		0.6 –2.0 (C)	2.0 – 2.8 (S	Refusal (S)
A24	0.0 – 1.0	1.0 – 4.0 (A)	4.0 – 5.0 (A)		EOR
A25	0.0 – 0.5	0.5 – 2.1 (C) 2.1 – 5.1 (RS)			EOR
A26	0.0 – 0.5	0.5 – 4.5 (RS)			EOR
A27	0.0 – 0.5	0.5 – 4.4 (RS)			Refusal (S)

Table 5.4: Summary of test pits in Borrow Area B

Test Pit No.	LAYER THICKNESS (m)				COMMENTS EOR = End of Reach of excavator
	Organic soil	Clayey sand/silt A = Alluvium C = Colluvium P = Pedogenic ferricrete RD = Residual dolerite RS = Residual shale	Boulders and clayey silt C = Colluvium A = Alluvium	Soft rock P = pedogenic S = Shale D = Dolerite	
B1	0.0 – 0.5	0.5 – 2.5 (A)			EOR
B2	0.0 – 0.4		0.4 – 1.9 (A)		EOR
B3	0.0 – 0.3	0.3 – 1.85 (A)			Seepage 0.9m EOR
B5	0.0 – 0.8	0.8 – 5.0 (C)			Seepage 3.0m EOR
B6	0.0 – 0.8	0.8 – 5.0 (C)			EOR
B7	0.0 – 0.6	0.6 – 2.6 (P)	2.6 – 5.0 (A)		EOR
B8	0.0 – 0.6	0.6 – 2.5 (P)	2.6 – 5.0 (A)		EOR
B9	0.0 – 1.2	1.2 – 5.0 (C)			Seepage 2.1m EOR
B10	0.0 – 0.6	0.6 – 5.0 (A)	5+(A)		Seepage 2.5m EOR
B11	0.0 – 0.6	4.2 – 5.1 (C)	0.6 – 4.2 (C)		Seepage 4.0m EOR
B14	0.0 – 1.0	1.0 – 2.2 (C)		2.2 – 3.5+ (S)	Seepage 2.5m Refusal (S)
B15	0.0 – 0.6	0.6 – 4.5 (C)	4.5 – 5.0 (C)		Seepage 3.5m EOR
B16	0.0 – 0.9	0.9 – 5.0 (C)			EOR
TPE1	0.0 – 0.5	0.0 -1.8 (C)	1.8 – 5.0 (C)		EOR
TPE2	0.0 – 0.3	0.3 – 5.0 (C)			Seepage 3.0m EOR
TPE3	0.0 – 0.5	0.5 – 1.05 (C)	1.05 – 5.0 (C)		Seepage 1.05m EOR
TPE4	0.0 – 0.5	0.5 – 2.8 (C)	2.8 – 5.0 (C)		Seepage 2.8m EOR
TPE5	0.0 – 1.1		1.1 – 2.8 (C)		Refusal (C)

Table 5.5: Summary of test pits in Borrow Area C

TP No.	LAYER THICKNESS (m)				COMMENTS EOR = End Of Reach of excavator
	Organic soil	**Clayey sand/silt** A = Alluvium C = Colluvium P = Pedogenic ferricrete RD = Residual dolerite RS = Residual shale	**Boulders and clayey silt** C = Colluvium A = Alluvium	**Soft rock** P = pedogenic S = Shale D = Dolerite	
C1	0.0 – 0.5	0.5 – 2.2 (RS)		2.2 – 3.0 (S)	Refusal (S)
C2	0.0 – 0.2			0.2 – 3.0 (S)	Refusal (S)
C3	0.0 – 0.5	0.5 – 1.2 (RS)		1.2 – 2.3 (S)	Refusal (S)
C4	0.0 – 0.5	0.5 – 1.4 (RS)		1.4 – 3.0 (S)	Refusal (S)
C5	0.0 – 0.3	0.3 – 1.0 (RS)		1.0 – 1.8 (S)	Refusal (S)
C6	0.0 – 0.3			0.3 – 2.8 (S)	Refusal (S)

5.1.3 Laboratory tests

Small samples for foundation indicator tests and duplicate large samples for compaction and other tests were taken from representative soil horizons encountered in the test pits.

Geostrada soils laboratory in Pretoria conducted the following tests:

- 13 Foundation indicator tests consisting of grading analysis to minus 0.002 mm, and determination of the Atterberg Limits.
- 2 Standard Proctor compaction tests to determine the Proctor Dry Density and the Optimum Moisture Content.
- 2 Double Hydrometer tests to determine the dispersiveness of the soil.
- 2 Flexible Wall Permeability tests.

a) Grading and Atterberg Test Results

The results of the Grading and Atterberg tests are summarised in **Table 5.6**, and the detailed results are provided in **Annexure B**[1]. The selection of samples for laboratory testing was based on the soil type (grain size), which was visually determined during the soil profiling. Since this determination is a rough estimate that is affected by the moisture condition of the soil, grain shape, grain composition and experience of the profiler, both the profile descriptions

[1] *The particle size distribution graphs show an abrupt step which is the correction between the mechanical sieve and hydrometer tests. This cannot be explained by the laboratory. It does, however, impact on the design of the embankments. However, this material must be re-tested during the detailed design phase.*

and the laboratory results (in italics) for grain sizes are given in the fourth column of **Table 5.6**.

All the samples described, except those from TPA7 and TPA27, comply with the specification for impervious core material. The PI's and Liquid Limits of TPA7 and TPA9 are slightly higher and will require attention during mixing; however they represent a small portion of the borrow area can still be used if mixed with material from the surrounding areas.

b) Standard Proctor compaction test results

Standard Proctor compaction tests were done on two samples made up by mixing material from TPA10, TPA17 and TPA19 (residual silty sand) and from TPA15, TPA22 and TPB9 (residual clayey silty sand). These two combinations are considered representative of the material types encountered in the investigated areas. The combined samples were also tested for Grading and Atterberg Limits. The test results are summarised in **Table 5.7**. The detailed results are included in **Annexure B**.

Table 5.6: Summary of laboratory foundation indicator test results

Site	Test Pit No.	Depth (m)	Visual Soil Profile Description (*Lab result in italics %*)	Soil Properties: Atterberg Limits (%) LL	PI	LS	USCS	PE
Smithfield Borrow Area A	TPA7	0.4 – 3.4	Residual shale: clayey silt *Silty (21%) clayey (37%) sand (42%)*	59	32	14.5	CH	High
	TPA9	1.0 – 5.0	Dolerite boulders in sandy silt *Silty (27%), sandy (33%) clay (35%)*	69	42	20	CH	Very High
	TPA10	1.0 – 4.9	Residual shale: silt *Silty (20%) gravelly (23%) sand (52%)*	45	18	9.5	SC	Low
	TPA11	0.5 – 5.0	Residual shale: clayey silt *Gravelly (15%) silt-clay (23%) sand (39%)*	44	26	13	CL	Medium
	TPA15	1.4 – 4.4	Residual shale: clayey silt *Clayey (26%) sandy (30%) silt (38)*	37	20	7	CL	Medium
	TPA17	1.0 – 5.0	Dolerite boulders in sandy silt *Silty (15) sandy (39%) gravel (39%)*	45	21	8.5	SC	Low
	TPA19	1.5 – 5.0	Residual shale: silt *Silty (15%) sand (79%)*	35	20	6	SC	Low
	TPA22	0.5 – 5.0	Residual shale: clayey silt *Clayey (21%) silty (24%) sand (53%)*	45	21	9.5	CL	Medium
	TPA27	0.5 – 4.4	Residual shale: clayey silt *Clayey (5%) silty (19%) sand (76)*	22	10	3.5	SC	Low
Smithfield Borrow Area B	TPB5	0.8 – 5.0	Residual shale: clayey silt *Clayey (26%) silty (34%) sand (38%)*	36	23	8	CL	Medium
	TPB7	0.6 – 2.6	Pedogenic clayey silt *Silty (16%) clayey (24) sand (44)*	42	14	6	ML	Low
	TPB9	1.5 – 5.0	Residual shale: clayey silt *Clayey (22%) silty (32%) sand (43%)*	46	19	7.5	CL	Medium
	TPB11	1.0 – 4.2	Residual shale: clayey silt *Clayey (18%) silty (26%) sand (45%)*	36	21	7	CL	Medium
Legend	LL		=	Liquid Limit				
	PI		=	Plasticity Index				
	LS		=	Linear Shrinkage				
	PE		=	Potential Expansiveness				
	USCS		=	Unified Soil Classification System				

Table 5.7: Summary of standard proctor compaction results

Site	Test Pit No.	Depth (m)	Soil Profile Description Based on Grading Analysis	USCS	Proctor Compaction	
					MDD (kg/m^3)	OMC (%)
Smithfield Borrow Areas A and B	TPA10 TPA17 TPA19	1.0 – 5.0	Clayey (23%) silty (23%) sand (39%)	SC	1825	14.3
	TPA15 TPA22 TPB9	1.0 – 5.0	Sandy (24%) silty (36%) clay (36%)	CL	1730	16.0
Legend:	USCS	=	Unified Soil Classification System			
	MDD	=	Maximum Dry Density			
	OMC	=	Optimum Moisture Content			

The compaction characteristics of both samples are according to the specification for impervious core material.

c) Double hydrometer test results

Double hydrometer tests were done on two samples made up by mixing material from TPA10, TPA17 and TPA19 (residual silty clayey sand) and from TPA15, TPA22 and TPB9 (residual sandy silty clay). These two combinations are considered representative of the material types encountered in the investigated areas. The combined samples were also tested for Grading and Atterberg Limits. The detailed results are included in **Annexure B**.

The silty clayey sand has a dispersion of 14% and can be considered non-dispersive, while the sandy silty clay has a dispersion of 32% and can be considered marginally dispersive. This is not a critical design aspect and can be dealt with by proper filter design and good compaction control during construction.

d) Permeability test results

Flexible wall permeability tests at a pressure differential of 10 kPa were done on two samples made up by mixing material from TPA10, TPA17 and TPA19 (residual silty clayey sand) and from TPA15, TPA22 and TPB9 (residual sandy silty clay). These two combinations are considered representative of the material types encountered in the investigated areas. The combined samples were also tested for Grading and Atterberg Limits and recompacted to their Proctor densities of 1825 kg/m^3 and 1730 kg/m^3 and Moisture contents of 14.3% and 16.0% respectively. The detailed results are included in **Annexure B**.

The measured permeabilities were as follows:

- Residual silty clayey sand: 8.5 x 10^{-12} m/s
- Residual sandy silty clay: 7.6 x 10^{-12} m/s

The measured permeabilities are much lower than the values typically obtained for CL and SC soils. However these samples were tested in a newly developed flexible wall permeameter with a consolidation pressure of 800 kPa (pressure exerted by about 40 m of soil) and one might expect a lower permeability. Also, from the grading curves, the D_{10} of the soils could be extrapolated as 0.0005 and 0.00002 mm, giving Hazen permeabilities of 2 x 10^{-7} m/s and 4 x 10^{-10} m/s respectively. These values render the materials suitable as impervious core.

5.1.4 Available volumes of earth fill materials

Based on the information from the test pits and laboratory testing, the areas with suitable impervious material were delineated and are shown on **Figure A5.1.** The estimated volumes of earthfill materials are given in **Table 5.8.** Area C is not included here since it overlaps with Quarry I and has less than 1 m of soil above soft rock.

Table 5.8: Estimated volumes of earthfill materials found (m^3)

Type of Material	Area A	Area B	Excavation for ECR Dam	Excavation for RCC dam	Excavation for saddle dam	Quarries I to IV (see Table 5-17)
Overburden for spoil: Organic topsoil	120 000	100 000	56 000	8 000	20 000	115 000
Impervious fill: Residual silty clayey sand and sandy silty clay	800 000	850 000	380 000	120 000	0	252 000
Semi-pervious fill: Completely and highly weathered shale	0	0	0	210 000	110 000	900 000

It appears that Borrow Areas A and B can provide about 1.65 million m^3 impervious material for an Earth Core Rockfill (ECR) or Zoned Embankment Dam. This is slightly more than found in the pre-feasibility investigation, but 720 000 m^3 short of the 2.37 million m^3 that has to be proved for the presently proposed main and saddle dams.

In Quarry I, a volume of about 600 000 m^3 of completely and highly weathered shale has to be removed before the hard shale and dolerite can be reached (see **Section 5.2.1** of this report). A large portion of this material will break down to a silty sand during construction and can possibly be used as a semi-pervious transition zone between the impervious core and the rockfill. In this way the volume of impervious material could possibly be reduced. Quarries II, III and IV might yield another 300 000 m^3 of similar material.

If Quarry II (plunge pool) is developed, another 200 000 m^3 of soil will have to be removed as overburden (see Section 5.2.2 of this report). The suitability of this material for use in an impervious zone must be further investigated.

Groundwater seepage was encountered in most of the test pits of Area B, but these pits were dug during a period of prolonged rainfall, and if construction is scheduled so that material from Area A is borrowed during the wet season and from Area B during the dry season, this potential problem could be overcome.

Not included in the above, are considerable volumes of clayey sand (SC) and clayey gravel (GC) to be excavated for the foundations of a rockfill dam on the right flank between chainages 690 m and 1170 m. This material might be suitable as semi-pervious fill for the construction of the saddle dam or parts of the main dam. Further sampling and testing of these materials will have to be done during the design stage.

