NCWABENI OFF-CHANNEL STORAGE DAM FEASIBILITY STUDY: MODULE 1: TECHNICAL STUDY

SUPPORTING REPORT 2:

GEOTECHNICAL AND MATERIALS INVESTIGATION

FINAL

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EXECUTIVE SUMMARY

TERMS OF REFERENCE AND SCOPE OF WORK

A pre-feasibility study by DWA for the Ncwabeni Off-Channel Storage Project near Port Shepstone that was completed in February 2005, identified two alternative dam sites namely the D2 site on the Ncwabeni River and the D3 site on the Gugamela River. Capital costs and the UVRs associated with the two alternatives were found to be almost identical but Site D2 was preferred from environmental and social perspectives. Selection of the preferred option would therefore largely depend on costs associated with geotechnical conditions at the dam sites (including the possibility of a fatal flaw at D2), availability of construction materials and slope stability.

During 20011, BKS (Pty) Ltd undertook feasibility stage geotechnical and materials investigation to assist with the site selection and other studies which will allow accurate costing and commencement of construction works.

The purpose and scope of the geotechnical investigations were to:

- *Review the available geotechnical information.*
- *Describe the general geology of the area and prepare a geological map.*
- *Investigate any potential flaws for construction of a dam at Site D2.*
- *Investigate sources for dam construction materials.*
- *Undertake geotechnical investigations for the dam foundations, spillway structure, diversion weir in the Umzimkulu River, pump station and the rising main.*
- *Assess the stability of slopes around the reservoir rim.*

This report describes the site conditions, based on a desktop study and field investigations conducted from 21 February 2011 to 3 March 2011 and from 25 July 2011 to 7 October 2011. Laboratory testing was completed in December 2011.

GEOLOGY

The project area including the dam centre line, the spillway line, weir sites, pump station site, quarry site and most of the borrow areas are underlain by granitic bedrock of the Oribi Gorge Suite. This rock is weathered to various degrees and depths across the area, with slightly weathered to unweathered rock on surface or at shallow depth near the rivers and deeper weathering in the higher lying areas.

The bedrock is generally covered by layers of colluvial clayey silty sand in the higher lying areas, while there are well-developed layers of alluvial silt, sand and gravel closer to the rivers.

FATAL FLAW

During the initial stages of the investigation, attention was given to the possible occurrence of a fatal flaw at Site D2. However nothing that would rule out the construction of a dam or would result in excessive cost was identified.

CONSTRUCTION MATERIALS

It was not possible to locate sufficient quantities of impervious core material for construction of a zoned embankment dam within the dam basins or elsewhere in the area. The only alternative type of embankment is a concrete-faced rockfill dam. Such a dam comprising of "soft" and "hard" rockfill zones can be constructed from completely and highly weathered granite (soft *rockfill) and moderately weathered to unweathered granite (hard rockfill) that can be obtained from a proposed quarry in the D2 dam basin. Good quality concrete aggregate for construction of the concrete face can be obtained from rock in the bottom part of the quarry.*

DAM FOUNDATIONS

A wide fault zone and generally deep weathering on the left flank of site D2 will require very deep excavation for a concrete gravity (RCC) dam indicating that its cost will far exceed those for embankment or composite dams.

A composite dam with a concrete overspill section in the river and along the right flank might be considered, but the left embankment will have to be supported by a long flank wall, the founding levels of which have not been investigated, but are expected to be about 10 m deep.

Investigations along the dam centre line show that founding conditions for a rock-fill dam are favourable with excavation depths not exceeding about 5 m and grouting to depths of between 20 m and 40 m. However, the plinth for an upstream faced rockfill dam will be located a considerable distance upstream of the dam reference line, and for the design purposed, additional geotechnical investigations will have to be conducted.

SPILLWAY AND RISING MAIN

The proposed spillway control structure on the upper left flank may have to be relocated because of deep weathering in that area. The proposed return channel will need to be concrete-lined and will be founded on highly weathered (soft rock) granite with local zones of completely weathered granite (very soft rock).

It is recommended that additional investigations be conducted during the design stage to select the most favourable position for the spillway control structure and to determine the required founding levels.

WEIR FOUNDATIONS

The lower weir site has good founding conditions ion the left flank and in the river section. However, the flat-lying right flank is underlain by thick alluvium.

PUMP STATION

Alluvium of variable thickness is underlain by good quality founding rock. Founding levels will depend on the position selected for the pump station.