5.1.5 Earthfill material parameters for design (borrow area material only)

From the results of the test pitting and the laboratory results it is estimated that about 50% of the soils in Borrow area A and B is silty, clayey sand (SC) and 50% sandy silty clay (CL). The average properties of these types of material are given in **Table 5.9**. From **Table 5.9** it is evident that both soil types are suitable for use as impervious fill.

Completely and highly weathered shale that covers the hard shale in the Quarry areas might be suitable as semi-pervious fill in a zoned embankment. Similar materials from the Zalu Dam site have previously been tested and their properties are also given in **Table 5.9** (see **Section 5.4.6.4**).

Table 5.9: Earthfill material properties for design

Material type	Properties of compacted material					
	Liquid Limit (%)	Plasticity Index (%)	Permeability (m/s)	MDD Density (Std Proctor) (kg/m^3)	OMC (%)	Dispersion (%)
Clayey sand (SC)	37 (30-60)	17 (12-35)	1×10^{-11} ($<1\times10^{-8}$)	1730 (1450-1880)	16.0 (12-25)	14 (<35)
Sandy silty clay (CL)	40 (30-60)	23 (12-35)	1×10^{-11} ($<1\times10^{-8}$)	1825 (1450-1880)	14.3 (12-25)	32 (<35)
Highly and completely weathered shale	27 <30	7 (4-12.5)	1×10^{-6} (1×10^{-7} – 1×10^{-5})	1960 (1750-2000)	13 (10-15)	n/t (-35)

(Specification in brackets)

5.2 ROCK MATERIALS

The volume of rock required for a concrete gravity dam is about 800 000 m^3 which means about 1.6 million m^3 of aggregate quality rock must be proved.

The volume of rockfill (including transition and filter zones) required for a Earth Core Rockfill Dam (ECRD) is about 4 million m^3 which means that about 8 million m^3 of rockfill material must be proved.

An additional volume of 22 000 m^3 of rock is required for coarse and fine (crushed rock) concrete aggregate for the tunnel (inlet half) and the intake structure.

The following potential sources for rock material have been identified (see **Figure A5.2**).

- Quarry Area I on the left flank, located below FSL of the dam.
- Quarry II at the proposed plunge pool excavation downstream of the dam on the left flank.
- Quarry III at the proposed spillway approach cut on the upper left flank.
- Quarry IV at the tunnel inlet portal.

5.2.1 Quarry I (Left flank)

Based on surface observations, a potential quarry area of about 450 000 m^2 was identified on the left side of the river (See **Figure A5.2**).

a) Seismic Refraction Survey

Open Ground Resources conducted two seismic lines (Q1Q2 = 595 m and Q3Q4 = 715 m) across the lower part of the proposed quarry and one line (Q5Q6 = 355 m) perpendicular to these (see ***Supporting Document 2*** (AECOM, et al., 2014)). The positions of the seismic lines are shown on **Figure A5.2**.

The 3 500 m/s seismic velocity line usually represents slightly weathered medium to closely jointed dolerite and has been drawn in on sections containing the results of boreholes (See **Figures A5.3, A5.4** and **A5.5**).

On **Figure A5.3**, the seismic line shows good correlation with the depth of sound dolerite in Borehole QLS1 but it over-estiymates the depth by about 5 m in Borehole QLS3. A seismic high between seismic chainages 230 m and 300 m might be indicative of a dolerite dyke intersecting the line just to the south of QLS2. Two seismic lows at seismic chainages 220 m and 540 m were used to interpret the positions of suspected faults.

On **Figure A5.4**, the seismic line over-estimates the depth of slightly weathered dolerite by 14 m in Borehole QLS7 and by 5 m at Borehole QLS5. Two seismic low anomalies at seismic chainages 245 m and 560 m were used to interpret the positions of the same suspected faults as above.

On **Figure A5.5** the seismic line appears to correspond with the depth of slightly weathered dolerite as interpreted from the boreholes. A seismic high at seismic chainage 190 m might be indicative of the same dolerite dyke as above, intersecting the line just to the west of QLS8.

After the initial drilling results became available, it turned out that most of the seismic work was done in the area to the north of a major east-west trending fault along which the main dolerite sill was thrown down so that the area is covered by a thick succession of shale (see **Figure A5.2**). Shallow dolerite that is more suitable for quarry development occurs to the south of the fault and this area was investigated further by means of boreholes (see Selected Quarry I in **Figure A5.2**).

b) Core drilling

A total of 20 cored boreholes (QL1 – QL12, QLS 1 – QLS3, and QLS5 – QLS9) were drilled in the area for the proposed quarry. The results of the drilling are summarised in **Table 5.10**.

Table 5.10: Results of drilling in Quarry I

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING O = Organic soils SC = Clayey sand or silt GC = Gravel in clay matrix S = Shale I = Indurated shale D = Dolerite					
	Transported Soil	Residual soil/ Completely Weathered	Highly Weath	Moderately Weathered	Slightly Weath.	Unweathered
	Layer thickness (m)					
QL 1	0.0 – 0.7 (GC)	0.7 – 1.64 (S)	1.64 – 5.6 (S)	5.6 – 12.6 (S) 16.1 – 16.5 (I)	12.6 – 16.1 (I) 16.5 – 17.56 (I)	17.56 – 30.07 (D)
QL 2	0.0 – 2.28 (GC) 2.28 – 7.32 (SC)		7.32 – 8.62(S)	8.62 – 16.04 (I)		16.04 – 30.1 (D)
QL 3	0.0 – 0.9 (O) 0.9 – 6.89 (SC) 6.89 – 7.2 (GC)	7.2 – 9.5 (S) 11.0 – 14.14 (S)	9.5 – 11.0 (S)	14.14 – 19.85 (I)		
QL 4	0.0 – 0.78 (O) 0.78 – 7.19 (GC)			7.19 – 10.47 (S) 13.16 – 20.22 (S)	10.47 – 13.16 (S)	
QL 5	0.0 – 0.85 (O)		0.85 – 2.3 (S)	2.3 – 3.81 (S)	3.81 – 9 98 (I)	9.98 – 20.0 (D)
QL 6			0.0 - 0.9 (S)	0.9 – 2.3 (S)	2.3 – 3.5 (D)	3.5 – 20.77 (D)
QL 7			0.0 – 3.2 (S)	3.2 – 4.5 (S)	4.5 – 7.5 (I)	7.5 – 30.57 (D)
QL 8			0.0 – 3.55 (S)		3.55 – 6.0 (D)	6.0 – 30.07 (D)
QL 9	0.0 – 0.2 (O) 0.2 – 1.5 (GC)	1.5 – 2.6 (S)	2.6 – 3.5 (S)	3.5 – 14.5 (S)	14.5 – 20.77	
QL 10	0. - 0.1 (O) 0.1 – 6.73 (GC)				6.73 – 20.1 (S)	
QL 11	0.0 – 2.35 (SC) 2.35 – 2.51 (GC)	2.51 -4.57 (S)	4.57 – 9.4 (S)	9.4 – 9.69 (S)	9.69 – 16.5 (I)	16.5 – 30.29 (D)
QL 12	0.0 – 0.3 (O)	0.3 – 0.53 (S)	0.53 – 1.3 (S)	1.3 – 4.6 (S)	4.6 – 6.82 6.82 – 7.5 (D)	7.5 – 30 05 (D)
QLS 1	0.0 – 0.36 (O) 0.36 – 1.45 (GC)	1.45 – 4.0 (D)	4.0 – 5.26 (D)	10.7 – 11.02 (D)	5.26 – 10.7 (D) 11.02 – 20.0 (D)	
QLS 2	0.0 – 1.0 (O) 1.0 – 7.38 (SC)	7.38 – 8.07 (I)		8.07 – 14.7 (I)	14.7 – 20.0 (D)	
QLS 3	0.0 – 1.3 (O) 1.3 – 3.57 (SC) 3.57 – 4.3 (GC)			4.3 – 5.5)S) 5.5 – 12.85 (I)	12.85 – 20.0 (D)	
QLS 5	0.0 – 0.5 (O)			0.5 - 14.55 (D)	14.55 - 17.48(D) 17.48 – 19.86 (I)	
QLS 6	0.0 – 0.5 (O) 0.5 – 2.0 (SC) 2.0 – 3.5 (GC)			3.5 – 12.1 (S)	12.1 –20.02 (S)	
QLS 7	0.0 – 0.9 (O) 0.9 – 7.2 (SC)	7.2 – 7.5 (S)	7.7 – 10.5 (S)	10.5 – 15.47 (D)	15.47 – 20.0 (D)	
QLS 8	0.0 – 0.77 (O) 0.77 – 2.45 (SC)	2.45 – 2.55 (S)	2.55 – 4.5 (S)	4.5 – 10.97 (I)	10.97 – 19.2 (I)	19.2 – 20.0 (D)
QLS 9	0.0 – 0.29 (O) 0.29 – 3.1 (GC) 3.1 – 5.85 (SC)			5.85 – 15.1 (S) 15.1 – 20.07 (I)		

The geology of the investigated area is illustrated on the geological map (**Figure A4.2**). The lower-lying northern part of the area is covered by a substantial thickness (2 – 4 m) of transported material (clayey sand, or gravel and cobbles in a matrix of clayey sand). Bedrock comprises a 10 m to 15 m succession of horizontally bedded shales overlying a thick dolerite sill with its top at about 860 masl or about river bed level. A major east-west trending fault displaced the dolerite sill upwards on its south side to between approximate elevations 870 m and 895 masl where it is closer to surface and easier to quarry. In the area to the south of the fault, the dolerite sill is still mostly covered by between 3 m and 15 m of shale that protects the dolerite against surface weathering. The shale is completely to highly weathered to an average depth of about 5 m and moderately weathered to an average depth of about 10 m. Along the dolerite contacts, the shale had been indurated to various degrees and for various distances away from the contact. The dolerite below the shale is generally slightly weathered to unweathered.

Based on the results of the seismic survey and the drilling, a quarry area of about 250 000 m^2 was selected in the southern part of the investigated area (see **Figure A5.2**).

The geology of the selected quarry area is illustrated by the sections on **Figures A5.6** to **A5.8**. It shows the presence of a 15 m to 25 m thick sill of slightly weathered to unweathered dolerite that is largely covered by shales in various degrees of weathering. The lowest level of the quarry floor is about 870 masl that is 10 m above river bed level.

5.2.2 Quarry II (Spillway plunge pool)

A spillway on the left flank will require a large plunge pool in an area where a thick dolerite sill occurs below a layer of shale. This offers the possibility of another quarry site.

a) Seismic refraction survey

Open Ground Resources conducted one seismic line (S1S2 = 595 m) along the proposed spillway return channel (chute) (see ***Supporting Document 2*** (AECOM, et al., 2014)). This line runs through the purposed Quarry II (see **Figure A5.2**).

The 3 500 m/s seismic velocity line usually represents slightly weathered medium to closely jointed dolerite and has been drawn in on the section containing also the results of boreholes (see **Figure A5.9**).

Figure A5.9 shows that the seismic line over-estimates the depth of slightly weathered dolerite by between 6 m and 14 m along the upper part of the traverse and then indicates a rapid increase in the depth of weathering along the steep slope close to the river. A low seismic velocity anomaly at seismic chainage 200 m might represent a fault or shear zone.

b) Core drilling

Eight boreholes (DSS1 – DSS3 and DS4 – DS8) were drilled in this area. BH 1001 from the pre-feasibility investigation provides additional information. The positions of boreholes are shown on **Figure A5.2**.

The results of the drilling are contained in **Annexure B** and are summarised in **Table 5.11**.

Table 5.11: Results of drilling in the plunge pool area

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING O = Organic soil SC = Clayey sand or silt GC = Gravel in clay matrix S = Shale I = Indurated shale D = Dolerite					
	Transported Soil	Residual soil/ Completely Weathered	Highly Weath	Moderately Weathered	Slightly Weath.	Un-Weathered
	Layer thickness (m)					
DSS1	0.0 – 0.5 (O)	0.5 – 4.36 (S)	4.36 – 11.65 (S) 17.4 – 17.7	11.65 – 16.55 (I) 11.65 – 17.4 (D)	17.7 – 20.17 (D)	0.0 – 0.5 (O)
DSS2	0.0 - 1.81 (O)	1.81 – 2.83 (D)			2.83 – 5.0 (D)	5.0 – 20.28 (D)
DSS3	0.0 – 0.13 (O)	5.32 – 14.6 (D) 18.27 – 19.02 (I)	0.13 – 0.7 (D) 15.1 – 16.7 (I)	14.6 – 15.1 (D) 16.7 - 18.27 (I) 19.02 – 22.25 (I)	0.7 – 5.32 (D)	0.0 – 0.13 (O)
DS4	0.0 – 0.7 (O)	0.7 – 1.2 (D) 2.85 – 5.8 (D) 9.9 – 12.0 (D)	7.63 – 8.66 (D) 12.0 – 17.38 (D) 17.38 - 20.2 (I)	8.66 – 9.9 (D) 20.2 – 27.0 (I)	1.2 – 2.85 (D) 5.8 – 7.63 (D) 27.0 – 30.0 I)	0.0 – 0.7 (O)
DS5	0.0 – 0.5 (O)	0.0 – 1.8 (S)	1.8 – 3.6 (S) 8.3 – 9.2 (S)	3.6 – 5.3 (S)	5.3 – 8.3 (S) 9.2 11.24 (S) 11.24 – 14.9 (I)	14.9 – 17.19 (I)
DS6	0.0 – 1.1 (O) 1.1 -1.74 (SC) 1.74 – 3.0 (GC)	17.16 – 17.87 (D)	3.0 5.1 (D)	5.1 – 7.55 (D) 24.98 – 29.17 (D)	7.1 – 17.16 (D) 17.87 – 24.98 (D) 29.17 - 40.15 (I)	0.0 – 1.1 (O) 1.1 -1.74 (SC) 1.74 – 3.0 (GC)
DS7	0.0 – 0.5 (O) 0.5 – 0.94 (GC)	0.94 – 11.65 (D)	29.4 – 30.0 (I)	11.65 – 12.75 (D) 22.0 – 25.8 (D) 25.8 - 29.4 (I)	12.75 – 22.0 (D)	0.0 – 0.5 (O) 0.5 – 0.94 (GC)
DS8	0.0 – 0.8 (O)	0.8 – 1.7 (S) 4.61 – 4.92 (S)	1.7 – 4.61 (S)	4.92 – 6.1	6.1 – 10.3 (D)	10.3 – 30.0 (D)

The main dolerite sill that underlies Quarry I extends across the dam centre line into the plunge pool area where it occurs between contours 870 m and 890 m (see **Figures A4.2**, **A5.9** and **A5.10**).