RESERVOIR SLOPE STABILITY

In the unlikely event of a slope failure along the rim of the reservoir, the volume of mobilised material will be very small (less than 0.3% of the reservoir volume) and the effect on the dam level will be minimal.

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List of Acronyms

ACR	Asphalt core rockfill
BCR	Bentonite core rockfill
CCR	Clay core rockfill
CFT	Concrete faced rockfill,
DWA	Department of Water Affairs
EIA	Environmental impact assessment
FSL	Full storage level
GPS	Global positioning system
LL	Liquid Limit
LS	Linear Shrinkage
Masl	mean annual sea level
MDD	Maximum Dry Density
OCS	Off-channel Storage
OMC	Optimum Moisture Content
PGA	Probable Maximum Acceleration
PI	Plasticity Index
PSP	Professional service provider
RCC	Roller compacted concrete
RQD	Rock quality designation
RWSS	Regional Water Supply Scheme
TPH	Test Pit (Horseshoe area)
URV	Unit reference value
USCS	Unified Soil Classification System

1 INTRODUCTION

1.1 BACKGROUND TO THE PROJECT

The Umzimkhulu Regional Water Supply Scheme (RWSS), which forms part of the KwaZulu Natal's Lower South Coast System, supplies water to the coastal region from Hiberdeen to Margate, including Port Shepstone. The water is presently sourced from non-regulated river flows in the Umzimkhulu River. Abstraction is at the St. Helen's Rock works near Port Shepstone where water is treated and from where it is distributed to various user nodes.

The *Southern KwaZulu-Natal Water Resources Pre-feasibility Study Phase 1* (DWA, 2002), concluded that during dry periods, the river flow is sufficient to meet the water requirements, even without provision for the release of the ecological Reserve. The study recommended that, in order to provide for the water requirements for all user sectors, including the Reserve, the construction of an off-channel storage (OCS) dam in one of the tributaries to the Mzimkhulu River, should be considered. The reservoir can be filled from its incremental catchment, supplemented by pumping from the Mzimkhulu River during times of high river flows. During times of low flows water can be released back into the Mzimkhulu River for abstraction downstream at the existing St. Helen's Rock abstraction works.

The *Southern KwaZulu-Natal Water Resources Pre-feasibility Study Phase 2* (DWA, 2005), investigated numerous options with regard to the position of the potential OCS dams. Four competitive sites which are located about 20km north-west of Port Shepstone were selected as the most feasible OCS dam sites. Two of the sites (D2 and D2A) are located on the Ncwabeni River while the other two (D3 and D3A) are on the Gugamela River. Conceptual designs for dams at these sites were undertaken as part of the afore-mentioned study.

Following on the above, the Reconnaissance Phase of the *Mzimkhulu River Off-Channel Storage Pre-feasibility Study* (DWA, 2007), re-assessed all four OCS dam options on the basis of more detailed hydrological modelling and updated information regarding water requirements, topographical surveys, geotechnical and flood hydrology data. It was established that the D3 site on the Gugamela River and the D2A site on the Ncwabeni River were distinctly less favourable than the other two sites and were therefore not investigated further. The study concluded that the geological conditions for the sites on the Ncwabeni River are superior to those on the Gugamela River and that the construction of a Roller Compacted Concrete (RCC) dam at the D2 site on the Ncwabeni River appeared the most feasible option.

Subsequently the *Ncwabeni Off-channel Storage Dam Feasibility Study: Module 1: Technical Study* (this study) was initiated to conduct a comprehensive engineering investigation at the feasibility level for the proposed Ncwabeni Off-Channel Storage Scheme. The possible dam at site D2 on the Ncwabeni River was to be considered first and if a fatal flaw or substantial increase in cost is identified, site D3A on the Gugamela River should then be considered.

1.2 SCOPE AND ORGANISATION OF THE STUDY

The key objectives of the study are to:

- Recommend the optimum scheme configuration;
- Undertake feasibility level dam foundation, reservoir slope stability and construction material investigations including quarries;
- Undertake the necessary supporting investigations and studies to support the feasibility study required for the implementation of the scheme;
- Do sufficient design of infrastructure to obtain cost estimates;
- Collaboration with the appointed PSP that will be responsible for the EIA process;
- Optimise the engineering and economic parameters and determine cost estimates for the following components of the scheme; the dam, pipeline, pump station, abstraction works, diversion weir and access roads; and
- Provide institutional arrangements for the smooth implementation of the scheme.

The sequence of activities in order from beginning to end followed in this feasibility study included the following:

- Water resource including yield analysis for both the Gugamela and Ncwabeni Dam sites;
- Foundation and construction materials investigation;
- Cost comparison of dam types for both Gugamela and Ncwabeni Dams for the yield associated with the most likely water demand;
- Selection of site and dam type;
- Cost comparison of scheme for incremental yield to the highest water demand;
- Hydraulic model study of Mzimkhulu River to identify three possible sites for abstraction works;
- Selection of the best layout for abstraction works and diversion weir.
- Selection and cost comparison of abstraction work, diversion weir, pump station, pipeline and access road layouts for the three identified sites;
- Hydraulic model study of selected abstraction work, diversion weir and pump station for optimisation;
- Optimization and cost comparison analysis of selected scheme;
- Conceptual design of selected scheme; and
- Determination of URV of water supplied.

The activities specific to the ***geological and materials task*** included

- Review the available geotechnical information.
- Describe the general geology of the area and prepare a geological map.

- Investigate any potential flaws for construction of a dam at Site D2.
- Investigate sources for dam construction materials.
- Undertake geotechnical investigations for the dam foundations, spillway structure, diversion weir in the Mzimkulu River, pump station and the rising main.
- Assess the stability of slopes around the reservoir rim.

1.3 PURPOSE OF THE REPORT

The purpose of the report is to present 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 (21 February 2011 to 3 March 2011 and from 25 July 2011 to 7 October 2011). Laboratory testing was completed in December 2011.

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

2 PREVIOUS INVESTIGATIONS

2.1 SEISMIC HAZARD ANALYSIS

The Council for Geoscience (2005) reported on a Probabilistic Seismic Hazard Analysis for the proposed site and considered the effect of all the historically recorded earthquakes located within a radius of 320 km from the site to determine

- the mean return periods for the Probable Maximum Acceleration (PGA);
- the annual probability of being exceeded for a specified value of the PGA, and
- uniform ground acceleration spectra.

The service, abnormal and extreme curves show a spectral acceleration peak of approximately 0.02g (at 3Hz), 0.05g (at 5Hz) and 0.15g (at 20Hz).

2.2 FOUNDATION INVESTIGATION

In May 2007, the Council for Geoscience (2007) reported on an engineering geological pre-feasibility study of Site D2 on the Ncwabeni River and Site D3A on the Gugamela River.

The investigations comprised a desktop study of available information, exploratory drilling, discontinuity measurements and limited laboratory testing. At site D2, which is being considered, two rotary-cored boreholes were drilled on the right flank (between the road and the river) and three holes were drilled on the left flank between the river and elevation 139 masl (the present proposed full supply level (FSL) is about 162 masl).

Core recovery and RQD values in the very coarse-grained, moderately weathered granite were generally low due to poor drilling methods, and this raised questions about the predicted excavation depths for a concrete structure of 7-8 m on the lower left flank and up to 25 m on the upper left flank. A review of the log of BH 1004 on the lower left flank showed that founding quality rock was encountered at a depth of 18.4 m, which is much deeper than the depth of 7-8 m that the Council for Geoscience’s report predicted. It was recommended that the possible effects of a major fault and several other faults on the left flank be investigated.

The right flank was considered too steep for an embankment dam. For a clay core on the left flank, excavation depths of 3-6 m were predicted.

2.3 MATERIALS INVESTIGATIONS

In June 2007, the DWA Materials Laboratory reported on a materials investigation within the basin areas of Sites D2 and D3A (DWA, 2007). It was predicted that at Site D2, about 50 000 m^3 of semi-pervious material and 55 000 m^3 of impervious material could be obtained. At Site D3A, the predicted volume of semi-pervious material is about 6 000 m^3 and about 385 000m^3 of impervious material.

3 INVESTIGATION METHODS AND STANDARDS

3.1 SITE CONDITIONS

The study area is densely vegetated, mostly by indigenous trees. Given that access was almost impossible due to the dense vegetation, the bush was cleared using a 20-ton tracked excavator along the centreline and spillway line to facilitate the geophysical survey and the setting out and excavation of test pits and boreholes.