5.2.3 Quarry III (Spillway approach)

The preliminary design proposes a by-wash spillway on the upper left flank. This will require an approach cut excavated to elevation 927 masl as shown on **Figure A5.2**.

a) Core drilling

Boreholes DS1 and DS2 were drilled to investigate this area. The results of the drilling are summarised in **Table 5.12**.

Table 5.12: Results of drilling in the spillway approach area

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING O = Organic soil SC = Clayey sand or silt GC = Gravel in clay matrix S = Shale I = Indurated shale D = Dolerite					
	Transported Soil	Residual soil/ Completely Weathered	Highly Weath	Moderately Weathered	Slightly Weath.	Un-Weathered
	Layer thickness (m)					
DS 1	0.0 – 0.5	0.5 – 0.8 (D)	0.8 – 7.95 (D) 9.5 – 10.7 (D)	7.95 – 9.5 (D) 10.7 – 18.2 (D)	18.2 – 25.0 (D)	0.5 – 0.8 (D)
DS 2	0.0 – 0.7	0.7 – 0.9 (S)	0.9 – 3.82 (S)	3.82 - 5.92 (I) 5.92 - 15.37 (D)		15.37 -20.00 D)

Figure A5.11 shows a dolerite sill located between about elevations 942 m and 920 m with a thin cover of shale on top and separated from the lower sill (of Quarry I) by a layer of shale. Due to exposure to weathering from all round the hill, this dolerite is highly and moderately weathered to elevations 932 – 928 masl. This hill will be excavated to a level of 927 masl as part of the spillway approach cut and will yield mainly weathered dolerite and only a small volume of hard dolerite.

5.2.4 Quarry IV (Tunnel inlet)

A large excavation will be required for the tunnel inlet portal and the excavated material is potentially useful as embankment fill.

a) *Seismic refraction survey*

Open Ground Resources conducted one seismic line (I1I2 = 355m) along the proposed tunnel inlet (see ***Supporting Document 2*** (AECOM, et al., 2014)) and **Figure A5.2**).

The 3 500 m/s seismic velocity line usually represents slightly weathered medium to closely jointed dolerite and has been drawn in on the section containing also the results of boreholes (see **Figure A5.12**).

The seismic survey shows strong rock at relatively shallow depths between chainages 150 m and 300 m, but this has not been confirmed by the drilling. The sound rock surface is undulating, indicating variations in the depth of weathering and the degree of induration of the shale.

b) *Core drilling*

Three rotary cored boreholes were drilled at the proposed tunnel inlet section. The results of the drilling are contained in **Annexure C** and are summarised in **Table 5.13**.

Table 5.13: Results of drilling in the tunnel inlet portal area

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING O = Organic soil SC = Clayey sand or silt GC = Gravel in clay matrix S = Shale I = Indurated shale D = Dolerite					
	Transported Soil	Residual soil/ Completely Weathered	Highly Weath	Moderately Weathered	Slightly Weath	Un-weathered
	Layer thickness (m)					
BH1	0.0 – 0.9 (SC)	0.9 – 3.16 (S)	3.16 – 4.0 (S)		4.0 – 15.1 (S)	
BH2	0.0 – 5.83 (GC)			5.83 – 5.95 (S)	5.95 – 15.16 (I)	
BH3	0.0 – 1.5 (GC)	1.5 – 4.4 (S)		4.4 – 13.33 (I)	13.33 – 40.15 (I)	

No dolerite was intersected, but most of the shale is very strong (indurated) from shallow depth. Most of the material from this source will comprise hard shale.

5.2.5 Laboratory tests on quarry materials

a) Unweathered and slightly weathered dolerite and shale

Unconfined compressive strength (UCS) and Elastic Modulus tests were done on unweathered dolerite and slightly weathered indurated shale from Boreholes BH2 and BH3 at the inlet portal of the tunnel and BH 5 along the tunnel line. The results are included in **Annexure B** and summarised in **Table 5.14**.

Table 5.14: Summary of laboratory tests on unweathered dolerite and slightly weathered and unweathered shale

BH No.	DEPTH (m)	ROCK TYPE	UCS range (MPa)	TANGENT E MODULUS RANGE (GPa)	BRAZILIAN TENSILE STRENGTH RANGE (MPa)	DENSITY (kg/m^3)
2	5.95 – 12.0	Slightly weathered indurated shale	166 - 228	32.8 - 34.8	14.7 - 18.2	2660
3	27.0 – 40.0	Slightly weathered indurated shale	213 - 243	36.1 - 38.4	37 - 48.5	2680
5	387 - 397	Unweathered dolerite	212 - 349	44.7 - 63.6	31.3 - 41.6	2720

The above strengths and E-moduli are well above the required values of 50 MPa – 100 MPa and about 15 GPa for rockfill and concrete aggregate. It must, however, be remembered that the shale is prone to rapid deterioration and that these values are only applicable for the unslaked material.

b) Highly and moderately weathered dolerite

The highly and moderately weathered dolerite was not tested since it comprises hard rock blocks embedded in a soft silt matrix. The properties of the hard rock blocks are similar to the slightly weathered dolerite (see **Table 5.14** above), while the matrix material is classified as soil.

c) Slightly weathered and moderately weathered shale

Unweathered to moderately weathered shale can be subdivided into two types, namely (i) rock that is prone to slaking (carbonaceous shale) and (ii) rock that is indurated and relatively durable. The following core samples were taken for laboratory testing:

- MW: Moderately weathered shale (not prone to slaking)
- MWS: Moderately weathered shale prone to slaking
- SW: Slightly weathered shale (not prone to slaking)
- SWS: Slightly weathered shale prone to slaking.

Each of the above samples was split into two parts. The one part (Part 1) was crushed to minus 19 mm and the other part (Part B) was placed outside for a period of 3 weeks where it was exposed to the atmosphere and was sprayed with water every second day. It was then crushed to a size of minus 19 mm.

Both sets of samples (Part 1 and Part 2) were then graded and subjected to Atterberg Limit tests. ACV tests (Wet and Dry) were done on the Part 1 samples.

The results of the laboratory tests appear in **Annexure B**, and a summary is given in **Table 5.15**.

Table 5.15: Summary of laboratory tests on slightly and moderately weathered shale

SAMPLE No PROPERTY	SW Part 1 crushed	SW Part 2 exposed	SWS Part 1 crushed	SWS Part 2 exposed	MW Part 1 crushed	MW Part 2 exposed	MWS Part 1 crushed	MWS Part 2 exposed
-19 mm (%)	100	69	100	82	100	80	100	89
-4.75 mm (%)	46	15	54	31	48	18	41	34
- 2mm (%)	21	7	26	13	25	8	17	12
-0.425 mm (%)	7	3	10	5	9	4	7	4
-0.075 mm (%)	2	2	4	3	4	3	2	3
GM	2.7	2.88	2.6	2.79	2.62	2.85	2.74	2.81
LL (%)	NP[1]	NP	20	20	NP	NP	NP	NP
PI (%)	NP	NP	3	5	NP	NP	NP	NP
LS (%)	NP	NP	1.5	1.5	NP	NP	NP	NP
ACV Dry (%)	26.7	NT[2]	35.3	NT	33.5	NT	38.3	NT
ACV Wet (%)	29.5	NT	46.2	NT	42	NT	47.4	NT
Dry/Wet Ratio (%)	91	NT	76	NT	80	NT	81	NT

[1] *Non Plastic*

[2] *Not Tested*

The results of the grading before and after exposure are not as expected since the percentage fines (-0.425 mm and -2.0 mm) decreased after the samples were exposed to the atmosphere. This is ascribed to differences in the crushing of Part I and Part II samples.

From the above results it appears that slightly weathered shale (that is prone to slaking) has a clay mineral content large enough to give some plasticity (PI = 3 – 5).

The effect of short-term (3 weeks) exposure on the material properties does not seem to be dramatic in terms of grading (GM = 2.6 – 2.88).

The reduction in strength (ACV) after wetting varies between 24% for the slightly weathered shale that is prone to slaking to about 9% for the slightly weathered shale that is not prone to slaking. The latter type of shale meets the ACV specification for concrete aggregate. For the moderately weathered shale, the reduction in ACV after wetting is about 20 %.

The overall result of the durability testing is reassuring in that it appears that the indurated shale (visually not prone to slaking) is strong, most of the decrease in strength takes place within a short period (days rather than weeks), and that all slightly weathered and unweathered shale can be considered suitable as hard rockfill, provided that it is protected by an outer shell of durable rock.

d) Highly and moderately weathered shale

These materials were not tested during the current investigation, but similar materials were tested during the investigation for the Zalu dam site near Lusikisiki (Refer to *Materials and Geotechnical Investigation Report, PWMA 12/T60/00/4411 for the Feasibility Study for Augmentation of the Lusikisiki Regional Water Supply Scheme*).

The samples were subjected to primary crushing to the minus 25 mm size and subjected to Grading, Atterberg, Aggregate Crushing Value (ACV) Standard Proctor and Consolidated Undrained Triaxial testing.

Since it is not known to what extent the weathered shale will break up during quarrying, transport, compaction and consolidation in a dam embankment, it was decided, for the sake of conservatism, to conduct the triaxial tests on the minus D_{10} fraction. However, after the primary crushing (to minus 25 mm) the minus D_{10} fraction of the highly weathered and moderately weathered shale produced too little material (<10% of the sample) and it was then decided to crush the entire sample to minus 0,425 mm for the triaxial testing. The samples were compacted to 95% of Standard Proctor density and the moisture content adjusted to about 20% before testing.

The results are shown in **Table 5.16**.

Table 5.16: Summary of laboratory tests on crushed rock cores

Sample Origin	Highly weathered shale	Moderately weathered shale
Laboratory Number	2/4971	2/4972
Grading description (after crushing to minus 25 mm)	Silty (11%) sandy (34%) gravel (51%)	Sandy (22%) gravel (78%)
LL (%)	27	Non plastic
PI (%)	7	Non plastic
LS (%)	2,5	1.5
PE	Low	Low
ACV (%)	39,9	29,2
Standard Proctor MDD (kg/m^3)	1960	2049
Standard Proctor OMC (%)	13,4	10,3
Cohesion (effective) (kPa)	0	13
Friction angle (effective) (degrees)	35	35

5.2.6 Available volumes from quarry areas

Based on the information from the drilling, the various types of material available from the quarries can be described as follows:

Slightly weathered and unweathered dolerite is very strong rock with staining along the major joint planes. It is a durable rock and is the only suitable source for concrete aggregate, rip-rap and filters (if crushed).

Highly and moderately weathered dolerite comprises strong boulders (corestones) in a matrix of clayey silt. This material can be considered for use as “dirty rockfill” in certain zones of a rockfill dam. Highly weathered dolerite typically contains between 10 % and 50 % rock, while moderately weathered dolerite comprises of more than 50 % corestones. These corestones can vary in size between 100 mm and 1 200 mm. Blasting is generally not very efficient and fragmentation is difficult to control. It might be necessary to remove the blocks that are too large for placing in a particular zone of the dam. These blocks might be suitable for use as rip-rap.

Unweathered to moderately weathered shales are generally medium strong to strong rocks in situ, but are prone to rapid slaking upon exposure to the atmosphere. With increased degree of induration, the potential for slaking decreases. This shale material, can be considered as rockfill, but must be covered by durable (dolerite) rock outer zones.

Completely and highly weathered shale can be considered for use as semi-pervious material or as transition between a clay core and soft rockfill zones.

Clayey sand transported surface material is suitable as impervious core (refer to Earth Fill materials **Section 5.1**) while the **sand, clay and boulders** might be considered as “dirty rockfill”.

The estimated volumes of the above materials are given in **Table 5.17**.