3.2 GEOLOGICAL MAPPING

Access to the area is very restricted by the dense vegetation, while thick soil cover masks the bedrock and geological structures everywhere except along stream and river courses. Geological mapping of the ground surface was therefore only possible along the road, the river and the access tracks for the drilling and test pitting, while subsurface information was available from the geophysical survey, the test pits and boreholes.

The geological map on a scale of approximately 1:64 000 (Figure 2 in the Council for Geoscience pre-feasibility report) was used as a basis, and was updated with the additional information from the present investigations. This map is shown in **Figure A2**.

3.3 SEISMIC REFRACTION SURVEY

Seismic refraction surveys were undertaken to determine the depth to sound bedrock along the dam centre line, the spillway line and a proposed quarry site, and to identify local zones of deeper weathering characteristic of fault or shear zones. The results of a seismic survey are used to select the most appropriate positions for exploratory boreholes and to assist with the interpretation of geotechnical conditions between boreholes.

Seismic refraction surveys along the left flank of the dam centre line (400 m), the spillway line (200 m) and at a proposed quarry site (196 m) were conducted by E&EGS cc in March 2011. The report is attached as **Annexure F**.

The very steep slopes of the right flank and the lower part of the spillway line made it impossible to survey these areas. For the same reason, only one seismic traverse could be done along a contour at the proposed quarry site.

3.4 TEST PITS

Test pits were excavated to investigate the subsurface conditions to a depth of about 4 m. A Komatsu PC220 20-ton excavator with a 1.14 m wide bucket was used (instead of a TLB) to speed up the work and to penetrate the layer of hardpan ferricrete. The refusal depth of a 20-ton excavator can usually be taken to represent the founding depth for the core trench of an embankment dam.

Table 1 shows that 69 test pits were excavated during the site investigations. Eight test pits were excavated along the proposed dam centreline, four along the spillway line and four in the spillway approach area. Test pits were also excavated to identify suitable embankment materials for the construction of the dam: 14 in the area referred to as the horseshoe, nine in the D2 basin and 30 pits in the D3 basin.

Table 1: Summary of Test Pit Excavations

Project Area	Number of Test Pits	Test Pit IDs
Dam Centreline	8	TP01 – TP08
Spillway	4	TP09 – TP12
Spillway Approach Area	4	TP13 – TP16
Horseshoe	14	TP17 – TP24 and TPH01-TPH06
D2 Basin	9	TP25 – TP33
D3 Basin	30	TP34 – TP54, TP56 and TP46a - TP52a, TP54a

The test pits were excavated to a maximum depth of 4 m or to partial refusal (characterised by slow progress), whichever occurred first. Test pits were profiled by an engineering geologist according to the current standards and practice in South Africa (Brink and Bruin, 2002). Representative samples were taken for laboratory testing.

Each test pit was positioned using a hand-held GPS and were subsequently surveyed to an accuracy of 200 mm horizontally and vertically. The plans shown in **Figures A3** and **A4** in **Annexure A** indicate the positions of the test pits.

3.5 ROTARY CORE DRILLING

Rotary core drilling was undertaken to obtain relatively undisturbed samples of soil and rock to depths of 20-40 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 (Brink and Bruin 2002).

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 clayey materials, the packer was seated within a tight-fitted (drilled-in) casing, while in sandy / gravelly material, pre-grouting and re-drilling of the packer section was conducted. In this way, packer tests could be conducted from 1.5 m depth.

Samples comprising highly to moderately weathered granitic rock from boreholes in the quarry were taken for laboratory tests to determine their properties with respect to their use as soft rockfill.

Table 2 summarises the distribution of the core-drilled boreholes. Positions of boreholes were surveyed to an accuracy of 200 mm both horizontally and vertically. These positions are shown on **Figure A5.**

Table 2: Summary of Cored Boreholes

PROJECT AREA	NUMBER OF BOREHOLES	BOREHOLE No's
D2 Dam centre line	10	BL1 - BL8; BR1-BR2
D2 Spillway line	5	BS1 - BS5
D2 Spillway approach	3	BQ1, BQ3, BQ4
D2 Quarry	17	BQ7 - BQ20
		BQ22-BQ24
Weir sites	3	BW1 - BW3
Pump station	2	BP1 - BP2

3.6 LABORATORY TESTING

Disturbed soil samples were taken from the test pits for the following tests at the laboratory:

- Grading analysis to determine the particle size distribution, including hydrometer determination of the clay fraction,
- Determination of the Atterberg Limits to classify the soil and to give an indication of the soil properties,
- Proctor compaction,
- Permeability testing, and
- Tri-axial testing.

Two samples comprising highly to moderately weathered granitic rock from boreholes in the quarry were taken for the following laboratory tests to determine their properties with respect to their use as soft rockfill:

- Crushing to minus 25 mm,
- Grading analysis, including hydrometer determination of the clay fraction,
- Determination of the Atterberg Limits, where possible,
- Proctor Compaction and Tri-axial Testing.

4 GEOLOGY

4.1 GENERAL GEOLOGY

Information on the regional geology was obtained from Sheet 3030 Port Shepstone of the Geological Series of South Africa, at a scale of 1:250 000, while the geology of the surrounding area was mapped by the Council for Geoscience, as shown on **Figure A2**.

The geology, as inferred from the available maps, shows that the region is generally underlain by granitic rocks of the Oribi Gorge Suite of the Natal Structural and Metamorphic Province of Namibian Age. The geological maps indicate that the periphery of the study area is underlain by diamictite (mainly tillite) of the Dwyka Formation and by sandstones and shales of the Ecca Group of the Karoo Supergroup.

Based on the geological map, the D3 Basin is underlain predominantly by alluvial deposits of quaternary age, which is underlain by the Oribi Gorge Suite. The D2 basin is predominantly underlain by granitic rocks that are weathered to varying depths.

4.2 ORIBI GORGE SUITE

The Oribi Gorge Suite forms part of the Natal metamorphic province. Approximately 1 000 million years ago, subduction and collision along the southern margin of the Kaapvaal Craton produced the rocks of the Natal Metamorphic Province. The rocks were heated and deformed into a mountain range many thousands of kilometres long. Late-syntectonic Oribi Gorge granitoids, represented in the field by very coarse grained porphyritic granite, were emplaced after the cessation of the Margate and Mzumbe terranes and were only subjected to the waning stages of deformation. They were emplaced into and affected by large-scale shear zones associated with the collision of the Natal Metamorphic Province and the Kaapvaal Craton to the north.

4.3 DWYKA TILLITE

South Africa was part of the super continent Gondwana that drifted over the South Pole approximately 300 million years ago. Glacial activity of the region at the time scoured the sandstone and basement rocks beneath the sandstone to produce an extensive glacial pavement. Over a period of 120 million years, the glaciers retreated, depositing the poorly sorted diamictite (mainly tillite) unconformably into the lower part of the Karoo Supergroup.

4.4 ALLUVIAL DEPOSITS

Alluvial deposits, which are of quaternary age, usually comprise a sequence of sands, silts and clays. Climatic changes affect all aspects of a drainage system. The rise and fall of water levels lead to the deposit of sequences of sands, silts and clays. These deposits are usually derived from the weathering of rock types found in the region and transport via water. Alluvial deposits have been identified in the study area, probably from ancient rivers that were present a long time ago.

4.5 COLLUVIAL MATERIALS

Colluvial soils comprising gravels, sands, silts and clays are the products of in situ weathering of the various types of bedrock transported by water and gravity over short distances downslope. These soils occur everywhere except on the alluvium and areas of outcropping rock and are generally too thin (typically less than 3 m) to be shown on the geological map.

5 IDENTIFICATION OF POTENTIAL FATAL FLAWS

The pre-feasibility report by the Council for Geoscience reported that one major and several smaller faults appear to intersect the dam centre line on the left flank. Based on that, the first task of this feasibility geotechnical investigation was to determine whether any fatal flaws exist that would render the site unsuitable for the construction of a dam.

The main purpose of the seismic refraction survey was to locate zones of low seismic velocity that are usually associated with deep weathering along fault zones. Such a zone was found along the lower half of the left flank (between Ch 200 m and Ch 320 m on the seismic profile in **Figure A5**).

Boreholes BL3 and BL4 were drilled to investigate the above zone of low velocity material and the inclined BL3 encountered highly weathered material to a vertical depth of about 22 m, and moderately weathered rock extends beyond the maximum vertical depth of the hole (30 m). The presence of moderately weathered rock extending to over 30 m depth over a 40 m wide zone along the fault zone and the average excavation depth of over 16 m along the length of the centre line have major cost implications for the construction of a concrete dam, but cannot be considered a fatal flaw.

The lack of sufficient quantities of impervious material might be considered a fatal flaw for the construction of a cored embankment dam.