Table 5.17: Estimated volumes of materials from quarries

MATERIAL TYPE	ORGANIC SOIL (Spoil)	CLAYEY SAND (Impervious)	WEATHERED SHALE	DIRTY ROCKFILL (Dolerite boulders in clayey silt)	SHALE ROCKFILL	HARD ROCKFILL OR AGGREGATE (Dolerite)
QUARRY	VOLUME (m^3)					
I (Left Flank)	50 000	20 000	600 000	140 000	600 000	2 600 000
II(Plunge pool	40 000	200 000	170 000	70 000 +780 000[1]	44 000	720 000
III(Spillway approach)	20 000	25 000	20 000	815 000	10 000	123 000
IV(Tunnel inlet)	5 000	7 000	110 000	0	13 500	0
TOTALS	115 000	252 000	900 000	1 805 000	667 500	3 443 000

Note[1]: This volume represents the material around the downstream sides of the plunge pool.

From the above it appears that sufficient aggregate-quality dolerite is available for construction of a RCC dam. There will also be sufficient aggregate for the tunnel lining and intake structure.

The various quarries will not yield sufficient hard rock for a conventional hard rockfill dam. However, if the floor level of Quarry 1 is lowered by 10 m to elevation 865 m (5 m above river level) an additional 1,4 million m³ of hard shale can be obtained. If a zoned rockfill embankment comprising about 20:80 ratio of soft and hard rock is considered, the required volumes can be obtained.

The shale rockfill is prone to rapid deterioration (slaking) and will have to be used in the inner zones of the embankment so that the durable dolerite can be used as outside protective shells.

5.2.7 Design parameters for rock materials

Based on the laboratory tests and material properties derived from other studies and literature sources, the design parameters for the compacted embankment materials as shown in **Table 5.18** can be adopted:

Table 5.18: Material parameters for design (quarry areas only)

Material type	Properties of compacted material n/a = not applicable n/t = not tested est = estimates based on other projects						
	Liquid Limit (%)	Plasticity Index (%)	Density (kg/m^3)	OMC (%)	Permeability (m/s)	Cohesion (kPa)	Phi (°)
Semi-pervious fill: Highly weathered shale	27	7	1960	13.4	2,7 x 10^{-3}*	n/t	n/t
Soft rockfill: Moderately weathered shale	n/p	n/p	2049	10.3	1 x 10^{-1}*	0	35
Hard rockfill: Slightly weathered and unweathered shale, non-slaking	n/p	n/p	2100 (est)	n/a	n/t	0 (est)	38 (est)
Hard rockfill: Slightly weathered and unweathered shale, slaking	20	5	2100 (est)	n/t	n/t	0 (est)	36 (est)
Hard rockfill : Slightly weathered and unweathered dolerite	n/p	n/p	2200 (est)	n/a	n/t	0 (est)	40 (est)

* *Permeability based on Hazen's equation on D_{10} of samples crushed to minus 25 mm.*

6 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations were conducted at the following locations:

- Along the proposed dam centre line;
- Along the proposed spillway return channel on the left flank;
- Along the saddle embankment centre line; and
- Along the diversion tunnel alignment.

The spillway approach channel and spillway plunge pool are potential sources of construction materials and the results of these investigations are contained in **Section 5**.

6.1 DAM CENTRE LINE

The position of the centre line was selected with a view to a dam with FSL of 930 masl and is located slightly upstream (150 m on left flank and 0 m on right flank) from the previously drilled centre line as identified by BH 1001 – BH 1004 (see **Figure A6.1**).

6.1.1 Seismic refraction surveys

A Seismic Refraction survey by Open Ground Resources was conducted along the following lines (see ***Supporting Document 2*** (AECOM, et al., 2014)).

- Left flank centre line (Line L1L2 = 415 m);

- Right flank centre line (Line R1R2 = 785 m);

The positions of the seismic lines are shown on **Figure A6.1**.

Due to the steep slopes of the lower flanks and the presence of flowing water in the river (noise factor), seismic surveys could not be conducted in the central (river) section.

The 3 500 m/s seismic velocity line usually represents slightly weathered, medium to closely jointed dolerite, suitable as foundation for a concrete dam. Lines depicting this velocity have been drawn on sections containing the results of boreholes along the left flank (**Figure A6.2**), the central section (**Figure A6.3**) and the right flank (**Figure A6.4**).

Along Seismic Line L1L2 on the left flank, the 3 500 m/s (sound rock) velocity line varies in depth between about 28 m at section chainage 50 m to about 6 m at section chainage 430 m (**Figure A6.2**). The depth of sound dolerite appears to be over-estimated by about 5 m at Boreholes DLS2 and DL3. From section chainage 50 m to section chainage 270 m, the seismic line was used to estimate the top of the dolerite sill.

In the central section the seismic survey extends to chainage 450 m on the left side of the river (L1L2) and starts from chainage 610 m on the right side (see **Figure A6.3**). Between chainages 430 m and 450 m on the left flank, the seismic survey appears to have over-estimated the depth to sound dolerite by about 5 m. Between chainages 610 m and 730 m the seismic survey appears to have over-estimated the depth to sound dolerite by about 5 m to 7 m. Seismic low anomalies at chainages 680 m and 750 m might be indicative of shear or fault zones.

Along seismic line R2R2 on the right flank, the 3 500 m/s line varies between about 24 m depth at section chainage 790 m and 40 m at section chainage 1190 m (see **Figure A6.4**). This velocity line seems to correspond roughly with the top of the main dolerite sill, except that it tends to over-estimate its depth by 2 m to 8 m. The results of the seismic survey does not seem to reflect the presence of the thick layer of transported soil between section chainages 0 m and 220 m and also does not seem to differentiate between the transported soils and the underlying shales.

6.1.2 Core drilling and water testing

A total of 14 cored boreholes with Lugeon water testing were drilled to supplement the results of previous drilling and to investigate the anomalies revealed by the seismic refraction survey. Borehole 1004 from the pre-feasibility investigations falls along the present centre-line. The positions of the boreholes drilled during the previous and present investigations are shown on **Figure A6.1**.

Boreholes DLS 1 – DLS 3 and Boreholes DRS 1 – DRS 3 were drilled along the left and right flanks respectively to investigate seismic anomalies, while three boreholes (DL 1, DL 3 and DL4) and four holes (DR 1 – DR 4) were drilled on the left and right flanks respectively to supplement the available information. A summary of the borehole results is given in **Table 6.1**.

Table 6.1: Summary of borehole results along the dam centre line

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING Organic soil = O, Sandy clay = SC, Gravel in clay = GC, Gravel in sand = GS Shale = S; Indurated shale = I, Dolerite = D					
	TRANSPORTED SOIL	RESIDUAL SOIL/ COMPLETELY WEATHERED	HIGHLY	MODERATELY	SLIGHTLY	UN-WEATHERED
	LAYER THICKNESS (m)					
DLS 3	0.0 – 0.69 (O) 0.69 – 5.82 (SC)		5.82 – 10.8(S)	10.5 – 16.8 (S)	16.8 –25.6 (S)	
DL 1	0.0 -0.5 (O) 0.5 – 9.17 (GC)	9.17 – 10.3 (S) 26.96–28.46(S)	10.3 – 11,0(S)	11.0 – 22.6 (S)	22.6–26.96(S) 28.46–30.0(S)	
DLS 2	0 – 0.65 (O) 0.65 – 7.86 (SC) 7.86 – 8.42 (GC)	14.5 – 15.0 (S)	15.0 – 17.2(S)	8.42 – 14.5 (S) 17.2 – 30 (S)		
DLS 1	0 0 – 0.3 (O) 0.3 – 0.5 (SC)	0.5 – 1.5 (S)	1.5 – 6.0 (S)	6.0 – 13.93 (S)		13.93 – 25.2 (S)
DL 3	0.0 – 0.5 (O)	0.5 – 1.13 (D)	2.21 – 3.78(D)		1.13 –2.21 (D) 3.78 – 4.8 (D)	4.8 – 28.33 (D) 28.33– 30.51 (S)
DL 4	0.0 – 0.5 (O) 0.5 – 0.8 (GC)		0.8 – 1.12 (D)		1.12 – 5.0 (D)	5.0 – 25.8 (D) 25.8 – 30.0 (I)
DR 2	0.0 – 1.5 (SC) 1.5 – 3.6 (GS)			3.6 – 8.2 (S)	8.2 – 11.0 (S)	11.0 – 20.0 (S)
DR 1	0.0 – 0.6 (O) 0.6 – 2.36 (SC) 2.36 – 8.0 (GS)			8.0 – 11.0 (S)	11.0 – 15.5 (I)	15.5 – 31.8 (I) 31.8 – 39.27 (D) 39.27 – 40.0 (I)
DRS 1	0.0 – 1.03 (O)	1.03 – 3.0 (S)	3.0 – 5.84 (S)	5.84 – 11.0 (S)		11.0 – 25.1 (D)
DTS 1	0.0 – 0.53 (O)	0.53 – 2.33 (S) 4.59 – 5.1 (S)	2.33 – 4.59(S)	5.1 – 7.76 (S)	7.76 –11.0 (D)	11.0 – 35.06 (D)
DR 3	0.0 – 1.08 (O) 1.08 – 4.4 (SC) 4.4 – 11.1 (GC)		11.1 – 14.8(S)	14.8 – 25.1 (S)	25.1 –28.0 (D)	28.0 – 40.1 (D)
1004	0.0 – 0.8 (O) 0.8 – 9.2 (SC) 9.2 – 12.4 (GC)		12.4 – 13.8(S)			
DRS 2	0.0 – 0.6 (O) 0.6 – 11.36 (SC) 11.36 – 14.4 (GC)			21.5 – 22.3 (S) 22.3 – 23.2 (D)	14.4 – 21.5(S) 23.2–25.07(D)	
DR 4	0.0 – 0.36 (O) 0.36 – 5.27 (SC)		5.27 – 8.0 (S)	8.0 – 24.8 (S) 24.8 –25.0 (D)		
DRS 3	0.0 – 0.18 (O)	0.18 – 3.2 (S)	3.2 – 5.67 (S)	5.67 –11.35 (S) 11.35–17.9 (D)	17.9– 25.1 (S)	

Water pressure tests were conducted by using single packers in sections of 3 m length and maximum water pressures of about 15 x D for packers set in soil and 22 x D for packers set in rock (where D is the depth of the packer below ground surface). Each test section was subjected to five (ascending and descending) water pressures (each for a period of 10 minutes), and the corresponding water losses were measured. The Lugeon value for each increment of water pressure was calculated as follows:

Lugeon		=	*(1000 x V)/(T x L x P)*
where:	*V*	=	*Volume of water pumped in litres*
	T	=	*Time of water pumped in minutes*
	L	=	*Length of test section in metres*
	P	=	*Pumping pressure in kPa*

For each test section, five Lugeon values (one for each pressure increment) were calculated, and the most appropriate Lugeon value was selected from the flow pattern (laminar, turbulent, dilation, blockage or wash-out) according to the guidelines produced by Houlsbey (1976).

The results of the water pressure tests are presented in **Table 6.2**.

Table 6.2: Summary of water pressure test results

BH No.	Depth untested	LUGEON VALUES (values <0.1 Lugeon not given))		
		0.1 -1	1.1 – 4 (actual Lugeons)	>4 (actual Lugeons)
		Depth in m	Depth in m	Depth in m
DLS3	0 – 3			
DL1	0 – 3			
DLS2	0 – 6	12 – 15	6 -12	15 – 18 (4.9) 18 – 21 (total loss) 21 – 24 (23) 24 – 30 (total loss)
DLS1	0 - 3			
DL3	0 - 3	24 - 27	27 – 30 (1.6)	
DL4	0 – 2 29 - 30		23 – 26 (2) 26 – 29 (1.1)	
DR2	0 - 2	17 - 20	14 – 17 (1.3)	
DR1	0 – 2 35 - 40			2 – 5 (total loss) 8 – 11 (7) 11 – 14 (14) 23 – 26 (6.3)
DRS1	0 – 2 23 – 25.1			
DTS1	0 – 2 32–35.06	2 – 5 8 – 11 20 – 26 29 - 32		
DR3	0 – 2 38 – 40.1	23 - 29		8 – 11 (4.2)
1004				
DRS2	0 – 2 23–25.07	13 - 23	11 – 14 (1.7	8 – 11 (5)
DR4	0 – 2 23 - 25		17 – 20 (3) 20 – 23 (2)	
DRS3	0 – 2 23 – 25.1	5 - 14	14 – 20 (2.8) 20 – 23 (1.8)	

All boreholes along the dam centre line and the diversion tunnel lines were equipped with standpipe piezometers. The water level readings are given in **Table 6.3**.

Table 6.3: Water rest levels in boreholes

BH No	Date completed	Date measured	Water level (m)	Water level Elevation (masl)
DLS3	2013-02-16	2013-02-16 2013-04-15	3.6 dry	918.6 -
DL1	2013-02-21	2013-02-21 2013-04-15	10.0 dry	908.2 -
DLS2	2013-02-11	2013-02-11 2013-04-15	dry dry	- -
DLS1	2013-02-09	2013-02-09 2013-04-15	10.0 13.1	894.3 891.2
DL3	2013-03-04	2013-04-15	16.2	873.3
DL4	2013-03-13	2013-03-13 2013-04-15	1.04 23.4	878.2 855.9
DR2	2013-03-06	2013-04-15	3.2	854.3
DR1	2013-03-14	2013-04-15	1.7	855.6
DRS1	2013-02-15	2013-02-15 2013-04-15	3.6 dry	882.0 -
DTS1	2013-02-20	2013-04-15	dry	-
DR3	2013-02-27	2013-02-27 2013-04-15	15.7 dry	884.5 -
DR4	2013-03-08	2013-04-15	19.7	889.7
DRS3	2013-03-14	2013-03-14 2013-04-15	21 dry	904.13 -
DT1	2013-02-28	2013-02-28	16.1 7.5	875.2 883.8
DT2	2013-03-02	2013-03-02 2013-04-15	9.2 5.7	882.1 885.6
DT5	2013-03-19	2013-03-19 2013-04-15	12 18.5	864.7 858.2
DTS2	2013-02-18	2013-02-18 2013-04-15	3.1 12.4	892.4 883.1

6.1.3 Discussion of drilling results

a) Left flank (above elevation 890 m)

The results of the seismic survey, the drilling and the interpretation of the geology between section chainages 0 m and 440 m are shown on **Figure A6.2**.