6 CONSTRUCTION MATERIALS INVESTIGATION

Various types of dams have to be considered, and therefore the available quantities and properties of different types of construction material had to be investigated. The aim of the construction materials investigation was to locate the volumes of materials as shown in **Table 3.** These quantities do not take into account that during site investigations it is normal practice to prove twice the volume of material required. The quantities below apply to the D2 site and since the pre-feasibility studies showed the costs for the two alternative sites to be comparable, it was assumed that the quantities for the D3 site will be similar.

Table 3: Approximate Volumes of Construction Material Required

Material Types	Rollcrete (m^3)	Zoned Earthfill (m^3)	CFR (m^3)	CCR (m^3)	ACR (m^3)	BCR (m^3)
Strong rock	270 000	76 000	380 000	450 000	380 000	360 000
Soft rock			374 000	450 000	320 000	330 000
Filters						
Semi-pervious		1 052 000				
Clay		160 000		160 000		
Asphalt					50 000	
Bentonite						60 000

Note: CFT: Concrete faced rockfill, CCR: Clay core rockfill, ACR: Asphalt core rockfill, BCR: Bentonite core rockfill

The DWA specification for embankment materials is given in **Table 4**.

Table 4: DWA Specification for Embankment Materials

PROPERTY	EMBANKMENT ZONES		
	Impervious	Semi-pervious	Pervious
Clay content (%)	10-30	<25	<20
PI (%)	10-30	<10	<5
LL (%)	25-60	<25	<20
LS (%)	6-14	<5	<2
MDD (kg/m^3)	1350-1700	1600-1850	1700-2000
OMC (%)	12-25	10-15	8-12
Cohesion (kPa)	20-30	10-15	<10
Friction angle (^{0})	20-30	30-35	>35
Permeability (m/sec)	$1x10^{-8}$	$1x10^{-7}$	$1x10^{-5}$

Ideally, the borrow areas must be located below the full supply level (FSL) within the dam basin. The first areas to be investigated were the spillway approach area and other potential borrow areas within the D2 dam basin. When only limited quantities of pervious and semi-pervious materials were found there, the investigation was extended to the horseshoe area on the right side of the river, a short distance downstream of the D2 dam site. This area yielded limited quantities of mainly sandy material, and the search for impervious material was shifted to the D3 dam basin and the surrounding hills where red soils occur on the surface.

When sufficient quantities of impervious material could not be proved, additional investigations were conducted in the horseshoe area in an attempt to locate relatively clean sand for use in a sand/bentonite core.

Since it was not possible to prove sufficient quantities of pervious and semi-pervious earthfill, the rock quarry was expanded to prove sufficient material for a rockfill dam with an upstream concrete face.

6.1 EARTH FILL MATERIALS

6.1.1 Test Pits

Test pits were dug in:

- The spillway approach area where a large excavation will be required (see **Figure A3**);
- The D2 dam basin (see **Figure A3**);
- The horseshoe area that is located downstream of the D2 dam site on the right flank (see **Figure A3**) and
- The D3 dam basin (see **Figure A4**).

In the spillway approach area, information was also obtained from the cored boreholes, while some test pits were dug outside the D3 basin to investigate the red soils visible from the road. Ground water was not encountered in any of the test pits, except for TP26 and TP29 in the D2 dam basin. The detailed test pit logs are provided in **Annexure B,** and a summary of the results is presented in **Table 5**.

Table 5: Summary of Soil Profiles in Potential Borrow Areas

AREA	TEST PIT ID	TOPSOIL (Typically gravelly silty sand with plant roots)	PERVIOUS AND SEMI-PERVIOUS (Typically gravelly silty sand or clayey sand)	IMPERVIOUS (Typically gravelly sandy silty clay)	COMMENTS
Spillway Approach	TP13	0.0-0.3	0.3-1.4		Digging stopped – material hard
	TP14	0.0-0.4	0.4-1.4		Digging stopped – material hard
	TP15	0.0-0.4	0.4-2.6		Digging stopped – material hard
	TP16	0.0-0.4	0.4-2.2		Digging stopped – material hard
	BQ1	0.0-0.4	0.4-20.6		Cored borehole
	BQ3	0.0-0.1	0.1-3.6		Cored borehole
	BQ4	0.0-0.4	0.4-6.0		Cored borehole