The upper part of the left flank (section chainages 0 – 350 m) is occupied by 20 – 30 m of horisontally bedded shale that overlies a 30 m thick dolerite sill. From section chainage 350 m to 440 m the shale layer becomes thinner and at chainage 440 m the underlying dolerite crops out.

Above elevation 905 m (chainage 320 m), the shale bedrock is covered by a 0.3 m – 0.5 m thick layer of organic soil, followed by between 5 m and 9 m of colluvium comprising of sandy clay or boulders and cobbles in a matrix of sandy clay.

The shale bedrock is highly to completely weathered to depths of between 6 m and 11 m, and moderately weathered to depths of between 14 m and 22 m.

The upper 2 – 3 m of the boreholes have generally not been water pressure tested, but below this depth, the colluvial deposits appear to be impermeable.

The permeability of the shale is typically very low (0 – 0,5 Lugeons), but Borehole DLS2 is an exception with moderate (5 Lugeons) to very high (total loss) below a depth of 18 m. This high loss is not associated with a dolerite contact, but occurred in moderately weathered, closely jointed (RQD <25) shale that does not appear to differ from shale in which no loss was experience. This hole is the first one that was water tested, and it is possible that the packer did not seal because the inexperienced drilling staff did not know how to operate the system. From the appearance of the core, there is no reason to expect high permeability and if that should be the case, the rock appears to be groutable.

The dolerite below the shale at elevation 890 masl is generally unweathered, except where the shale cover is absent and the dolerite is weathered to a depth of about 5 m. The dolerite is generally impermeable, even where water tests have been conducted across the contacts with the overlying shale.

Ground water levels as measured in boreholes present a different picture of the overall permeability of the flank. Drilling water remains at a high level in the holes for a few days, but after a few weeks the water levels return to the natural deep water level that indicates a moderate permeability of the rock mass (presumably the shale).

b) Central river section

The results of the drilling and seismic survey and interpretation of the geology between section chainages 440 m and 790 m is shown on **Figure A6.3**.

The dolerite sill from the left flank extends across the river and into the right flank, but the bottom contact of the sill is above river level so that the underlying indurated shale occupies the entire river channel. Another dolerite sill of 6m thickness was encountered about 28 m below river level in Borehole DR 1.

The main dolerite sill appears to thin from about 26 m on the left flank to about 18 m on the lower right flank. Higher up along the right flank, from chainage 590 m to chainage 690 m the dolerite is covered by shale.

Due to the steepness of the flanks and the dense vegetation, boreholes could not be drilled closer than about 50 m to 70 m from the sides of the river channel on the left and right flanks respectively. In these holes the dolerite is slightly weathered from surface (left flank) and below the shale (right flank). No water losses were measured during Lugeon testing. Along the steep flanks there are continuous dolerite outcrops, but the rock mass contains moderately spaced horizontal and vertical joints that are wide open at surface due to stress relief and erosion.

Shale crops out in the left side of the river channel and is covered by alluvium in the right side. The shale had been indurated by the dolerite sill (that had since been removed by erosion). Lugeon values of 7 Lugeons and 14 Lugeons have been recorded between 8 m and 13 m (directly below the alluvium) and at about 20 m depth in Borehole DR1.

The shale on the right bank is moderately weathered to about 10 m and showed no water loss.

Ground water levels are all at or below river bed level indicating high permeability of the steep sided flanks.

c) Right flank (above elevation 890 m)

The results of the drilling and interpretation of the geology between section chainages 790 m and 1190 m is shown on **Figure A6.4**.

The right flank is underlain by shale into which at least two dolerite sills had intruded. The upper sill is about 7 m thick and occurs between elevations 907 m and 914 while the lower sill is estimated to be between 20 m and 30 m thick and is separated from the upper sill by about 18 m to 20 m of shale. The dolerite sill occurs about 5 m higher in Borehole DTS1 than in Boreholes DRS1 and DR2 located on either side of it. There are also seismic low anomalies on either side of Borehole DTS1 indicating that the faults intersecting Quarry Area I on the left flank might extend across the right flank. This area will have to be further investigated by means of inclined boreholes.

The entire right flank is covered by between 0,2 m to 1,2 m of organic soil, while the part between section chainages 590 m and 1170 m is covered by layers of transported material comprising 3 m to 10 m of colluvial sandy clay occurring on top of a 3 m to 6 m thick layer of weathered alluvial boulders in a matrix of sandy silt. This section represents an ancient river channel in which the alluvium had been deposited. At a later stage, the river course migrated towards the south, and the layer of alluvium was subsequently covered by colluvium from higher lying ground to the north. Over a period of thousands of years since its deposition, the alluvium weathered to the extent that most of the boulders were reduced in size or turned into soil.

Shale occurs below a shallow cover of colluvium between section chainages 1130 m and 1 190 m where it is highly weathered to 4 m and moderately weathered to 10 m. Below the alluvium, the shale is typically highly weathered to thicknesses of 2 m to 3 m and moderately weathered to thicknesses of between 5 m and 10 m.

The dolerite below the shale is slightly weathered to unweathered and gave no water loss.

The 5 m to 14 m thick layer of transported material has been water pressure tested in some sections while in others the packer did not seal. Lugeon values varied between 0 and 5.

Lugeon values in the shale are typically below 1, but there are exceptions in DR4 (3 Lugeons between 18 m and 21 m) and DR3 (2.7 Lugeons between 18 m and 21 m at shale /dolerite contact).

Ground water levels as measured in boreholes present a different picture of the overall permeability of the flank. Drilling water remains at a high level in the holes for a few days, but after a few weeks the water levels return to the natural deep water level that indicates a moderate permeability of the rock mass (presumably the shale).

6.1.4 Founding conditions for alternative dam types

a) Earth embankment shells

The shells of an earth embankment are typically founded on material with low organic content, low compressibility and with shear strength similar to the dam wall material. This means that a 0,2 m – 0,8 m thick layer of organic topsoil has to be removed along the centre line and that founding will take place on stiff transported sandy clay on the upper left flank and most of the right flank, weak completely weathered shale or weak residual dolerite along the lower left flank and right flanks and along the upper right flank and loose silty sand alluvium in parts of the river channel.

b) Rockfill embankment shells

The shells of a rockfill embankment are typically founded on material with low organic content, low compressibility and with shear strength similar to the dam wall material. This means that a 6 m to 10 m layer of colluvium and residual soil/completely weathered shale has to be removed from the upper left and right flanks, 1.5 – 5 m of residual soil/completely weathered shale or dolerite and medium dense river alluvium from the central section and 11,2 m to 14,4 m of transported sandy clay with boulders along a large part of the right flank (see **Figures A6.5**, **A6.7** and **A6.9**). This excavation will yield a large volume of material, most of which might be suitable as impervious and semi-pervious earthfill. Laboratory testing of this material will have to be conducted.

c) Plinth for Concrete Faced Rockfill

In the case of a concrete faced rockfill (CFR) dam, founding of the plinth will require material with either low permeability or good groutability. This depth corresponds approximately with the depth of excavation for the core trench (see **Figures A6.6**, **A6.8** and **A6.10**).

d) Core trench

The clay core of an earthfill or rockfill dam is normally founded on material that is either sufficiently impervious or can be rendered impervious by means of grouting. This depth of the core trench will in most sections be the same as for the plinth of a rockfill dam, except in the river section where it has to be taken through the layer of permeable alluvium (see **Figures A6.6**, **A6.8** and **A6.10**).

e) Concrete gravity

A concrete gravity dam is normally founded on good quality rock with a minimum Rock Mass rating of 50, giving it an E-value of at least 10 GPa and with shear strength parameters along the most critically orientated discontinuities of C = 250 kPa and ϕ= 34 degrees.

Due to the presence of closely spaced continuous weak bedding planes in the highly and moderately weathered shale and to some extent also in the slightly weathered and unweathered shale, the above requirement for shear strength are only marginally met in slightly weathered and unweathered shale. This means that excavation depths for a concrete dam will extend to between 14 m and over 30 m on the upper left flank and 18 m to 25 m on the right flank (see **Figure A6.10**).

For concrete structures where shear strength is not critical (e.g. spillway chute), founding can take place on moderately weathered shale at considerably shallower depths (see **Figure A6.6**).

The steep flanks of the central section is underlain by the dolerite sill and excavation depths of 3 m to 5m on the left flank and 5 m to 11 m on the right flank are anticipated. In the left side of the river channel where there are outcrops of indurated shale, depths of 2 m to 3 m are expected, while in the right side, excavation of up to 10 m deep is required to remove the alluvial deposits and moderately weathered shale below (see **Figure A6.8**)

The recommended excavation depths at borehole positions are listed in **Table 6.4**.

Table 6.4: Recommended excavation depths at the various borehole positions

BH No.	ELEVATION (masl)	EXCAVATION DEPTHS IN METRES (VERTICAL)				
		Earth embankment shells	Rockfill embankment shells	Core and plinth	Concrete dam	Concrete chute
DLS 3	922.17	1.0	6.0	3.0	17.0	10.5
DL 1	916.23	0.5	10.3	10.6	23.0	11.0
DLS 2	914.34	0.7	8.4	8.4	30 +	8.5
DLS 1	904.25	0.3	3.0	4.0	14.0	6.0
DL 3	889.54	0.5	2.2	3.5	4.0	4.0
DL 4	879.25	0.5	1.5	2.0	2.0	2.0
DR 2	857.46	3.6	3.6	3.6	8.5	8.5
DR 1	857.32	2.5	5.0	10.0	10.0	10.0
DRS 1	884.58	1.1	4.5	1.5	11.0	11.0
DTS 1	888.42	0.6	5.2	3.0	8.0	8.0
DR 3	900.15	1.1	11.2	11.0	25.0	15.0
1004	901.20	1.0	12.5	12.5	13+	13+
DRS 2	903.81	1.0	14.4	15.0	15.0	14.5
DR 4	909.44	0.5	7.5	7.5	25.0	8.0
DRS 3	925.13	0.9	3.2	3.5	18.0	6.0

f) Grouting

In 5 of the 14 boreholes Lugeon values of more than 4 have been recorded and in 7 holes values exceeding 1.1 Lugeon. Only two holes (DLS2 on the upper left flank and DR1 in the river channel) experienced significant losses and in both cases the losses occurred in shale. The core shows no evidence of faulting and shearing and is not different from shale in holes where no losses were experience. It is therefore possible that the packers did not seal properly, especially in the case of DLS2.

Ground water levels are generally low along the flanks, indicating a moderate permeability of the rock mass.

For a dam of this height, it will be necessary to make provision for a grout curtain to a depth of about 66% of the water head along the centre line. Although grout penetration might be small except in local zones, the drilling, water test and grout records from a grouting operation is very important and can be considered the final stage of a geotechnical investigation when sub-surface information is obtained at close intervals below the footprint of the dam.

g) Spillway plunge pool

Rock in the area downstream of the central overflow section comprises indurated shale in the river channel and dolerite along the flanks. The shale is very strong and widely jointed, but it might be prone to slaking upon alternative cycles of wetting and drying. The dolerite is also very strong, but there are open joints near ground surface that will give rise to erosion by flowing water. It will therefore be necessary to provide a concrete-lined plunge pool downstream of the overspill section.

6.2 LEFT FLANK SPILLWAY AND RETURN CHANNEL

The layout of the proposed spillway control structure and return channel is shown in **Figure A6.1**.

6.2.1 Seismic survey

A Seismic refraction survey by Open Ground Resources along Line S1S2 (595 m) from the 920 m contour on the left flank to the river (see report in **Annexure C**) was discussed in **Section 5.2.3** of this report.

6.2.2 Core drilling

Five cored boreholes represent conditions in the area of the spillway return channel (chute). Their positions are shown on **Figure A6.1** and the results are summarised in **Table 6.5**.

Table 6.5: Summary of borehole results along spillway return channel

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING Organic soil = O, Sandy clay = SC, Gravel in clay = GC, Gravel in sand = GS Shale = S; Indurated shale = I, Dolerite = D					
	Trans-Ported Soil	Residual soil/ Completely Weathered	Highly Weath.	Moderately Weathered	Slightly Weath.	Un-Weathered
	Layer thickness (m)					
DS1	0.0 – 0.5 (O)	0.5 – 0.8 (D)	0.8 – 7.95 (D) 9.5 – 10.7 (D)	7.95 – 9.5 (D) 10.7 – 18.2 (D)	18.2– 25.6 (D)	
DLS3	0.0 - 0.7 (O) 0.7 – 4.8 (SC)	4.8 – 5.8 (S)	5.8 – 10.8 (S)	10.8 – 16.8 (S)	16.8 – 26.6(S)	
DSS1	0.0 – 0.5 (O)	0.5 – 4.36 (S)	4.36–11.65(S) 17.4 – 17.7(S)	11.65 – 17.4(S)	17.4–20.17(S)	
DSS2	0.0 – 1.0 (O)	1.0 – 2.83 (D)			2.83 – 5.0 (D)	5.0 – 20.28(D)
DS7	0.0 – 0.5 (O)	0.5 – 10.0 (S) 10.0–11.65 (D)		11.65–2.75(D) 22.0 – 30.0 (S)	12.75–22.0(D)	

The section line through Boreholes DS1 – DLS3 – DSS1 – DS7 (**Figure A6.12**) runs more or less along the right hand side of the proposed 100 m wide return channel. The length of the channel from the control structure to the proposed plunge pool is approximately 150 m.

The control structure can be founded on slightly weathered shale at depth ranging between 15 m and 20 m below ground surface and the concrete lined channel can be founded on moderately weathered shale at depths of between 10 and 12 m.

6.3 SADDLE EMBANKMENT

The lowest ground level in the saddle area is at about elevation 611 masl. That means that an embankment of about 20 m height must be constructed. The position of the saddle is shown on **Figure A6.1**.

6.3.1 Seismic survey

A Seismic refraction survey by Open Ground Resources was conducted along Line E1E2 (595 m) (see report in ***Supporting Document 2*** (AECOM, et al., 2014)).

The 3 500 m/s seismic velocity line on **Figure A6.13** indicates the presence of shallow dolerite at Borehole SES 2, but appears to overestimate its depth by between 2 m and 5 m. The upper contact of the dolerite sill has been estimated accordingly.

The 2 000 m/s seismic line varies between 3 m and 7 m below ground surface and corresponds more or less with the depth of moderately weathered shale as found in the boreholes.

6.3.2 Core drilling

Four cored boreholes represent conditions in the area of the saddle embankment. Their positions are shown on **Figure A6.1** and the results are summarised in **Table 6.6**.

Table 6.6: Summary of borehole results along saddle embankment

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING Organic soil = O, Sandy clay = SC, Gravel in clay = GC, Gravel in sand = GS Shale = S; Indurated shale = I, Dolerite = D					
	Trans-Ported Soil	Residual soil/ Completely Weathered	Highly Weath.	Moderately Weathered	Slightly Weath.	Un-Weathered
	Layer thickness (m)					
SSS1		0.0 – 0.2	0.2 – 1.6 (S)	1.6 – 4.2 (S) 4.2 – 9.0 (I)	9.0 – 15.07 (I)	
SES1	0.0 – 0.5 (O) 0.5 – 1.36 (SC)			1.36 – 4.8(S)	4.8 - 15.0(S)	
SES2	0.0 – 0.1 (O)		0.1 – 1.0 (S) 4.55 – 4.88(S)	1.0 – 4.55 (S) 4.88 – 5.23 (S) 5.23 – 6.5 (D)	6.5 – 9.0 (D)	9.0 – 15.0 (D)
SES3	0.0 – 0.4 (O)		0.4 – 1.5 (S)	1.5 – 5.5 (S)	5.5 – 15.0 (S)	

Water pressure tests were conducted by using single packers in sections of 3 m length and maximum water pressures of about 15 x D for packers set in soil and 22 x D for packers set in rock (where D is the depth of the packer below ground surface). Each test section was subjected to five (ascending and descending) water pressures (each for a period of 10 minutes), and the corresponding water losses were measured. The Lugeon value for each increment of water pressure was calculated as follows:

Lugeon = *(1000 x V)/(T x L x P)*

where: *V* = *Volume of water pumped in litres*
T = *Time of water pumped in minutes*
L = *Length of test section in metres*
P = *Pumping pressure in kPa*

For each test section, five Lugeon values (one for each pressure increment) were calculated, and the most appropriate Lugeon value was selected from the flow pattern (laminar, turbulent, dilation, blockage or wash-out) according to the guidelines produced by Houlsby (Houlsby, 1976).

The results of the water pressure tests are presented in **Table 6.7**.

Table 6.7: Summary of water pressure test results

BH No.	Depths untested (m)	Ground water levels (m)	LUGEON VALUES (only values above 0)		
			0.1 -1	1.1 – 4 (actual Lugeons)	>4 (actual Lugeons)
			Depth in m	Depth in m	Depth in m
SES1	0 – 2 14 - 15	3.8		11 – 14 (1.5)	2 – 5 (5.6)
SES2	0 – 3	2.0	3 - 6		
SES3	0 – 2 14 - 15	13.5	8 - 14		
SSS1	0.0 – 15.07	4.6			

Both the water pressure tests and the high water tables indicate that the shale is generally impervious.

6.3.3 Discussion of results

The entire centre line is underlain by a thin (0.1 – 0.5 m) layer of organic soil, followed by highly weathered shale (to a depth of 1.5 m) followed by moderately weathered and later slightly weathered shale. A dolerite sill occurs at shallow depth at Borehole SES 2 but dips down towards the flanks.

The permeability of the moderately weathered shale is everywhere less than 2 Lugeon, except in Borehole SES 1 where a loss of 5.6 Lugeons was encountered to a depth of 5 m. The contact with the underlying dolerite appears to be tight.

6.3.4 Founding conditions for embankment dam

a) Earth embankment shells

The shells of an earth embankment are typically founded on material with low organic content, low compressibility and with shear strength similar to the dam wall material. This means that a 0.1 m – 0.5 m thick layer of organic topsoil has to be removed along the centre line and that founding will take place on highly weathered shale.

b) Clay core

The clay core of an earthfill or rockfill dam is normally founded on material that is either sufficiently impervious or can be rendered impervious by means of grouting. It must also have a density that is equal to or greater than the compacted density of the core.

This depth corresponds with the moderately weathered shale that occurs at depths of between 2 m and 4 m (see **Figure A6.14**).

The recommended excavation depths at borehole positions are listed in **Table 6.8**.

Table 6.8: Recommended excavation depths at the various borehole positions

BH No.	ELEVATION (masl)	EXCAVATION DEPTHS IN METRES (VERTICAL)				
		Earth embankment shells	Rockfill embankment shells	Core and plinth	Concrete dam	Concrete chute
SSS1	930.2	0.5	1.6	2.0	n/a	n/a
SES1	917.4	1.5	1.5	3.2	n/a	n/a
SES2	911.9	0.5	2.0	3.0	n/a	n/a
SES3	915.2	0.5	1.5	2.5	n/a	n/a

c) Grouting

In only one borehole a Lugeon value of more that 2 was encountered (5.6 Lugeons between 2 m and 5 m in SES 1).

If Quarry I is developed just upstream of the saddle embankment, the flow path underneath the embankment will be considerably shortened and it is recommended that provision be made for a grout curtain to a level at least 20 m below the quarry floor (i.e. 850 masl). Although grout penetration might be very small, the drilling, water test and grout records from a grouting operation is very important and can be considered the final stage of a geotechnical investigation when sub-surface information is obtained at close intervals below the footprint of the dam.

6.4 DIVERSION TUNNELS

Five x 6 m diameter tunnels through the right flank have been proposed to divert the flow of the river during construction of an embankment dam. These tunnels will be located between the two lines running from Boreholes DT2 to DT1 and from Boreholes DTS2 to 1003 as shown on **Figure A6.1**. The tunnel inlet inverts will be at 860 masl and the outlet inverts at 856 masl.

6.4.1 Seismic survey

Seismic Refraction surveys by Open Ground Resources were conducted along Lines (T1 T2 = 335 m) and T3T4 = 235 m) on the left flank (see report in ***Supporting document 2*** (AECOM, et al., 2014)).

The seismic surveys could not be conducted over the full lengths of the tunnel alignments due to the steep slopes towards the river.

The 3 500 m/s seismic velocity lines have been drawn on sections along the tunnel alignments (**Figure A6.15** and **Figure A6.16**). This velocity generally represents good rock conditions for tunnelling.

The upper line T1T2 shows the 3 500 m/s line to be just above the 870 m elevation. There is a small velocity low anomaly near Borehole DR3. Based on borehole results, the seismic line appears to overestimate the actual depth of sound rock by between 6 m and 10 m.

The lower line T3T4 shows the 3 500 m/s line to be mostly above the 870 m elevation, except between section chainages 220 m and 230 m where there is a seismic low anomaly, followed by a seismic high from 240 m to 270 m. Based on borehole results, the seismic line appears to overestimate the actual depth of sound rock by about 8m in the dolerite.

6.4.2 Core drilling and water pressure testing

Three cored boreholes (DT 1, DR3 and DT2) fall along the upper tunnel line, while four holes (BH 1003, DTS1, DTS2 and DT5) have been used to interpret conditions along the lower line. Borehole DT5 is located about 30 m downstream of the lower line since the slope closer to the line was too steep for access.

The borehole positions are shown on **Figure A6.1** and the results are summarised in **Table 6.9**.

Table 6.9: Summary of borehole results along saddle embankment

BH No.	TYPE OF MATERIAL/DEGREE OF WEATHERING Organic soil = O, Sandy clay = SC, Gravel in clay = GC, Gravel in sand = GS Shale = S; Indurated shale = I, Dolerite = D					
	Trans-Ported Soil	Residual soil/ Completely Weathered	Highly Weath.	Moderately Weathered	Slightly Weath.	Un-Weathered
	Layer thickness (m)					
DT1	0.0 – 0.4 (O) 0.4 – 1.08 (SG)	1.08 – 1.7 (S)	1.7 – 5.4 (S) 8.0 – 8.43 (S)	5.4 – 8.0 (S) 8.43– 8.84 (D)	8.84– 13.6 (D)	13.6 – 35.01 (D)
DR3	0.0 – 1.08 (O) 1.08 – 4.4 (SC) 4.4 – 11.1 (GC)		11.1 – 14.8(S)	14.8 – 25.1 (S)	25.1 –28.0 (D)	28.0 – 40.1 (D)
DT2	0.0 – 0.76 (O) 0.76 – 1.96 (SG)		1.96 – 2.3 (S)	2,3 – 5.5 (S) 8.0 - 10.46 (S)	5.5 – 8.0 (S)	10.46- 16.25 (D) 16.25-20.0 (I) 20.0-25.11(S)
1003	0.0 – 1.0 (O) 1.0 – 2.0 (GS) 2.0 – 3.4 (GC)				3.4– 19.63 (D)	
DTS1	0.0 – 0.53 (O)	0.53 – 2.33 (S) 4.59 – 5.1 (S)	2.33 – 4.59(S)	5.1 – 7.76 (S)	7.76 –11.0 (D)	11.0 – 35.06 (D)
DTS2	0.0 - 0.36 (O) 0.36 – 5.0 (GC)		5.0 – 10.37 15.57–16.6(D)	10.37 -15.57	16.6–22.5(D)	22.5 – 25.1 (I)
DT5			0.0 – 1.1 (S)	1.1 – 5.84 (S)	5.84– 11.0 (S) 11.0-17.5 (I)	17.5 – 25.11(D)

Water pressure tests were conducted by using single packers in sections of 3 m length and maximum water pressures of about 15 x D for packers set in soil and 22 x D for packers set in rock (where D is the depth of the packer below ground surface). Each test section was subjected to five (ascending and descending) water pressures (each for a period of 10 minutes), and the corresponding water losses were measured. The Lugeon value for each increment of water pressure was calculated as follows:

Lugeon = *(1000 x V)/(T x L x P)*

where: *V* = *Volume of water pumped in litres*

T = *Time of water pumped in minutes*

L = *Length of test section in metres*

P = *Pumping pressure in kPa*

For each test section, five Lugeon values (one for each pressure increment) were calculated, and the most appropriate Lugeon value was selected from the flow pattern (laminar, turbulent, dilation, blockage or wash-out) according to the guidelines produced by Houlsby (Houlsby, 1976).

The results of the water pressure tests are presented in **Table 6.10**.

Table 6.10: Summary of water pressure test results along tunnel lines

BH No.	Depth untested (m)	LUGEON VALUES (only values above 0)		
		0.1 -1	1.1 – 4 (actual Lugeons)	>4 (actual Lugeons)
		Depth in m	Depth in m	Depth in m
DT1	0 - 2	5 - 11	2 – 5 (3.5)	
DR3	0 – 2 38 – 40.1	23 - 29		8 – 11 (4.2)
DT2	0 – 2 23–25.11	11 - 14	5 – 8 (3) 8 – 11 (1.2)	14 – 17 (9)
1003	0 - 3			
DTS1	0 – 2 32–35.06	2 – 5 8 – 11 20 – 26 29 - 32		
DTS2	0 - 2		5 – 8 (1.4) 14 – 17 (2) 17 – 20 (1.1)	2 – 5 (7) 8 – 11 (18) 20 – 23 (6.9)
DT5	0 – 2 23–25.11		8 – 11 (2.4) 11 – 14 (1.7) 14 – 20 (1.2) 20 – 23 (5)	2 – 5 (52) 5 – 8 (12)

6.4.3 Discussion of results

Although it has been proposed to construct 5 x 6 m diameter tunnels, this discussion of results will deal with two presumably representative tunnel lines along lines between Boreholes DT2 – DT1 (Upper Tunnel) and Boreholes DT5 (projected) – BH 1003 (Lower Tunnel).

Figures A6.17 and **A6.18** portray the conditions along the Upper Tunnel and Lower Tunnel respectively.