Table 5 Continued: Summary of Soil Profiles in Potential Borrow Areas

AREA	TEST PIT ID	TOPSOIL (Typically gravelly silty sand with plant roots)	PERVIOUS AND SEMI-PERVIOUS (Typically gravelly silty sand or clayey sand)	IMPERVIOUS (Typically gravelly sandy silty clay)	COMMENTS
Horseshoe area	TP17	0.0-0.7	0.7-2.2		Digging stopped – material hard
	TP18	0.0-0.5	0.5-4.2		Maximum reach
	TP19	0.0-0.7	0.7-2.7		Digging stopped – material hard
Horseshoe area	TP20	0.0-0.6	0.6-4.1		Digging stopped – material hard
	TP21	0.0-0.4	0.4-1.3	1.3-4.0	Maximum reach
	TP22	0.0-0.4	0.4-3.4		Digging stopped – material hard
	TP23	0.0-0.4	0.4-2.8		Digging stopped – material hard
	TP24	0.0-0.4	0.4-3.4		Maximum reach
	TPH01	0.0-0.6	0.6-4.1		Maximum reach
	TPH02	0.0-9.5	0.5-2.0		Digging stopped – material hard
	TPH03	0.0-0.6	0.6-3.5		Digging stopped – material hard
	TPH04	0.0-0.5	0.5-2.7		Digging stopped – material hard
	TPH05	0.0-0.6	0.6-3.2		Digging stopped – material hard
	TPH06	0.0-0.4	0.4-3.1		Digging stopped – material hard
D2 Basin	TP25	0.0-0.4	0.5-3.4		Digging stopped – material hard
	TP26	0.0-0.4	0.4-1.4		Collapse due to water
	TP27		0.0-4.0		Maximum reach
	TP28		0.0-2.5		Collapse due to water
	RP29	0.0-0.3	0.3-2.0		Refusal
	TP30	0.0-0.3	0.3-2.3		Refusal
	TP31	0.0-0.5	0.5-2.7		Digging stopped – material hard
	TP32	0.0-0.4	0.4-1.5		Digging stopped – material hard
	TP33	0.0-0.4	0.4-1.6		Digging stopped – material hard
D3 Basin	TP34	0.0-0.3	0.3-1.4		Digging stopped – material hard
	TP35	0.0-0.2	0.2-1.6		Digging stopped – material hard
	TP36	0.0-0.4	0.4-1.0	1.0-2.5	Digging stopped – material hard
	TP37	0.0-0.3	0.3-3.8	0.3-3.3	Digging stopped – material hard
	TP38	0.0-0.3	0.5-1.9	0.3-1.5	Digging stopped – material hard

Table 5 Continued: Summary of Soil Profiles in Potential Borrow Areas

AREA	TEST PIT ID	TOPSOIL (Typically gravelly silty sand with plant roots)	PERVIOUS AND SEMI-PERVIOUS (Typically gravelly silty sand or clayey sand)	IMPERVIOUS (Typically gravelly sandy silty clay)	COMMENTS
	TP39	0.0-0.5	2.2-3.0	0.5-2.2	Digging stopped – material hard
	TP40	0.0-0.5	0.5-2.2	2.2-2.9	Digging stopped – material hard
	TP41	0.0-0.3	0.3-2.0		Digging stopped – material hard
	TP42	0.0-0.3	0.3-0.7	0.7-4.0	Maximum reach
	TP43	0.0-0.4	2.0-2.3	0.4-2.0	Digging stopped – material hard
	TP44	0.0-0.6	0.6-3.0		Digging stopped – material hard
	TP45	0.0-0.5	0.5-3.6		Digging stopped – material hard
	TP46	0.0-1.0	1.0-3.3		Digging stopped – material hard
	TP47	0.0-0.4	0.4-3.0		Digging stopped – material hard
	TP48	0.0-0.08	0.8-2.5		Stopped due to slow progress
	TP49	0.0-0.3	0.3-3.9		Maximum reach
	TP50	0.0-0.2	0.2-3.9		Maximum reach
	TP51	0.0-0.3	0.3-2.6		Digging stopped – material hard
	TP52	0.0-0.2	0.2-1.1		Digging stopped – material hard
	TP53	0.0-0.6	0.6-2.1		Digging stopped – material hard
	TP54	0.0-0.5	0.5-1.7		Digging stopped – material hard
	TP56	0.0-0.4	0.4-3.0		Digging stopped – material hard
	TP46a	0.0-0.6	0.6-2.7		Digging stopped – material hard
	TP47a	0.0-0.6	0.6-3.4		Digging stopped – material hard
	TP48a	0.0-0.25	0.25-1.3		Digging stopped – material hard
	TP49a	0.0-0.7	0.7-2.5		Digging stopped – material hard
	TP50a	0.0-0.3	0.3-1.8		Digging stopped – material hard
	TP51a	0.0-0.5	0.5-1.9		Digging stopped – material hard
	TP52a	0.0-0.8	0.8-2.0		Digging stopped – material hard
	TP54a	0.0-0.4	0.4-0.7		Digging stopped – material hard