Differences in the elevation of the dolerite sill (e.g. on either side of Boreholes DRS3 and DT5), and in the thickness of the sill, together with some seismic anomalies, leave the possibility that the faults through Quarry Area I might intersect the tunnel lines. This will require additional investigations during the design stage.

The following material types are likely to be encountered:

- Moderately weathered shale
- Mixed face slightly weathered shale and dolerite
- Slightly weathered and unweathered dolerite
- Moderately weathered dolerite
- Highly weathered dolerite.

Rock mass parameters relevant to tunnel construction for the above material types are given in **Table 6.11**.

Table 6.11: Recommended tunnel support

MATERIAL TYPE	EXPECTED LENGTH (RANGE) (m)	SUPPORT TYPE			
		SHOTCRETE THICKNESS (mm)	ROCKBOLT SPACING (m)	STEEL SET SPACING (m)	OTHER
Highly Weathered Shale	10 - 20	Portal			
Moderately Weathered Shale	15 - 25	2 x 60 First layer immediate after exposure, second after bolting	1.5	n/a	Top heading and bench
Slightly Weathered Shale And Dolerite	20 - 40	2 x 50 mm First layer immediate after exposure, second after bolting	1.8	n/a	n/a
Slightly Weathered And Unweathered Dolerite	200 - 300	none	Ad hoc to support Unstable blocks	n/a	n/a
Moderately Weathered Dolerite	16 - 30	120	1.2	1.2	Top heading and bench
Highly Weathered Dolerite	20 - 30	Portal			

Depending on their positions, the 5 tunnels can vary in length between about 340 m and 400 m. In every case about one third of the tunnels will be in rock that requires substantial support, while the remaining part is in sound dolerite where only nominal rockbolt support might be needed.

Excavations for tunnel portals will result in steep slopes in moderately weathered shale (that is prone to rapid deterioration) and moderately to highly weathered dolerite (corestones in a soil matrix). Flattening to safe angles cannot be done due to the steep topography of the portal areas. On the upstream side, these slopes will be exposed to a fluctuating water level and slope failures might result in undermining of the upstream toe of the dam.

Provision will therefore have to be made for slope support, protection against erosion/slaking and drainage. This could be done by means of rock anchors, mesh-reinforced shotcrete and drainage holes.

7 RESERVOIR RIM STABILITY

7.1 BACKGROUND

Reservoirs surrounded by steep unstable slopes are subject to landslides that can displace material into the reservoir causing volumetric displacement of water and setting up surges and waves in the water body. This can lead to overtopping of the dam. Volumetric displacement by material can be dealt with as an incoming volume and subsequently leading to a rise in water level and capacity reduction. Calculation of the slip volume possibly threatening the dam can be made from a geological analysis of the surrounds of the basin. Three types of slips occur according to Vischer (Vischer, DL, 1986), namely (i) falls such as rock masses off a cliff with low volume and high energy intensity, (ii) slides such as slip-circle type slides also known as debris-flow and (iii) more gradual flows which are associated with long time intervals.

Not listed in the above types is the mechanism that caused the disastrous slide into the Vajont reservoir in Italy in 1963 when a large part of the mountain (260 000 000 m^3) on the left side slid along a curved bedding plane into the relatively small reservoir at a speed of about 30 m/s and created a wave of over 250 m high that swept 50 000 000 m^3 of water over the crest of the dam and also wiped out a town on the right bank. More than 2 500 people were killed on the right bank and downstream of the dam.

Huber and Hager (Huber A & Hager W.A, 1997) developed a generalised approach for estimating impulse waves under general conditions. Their earlier work is used for the calculation of wave heights in the SANCOLD Guidelines on Freeboard in Dams (South African National Committee on Large Dams, 1990) while Hager is the main author of a guideline for the calculation of landslide generated impulse waves in reservoirs published by the Laboratory of Hydraulics, Hydrology and Glaciology of the Swiss Federal Institute of Technology (VAW, 2009).

Input parameters to be obtained from a geological and topographical study are the location, volume, width and density (compactness) of potentially unstable material and the inclination of the sliding plane. From the guidelines by VAW (VAW, 2009) these data, together with information on water depth and positions of the critical impact areas (e.g. dam wall), can be used to predict wave heights at critical locations and wave run-up against slopes (e.g. dam walls).

7.2 GEOLOGY AND ROCK TYPES

The areas around the main dam and saddle dam and in parts of the dam basin are underlain by near-horizontally bedded rocks of the Volksrust Formation, Ecca Group, while the other parts of the dam basin are underlain by near-horizontally bedded rocks of the Estcourt and Adelaide Formations of the Beaufort Group. These formations belong to the Karoo Supergroup. Dolerite sills (see **Figure A7.1** included in **Annexure A**) had intruded these sedimentary strata mostly concordantly, while a few sub-vertical dolerite dykes are present.

The Volksrust Formation comprises dark grey shale, interbedded with sub-ordinate sandstone. Dark grey to black carbonaceous shale, siltstone and sandstone occur in the Estcourt Formation while the Adelaide Formation comprises siltstone, sandstone and sub-ordinate shale.

7.3 MODE OF WEATHERING AND STABILITY OF NATURAL SLOPES

7.3.1 Shale

Unweathered shale of this area is a very strong rock (UCS >150 MPa) and contains closely spaced bedding planes that are mainly horizontally disposed but occasionally dip at up to 7 degrees. The shale typically contains one or two moderately spaced near-vertical joint sets.

The shale rock mass has a low permeability and therefore provides little access for weathering agents, e.g. water and oxygen. However, when exposed to the atmosphere, the outer crust of the rock (100 mm – 150 mm) rapidly slakes and breaks in into gravel-size slivers of rock. Where these slivers are removed by water or gravity, the underlying rock continues to weather until a slope of about 30 degrees is reached as measured in the field.

7.3.2 Shale with interbeds of sandstone

Unweathered sandstone is stronger than shale (UCS > 200 MPa) and typically occurs as thin (100 mm – 300 mm) interbeds within the shale horizons. They dip at the same shallow angles as the shales. Being more brittle than shale, the vertical joint sets are typically closer spaced and the rock mass is generally more permeable.

Sandstone is very resistant to all forms of weathering and the sandstone layers usually stand out as vertical steps in the topography while the shale forms the flatter parts of the slopes. Steep slopes (up to 60 degrees) can develop in a shale/sandstone succession.

7.3.3 Sandstone

Where thick layers of sandstone occur, weathering is limited and very steep slopes (up to 75 degrees) can develop.

7.3.4 Shale/dolerite contact zones

The shale/dolerite contact zones are characterised by indurated shale and chilled (very fine-grained) dolerite, i.e. both rock types had been altered.

Indurated shale occurs in zones of up to 10 m thick along the dolerite contacts. It is typically very strong (>200 MPa) and bedding planes are generally less pronounced (welded), while a number of other joint sets may occur. These joints might be vertical or inclined and their orientations are difficult to predict. Indurated shale is generally not prone to rapid weathering (slaking).

The dolerite is fine-grained due to rapid cooling along the contact and had absorbed some of the silica from the shale, thus rendering the chill zone more resistant to weathering than unaltered dolerite.

Being more resistant to weathering than both shale and dolerite, the contact zones often form the top of plateaux where most of the original dolerite sills had been removed by weathering and erosion. Large areas indicated as dolerite on geological maps are actually underlain by shale/dolerite contact zones.

Natural slopes can vary between about 5 degrees on plateaux areas to about 40 degrees along escarpments.

7.3.5 Dolerite

Dolerite occurs as horizontally disposed layers that had intruded over large distances concordantly between layers of sedimentary rock. Occasional dolerite dykes occur in the area.

Unweathered dolerite is a very strong rock (UCS > 300 MPa) but often contains three or more sets of closely to medium spaced cooling joints that break the rock mass into more or less cubical blocks. Surface weathering agents (water and oxygen) enter the rock along these joints and weathering results in the formation of sub-rounded corestones within a matrix of clayey silt. When the soft matrix is removed by surface erosion, the rounded boulders remain on surface or roll down on account of gravity and eventually cover large areas on plateaux areas and along slopes. Due to a surface coating of iron oxides, these boulders are resistant to weathering and remain in place for a long time. Areas covered by dolerite boulders and doleritic soil are often shown as dolerite bedrock on geologic maps, while there is only a thin veneer (remnant of a dolerite sill) with weathered shale or sandstone occurring below.

Unweathered dolerite can form vertical slopes, but as a result of stress relief joints, slope angles in dolerite are typically 70 – 80 degrees. However, weathered dolerite comprising corestones in a matrix of clayey soil is notoriously unstable due to the clay content of the matrix. Slopes are characterised by slump features and are seldom steeper than 30 degrees.

7.4 MECHANISMS OF SLOPE FAILURE

The following mechanisms of slope failure might manifest in the dam basin:

7.4.1 Gradual undercutting of slope due to slaking of shale.

Where slaking and removal of slaked material is accelerated by wave action produced by the water surface of a dam, undercutting of the overlying stata will occur. Undercutting will not proceed far before a small volume of rock will separate along some of the closely spaced sub-vertical joints in the shale and fall into the water. The probability of failure is high (about once a year) but the volumes are very small.

7.4.2 Debris flow involving talus accumulated along many of the slopes

Many slopes are covered by doleritic talus comprising of dolerite boulders and clayey silt that are presently resting at a natural angle of repose. Saturation, rapid drawdown of the water level or earth tremors could serve as trigger to mobilise sliding of this material. The maximum thickness of scree occurs at the foot of slopes where sliding will not affect the dam water level, while higher up the slopes the thickness of the debris layer is expected to vary between 1 m and 5 m. The probability of this type of failure is moderate (about 1 event in 50 years) but the volumes are small.

7.4.3 Plane failure of a rock wedge on a joint plane that dips into the reservoir

Although no continuous steeply dipping joints have been observed anywhere in shale outcrops in the area, it would be prudent to consider the possibility of local continuous steep joints that dip towards the reservoir.

The dip of a potential sliding plane will have to be more than 25 degrees (typical lower bound friction angle in the shale) and less than the slope of the ground surface. The flatter the angle, the lower the risk of sliding and the larger the volume of the sliding mass, while steeper angles have higher risk of sliding but will mobilise smaller volumes.

The maximum volume of a potential slide depends on the angle of the plane(s) of which sliding occurs (lower bound of sliding mass), the slope of valley side (lateral boundary), the height of the slope (upper boundary) and the (horizontal) length of the section of slope that can fail.

The effect of a slide on the dam wall depends on the volume of water displaced by the sliding mass, the speed at which the sliding takes place and the position of the slide with respect of the dam wall.

In the case of the Smithfield reservoir, plane failure is the only mechanism than can result in a significant volume of slide material resulting in displacement of water and wave run-up. However, from a study of the geology, it was concluded that the probability of having a plane of weakness that dips steeper than 25 degrees into the reservoir and is continuous both along strike and dip directions for more than 20 m is extremely low.

7.5 INVESTIGATION OF THE DAM BASIN

The dam basin is located in an area where the uMkhomazi River had incised a deep valley into the surrounding landscape. Its tortuous course is structurally controlled by main joint sets in the sedimentary rocks and resistance provided by dolerite dykes and dolerite sills. The insides of bends usually have gentle slopes while outside beds have steeper slopes due to undercutting by the river.

The dam basin is about 12 km long as measured along the river, and the surface area of the reservoir at FSL is 9.53 km^2. The top 1 m layer of the dam basin thus represents a volume of 9 530 000 m^3. It also means that if 9.5 million m^3 of material gradually slides into a full dam, the water level will rise by about 1 m. If the slide takes place fast, a higher impulse wave might occur and its effect will depend on the position of the slide with respect to the dam wall.

The investigation took place in February and March 2014 and was split into the following phases:

7.5.1 Desk study

The desk study involved an inspection of geological maps, topographical maps and satellite imagery (Google Earth).

The available 1 m interval contour map of the dam basin does not extend above the full supply level (FSL) of the dam (930 masl) and this hampered the initial identification of potential critical slopes around the basin.

From a study of the contour map, 17 potential unstable slope areas were identified on the basis of slope angle (steeper than 25 degrees) and the position of the steep section along the slope. Where the steep section of the slope is located more than 20 m below FSL, it was argued that the probability of triggering a slide due to rapid drawdown is small, and should it occur, the dam level will be too low for overtopping. The positions of the 17 potential slide areas are shown on **Figure A7.2** included in **Annexure A**.

7.5.2 Field investigation

During the field inspection of the identified slopes, a GPS was used to determine the positions and elevations of points above the 930 masl contour. Although not very accurate (barometric heights), these points were used to determine the gradient of these slopes above FSL.

From the geological map and field visits, the rock types forming the slopes were identified. Where possible, the orientations and continuity of major joint planes that intersect the rock faces were inspected. Unfortunately in some cases the slopes were covered by scree or very dense vegetation and no rock outcrops were visible.

Details of the 17 areas of potental slope instability are given in **Table 7.1**.

Photographs of the more critical slopes were taken to further illustrate the inclinations, heights and geology. These are included in **Annexure D**.

7.5.3 Analysis

Of the 17 identified slopes, it was found that only 4 were steeper than 25 degrees and also in direct line of sight from the main or saddle dams. According to Huber and Hager (Huber A & Hager W.A, 1997), wave action from slides that are out of sight due to topographic features will have little impact on structures. One slope (Slope S13) that is out of sight of the dam walls, was identified as a potential slide that might result in large volumetric displacement and overtopping of the dam. Sections through the above 8 slide areas are shown on **Figure A7.3** included in **Annexure A**.

Table 7.1: Details of potential areas of slope instability

Slope area no.	Steepest section (degrees)	Width of potentially unstable part (m)	Area of potentially unstable part (m^2)	Type of sliding material	Average thickness of potentially unstable material (m)	Volume of potential sliding mass (m^3)	Comment
1	<25	Not measured					<25 degrees
2	30	400	20 000	Gravel*	0.2	4 000	Table 2
				Shale	2	40 000	Table 2
3	<25	Not measured					<25 degrees
4	<25	Not measured					<25 degrees
5	<25	Not measured					<25 degrees
6	42	200	40 000	Talus	2	80 000	Table 2
				Shale	5	200 000	Table 2
6a	72	200	20 000	Gravel*	0.3	6 000	Table 2
				Shale	25	500 000	Table 2
7	56	300	12 000	Talus	3	36 000	Table 2
				Shale	5	60 000	Table 2
8	36	250	28 000	Talus	3	84000	Out of sight
				Shale	7	196 000	
9	45	700	40 000	Talus	3	120 00	Out of Sight
				Shale	15	600 000	
10	55	300	24 000	Talus	3	72 000	Out of Sight
				Shale	25	600 000	
11	45	500	28 000	Gravel*	0.3	8 400	Out of Sight
				Shale	17	476 000	
12	50	100	6 000	Gravel*	0.3	1 800	Out of Sight
				Shale	17	102 000	
13	50	300	40 000	Talus	3	120 000	Table 2
				Shale	22	880 000	Table 2
14	30	150	6 000	Talus	3	18 000	Out of sight
				Shale	2	12 000	
15	<25	Not measured					< 25 degrees
16	36	220	12 000	Talus	3	36 000	Out of Sight
				Shale	3	36 000	
17	72	200	6 000	Talus	0,1	600	Out of sight
				Shale	25	150 000	

Note: Gravel denotes the slivers of shale that form as a result of slaking.*

The method proposed by VAW (VAW, 2009) was used to calculate the wave height and wave run-up at the centre of the main and saddle embankments as a result of complete rapid failure of each potential slide area. The results are given in **Table 7.2**.

Table 7.2: Probability of failure and effect on dam wall(s)

Slope area no.	Type of sliding material	Volume of potential sliding mass (m^3)	Probability of failure	Wave height main dam (m)	Run-up main dam (m)	Wave height saddle (m)	Run-up saddle dam (m)
2	Gravel*	4 000	moderate	0.00	0.00	0.16	0.18
	Shale	40 000	extr. low	0.00	0.00	0.67	0.92
6	Talus	80 000	moderate	1.15	1.41	0.09	0.09
	Shale	200 000	extr. low	2.12	2.84	0.15	0.18
6a	Gravel*	6 000	moderate	0.21	0.21	0.02	0.02
	Shale	500 000	extr. low	3.13	4.40	0.08	0.09
7	Talus	36 000	moderate	0.00	0.00	0.58	0.77
	Shale	60 000	extr. low	0.00	0.00	0.88	1.17
13*	Talus	120 000	moderate	0.01	0.01	0.01	0.01
	Shale	880 000	extr. low	0.10	0.10	0.10	0.10

Note: Rise in water level due to displacement – no impulse wave.

7.6 CONCLUSIONS

There is a moderate (1:50 year) probability of a talus/gravel failure from slopes (Areas 6 and 6a that are located about 1.5 km from the dam walls) that will result in a run-up wave of up to about 1.4 m against the main dam wall. There is also an extremely low (1:10 000 year) probability of a large rock slide from the same slope area that will result in a run-up to about 4.4 m against the main dam wall. The available freeboard will prevent overtopping of the dam wall in the event of such failures. Failures of the other identified slopes will have much smaller effects due to smaller potential slide volumes, longer distances from the dam walls and topographic barriers between the slide areas and the dam walls.

8 CONCLUSIONS

8.1 AVAILABLE INFORMATION

Information from published geological maps was used to describe the general geology of the area.

The pre-feasibility geotechnical investigation provided valuable information on sources for impervious embankment material and also limited information on founding conditions for a lower dam.

8.2 GENERAL GEOLOGY

The area around the site is underlain by rocks of the Volksrust Formation of the Ecca Group, comprising shales (mudrocks) with sub-ordinate sandstones. The sedimentary strata are essentially horizontal, and largely undisturbed. Regional dips of 3 – 7 degrees are recorded, while locally steeper dips are recognised and are ascribed to the intrusion of dolerites. Three near-horizontal dolerite sills have intruded mainly concordantly into the sedimentary strata and are responsible for the narrow river valley at the dam site and the presence of good quality rock for concrete aggregate and rockfill. A few faults with throws of up to 10 m have been mapped and one dolerite dyke traverses the left flank quarry area.

8.3 SEISMIC REFRACTION SURVEYS

Seismic refraction surveys have been conducted across the proposed quarry areas, the dam centre line and the diversion tunnels. Although the seismic velocities tended to over-estimate the depth of sound rock, they were extremely useful in showing the presence of dolerite sills below a cover of shale and also to identify the positions of faults.

8.4 SEISMIC RISK ANALYSIS

A Probabilistic Seismic Hazard Analysis (PSHA) was conducted by Dr A Kijko of the Natural Hazards Assessment Consultancy.

Results for the horizontal component of earthquake acceleration are as follows:

- Operating Basis Earthquake (Return period 144 years) = 0.016 g
- Maximum Design Earthquake (Return Period 475 years) = 0.021 g
- Maximum Credible Earthquake (MCE (Return period 10 000 years) = 0.113 g

The above results classify the site as of low seismic risk.

8.5 MATERIALS INVESTIGATION

8.5.1 Impervious fill

Three borrow areas located within 1.5 km from the dam and below 930 masl on the left side of the river were investigated as sources for impervious material for use in a ECR or zoned embankment dam. Two of these areas can provide about 1.65 million m^3 that is 150 000 m^3 short of the required 1.8 million m^3. This shortage can be supplemented by using some of the completely and highly weathered shale (overburden) from the rock quarry or soil overburden from the plunge pool excavation.

8.5.2 Concrete aggregate and rockfill

Four areas located within 1.5 km from the dam and below 930 masl on were investigated as sources for rockfill, concrete aggregate, rip-rap and filters.

The main quarry will produce up to 2.6 million m^3 of hard dolerite suitable as rockfill or concrete aggregate. However, in order to mine this material, about 0.6 million m^3 of completely to highly weathered shale and 0.6 million m^3 of moderately weathered to unweathered shale (and in some places weathered dolerite), have to be removed. The shales are prone to rapid disintegration upon exposure but can be used in a zoned soft rock/hard rock embankment, provided they are used in the inner zones and is protected on the outside by durable dolerite.

Sufficient hard dolerite is available for construction of a RCC dam.

Rock suitable as concrete aggregate for the tunnel lining and intake structure is also available.

If the shale overburden and underlying dolerite is combined, and the floor of Quarry 1 is excavated to elevation 865 masl, sufficient rockfill material is available for a zoned embankment with soft rock inner zones and durable hard outer shells.

8.6 SUMMARY OF AVAILABLE VOLUMES

A summary of the volumes of available construction materials is presented in **Table 8.1**.

Table 8.1: Summary of available volumes

SOURCE	TYPE OF CONSTRUCTION MATERIAL					
	Overburden for spoil: Organic topsoil	Impervious: Clayey sand transported material	Semi-pervious: Completely and highly weathered shale	Shale rockfill: Unweathered to moderately weathered shale	Dirty rockfill: Highly and moderately weathered dolerite	Hard rockfill, aggregate, rip-rap and filters; Unweathered and slightly weathered dolerite
Borrow Area A	120 000	800 000	0	0	50 000	0
Borrow Area B	100 000	850 000	0	0	100 000	0
Borrow Area C	0	0	0	0	0	0
Quarry I	50 000	20 000	600 000	600 000	140 000	2 600 000
Quarry II	40 000	200 000	170 000	44 000	850 000	720 000
Quarry III	20 000	25 000	20 000	10 000	815 000	123 000
Quarry IV	5 000	7 000	110 000	13 500	0	0
Excavation main wall: RCC dam	8 000	120 000	210 000	0	62 000 (Alluvial boulders and clay)	0
Excavation main wall: ECR Dam	56 000	380 000	0	0	200 000 (Alluvial boulders and clay)	0
Excavation main wall: CFR Dam	56 000	380 000	0	0	200 000 (Alluvial boulders and clay)	0
Excavation ECR saddle dam	0	0	110 000	0	0	0

8.7 Geotechnical Investigation

8.7.1 Concrete gravity dam

Founding conditions are generally not suitable because of deep weathering of shales along the higher flanks and the presence of a thick (up to14 m) layer of transported material on the right flank. However, in the central section a concrete dam can be founded on strong dolerite and strong indurated shale at depths between 2 m and 11 m. A concrete lined stilling basin will be required to protect the downstream rock against erosion.

8.7.2 Rockfill dam

The shells and plinth of a rockfill dam can be founded at depths between 3 m and 10 m on the left flank, 1.5 m and 5 m in the central section and 3 m to 15 m on the right flank.

8.7.3 Core trench

The core trench for embankment dams can be founded at between 4 m and 10.6 m on the left flank, 2 m and 11m in the central section and 3.5 and 15 m on the right flank.

8.7.4 Grouting

Lugeon water tests generally showed very low permeabilities but low gradients of the natural water table indicate the opposite. A grout curtain to a depth of 66% of the water head will have to be provided.

8.7.5 Spillway

The control structure for a side spillway on the upper left flank can be founded on slightly weathered shale at depths ranging between 15 m and 20 m below ground surface and the concrete lined channel can be founded on moderately weathered shale at depths of between 10 and 12 m.

8.7.6 Saddle embankment

The clay core of an earthfill or rockfill dam can be founded on moderately weathered shale that occurs at depths of between 2 m and 4 m.

The foundations are generally impervious, but a grout curtain is nevertheless recommended. If the proposed quarry is developed upstream of the saddle dam, a deep grout curtain will be required.

8.7.7 Diversion tunnels

Depending on their positions, the five proposed tunnels can vary in length between about 340 m and 400 m. In every case about one third of the tunnels will be in rock that requires substantial support, while the remaining part is in sound dolerite where only nominal rockbolt support might be needed.

Excavations for tunnel portals will result in steep slopes in moderately weathered shale (that is prone to rapid deterioration) and moderately to highly weathered dolerite (corestones in a soil matrix). Flattening to angles cannot be done due to the steep topography of the portal areas. On the upstream side, these slopes will be exposed to a fluctuating water level and slope failures might result in undermining of the upstream toe of the dam.

Provision will therefore have to be made for slope support, protection against erosion/slaking and drainage. This could be done by means of rock anchors, mesh-reinforced shotcrete and drainage holes.

8.8 RESERVOIR RIM STABILITY

There is a moderate (1:50 year) probability of a talus/gravel failure from slopes (Areas 6 and 6a) located about 1.5 km from the dam wall(s) that will result in a run-up wave of up to about 1.4 m against the main dam wall. There is also an extremely low (1:10 000 year) probability of a large rock slide from the same slope area that will result in a run-up to about 4.4 m against the main dam wall. The available freeboard will prevent overtopping of the dam wall in the event of such failures. Failures of the other identified slopes will have much smaller effects due to smaller potential slide volumes, longer distances from the dam walls and topographic barriers between the slide areas and the dam walls.

9 RECOMMENDATIONS

The construction and materials investigation was intended to provide information for various types of dams and to facilitate the selection of the type of dam and the most appropriate construction materials. Once that is done, the following additional geotechnical investigations for the specific layout and design are recommended:

9.1 FOR EARTH CORE ROCKFILL DAM

The suitability of the transported soils along the centre line on the right flank of the dam for use as embankment fill has to be investigated by means of excavator test pits, sampling and laboratory testing. This material must be excavated before a rockfill dam can be founded and it might be suitable for construction of the saddle embankment.

The soil overburden in the area for the plunge pool on the left flank has to be investigated by means of excavator test pits, sampling and laboratory testing. This material might be suitable for supplementing the required volume of impervious material for the clay core of the dam.

Additional inclined boreholes must be drilled and water tested in the area of Borehole DLS2 on the left flank to investigate the possible presence of a fault as cause for the anomalous high water losses in that hole.

Additional inclined boreholes must be drilled and water tested in the area of Boreholes DT5 and DR3 on the right flank to investigate the possible presence of faults that may result in sections of poor tunnelling conditions along the diversion tunnels.

Additional boreholes must also be drilled to confirm portal conditions for the diversion tunnels.

Additional laboratory tests must be undertaken to confirm the permeability and dispersivity of the impervious materials from Borrow areas A and B.

Additional laboratory tests must be conducted to determine the compaction characteristics and shear strength of the slightly weathered and unweathered shale for use as rockfill materials.

9.2 FOR CONCRETE GRAVITY DAM

Shear tests must be conducted along bedding planes in core samples of the shale on which the concrete dam is to be founded.

9.3 FOR OTHER STRUCTURES

Foundation investigations by means of core drilling must be undertaken at sites for gauging weirs.

Centre line investigations for temporary roads and for permanent road deviations must be undertaken by means of test pitting, sampling and laboratory testing.

Construction materials investigations for temporary roads and for permanent road deviations must be undertaken by means of test pitting, sampling and laboratory testing.

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