6.1.2 Laboratory Test Results

The laboratory testing carried out on the samples obtained from the test pits included:

- 17 foundation indicator tests consisting of grading analysis, including determination of the clay fraction and the determination of the Atterberg limits,
- 12 Proctor compaction tests,
- Five permeability tests with three tests at 90% Standard Proctor and two at 100%,
- A tri-axial test.

The foundation indicator testing and Proctor compaction tests were carried out by Soilco Materials Investigations (Pty) Ltd in Durban, and the permeability and tri-axial tests were carried out by Soillab (Pty) Ltd in Pretoria.

6.1.2.1 Grading and Atterberg Test Results

The results of the Grading and Atterberg tests are summarised in **Table 6**, and the detailed results are provided in **Annexure C**. 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 and the laboratory results (in italics) for grain sizes are given in the fourth column of **Table 6**.

The main component, which is sand in all 17 samples, had been correctly identified in 14 cases, while it had been described as clay for the other three samples. The secondary component (generally small percentages of either silt or clay) was correctly described in 9 cases. Since the main purpose of the soils investigation was to locate impervious clayey soils, the incorrect description of sandy material as clayey material, means that it is unlikely that potentially suitable impervious clayey samples had not been selected for laboratory testing. The initial description was therefore on the conservative (safe) side.

In the horseshoe area, where the TPH pits were dug for locating sandy soil (for mixing with bentonite), the main soil component was correctly described as sand in all 6 cases. However, the presence of clay was identified in only two of the samples, whereas laboratory tests show that all the samples contain clay (13-42%) that renders them unsuitable for mixing with bentonite.

Table 6: Summary of Laboratory Test Results

<table>
<tr><th rowspan="3">Test Pit No.</th><th rowspan="3">Depth (m)</th><th rowspan="3">Site</th><th>Soil Profile Description</th><th colspan="5">Soil Properties</th></tr>
<tr><th rowspan="2">(Lab result in italics)</th><th colspan="3">Atterberg Limits (%)</th><th rowspan="2">USCS</th><th rowspan="2">PE</th></tr>
<tr><th>LL</th><th>PI</th><th>LS</th></tr>
<tr><td>TP27</td><td>1.0-3.1</td><td>D2 Basin</td><td>Silty sand
Silty (6) clayey (12) sand (82)</td><td>CBD</td><td>SP</td><td>1.5</td><td>SM</td><td>Low</td></tr>
</table>

Table 6 Continued: Summary of Laboratory Test Results

Test Pit No.	Depth (m)	Site	Soil Profile Description (*Lab result in italics*)	Soil Properties: Atterberg Limits (%) LL	PI	LS	USCS	PE
TP27	3.1-4.0	D2 Basin	Gravely silty sand *Silty (5) clayey (7) sand (88)*	CBD	SP	0.5	SM	Low
TP30	0.3-1.8	D2 Basin	Silty sand *Clayey (10) silty (23) sand (67)*	**29***	**11**	**5.5**	**CL**	**Low**
TP30	1.8-2.3	D2 Basin	Gravely silty sand *Clayey (3) silty (22) sand (75)*	**21**	**12**	**6**	**SC**	**Low**
TP37	1.1-3.3	D3 Basin	Sandy silty clay *Clayey (7) silty (16) sand (77)*	CBD	NP	0	SM	Low
TP38	0.3-1.5	D3 Basin	Sandy silty clay *Clayey (14) silty (14) sand (72)*	23	6	3	SM	Low
TP42	0.7-2.4	D3 Basin	Sandy silty clay *Clayey (22) silty (23) sand (55)*	**27**	**14**	**7**	**CL**	**Low**
TP42	2.4-4.0	D3 Basin	Sandy silty clay *Clayey (2) silty (39) sand (59)*	**36**	**12**	**6**	**SM**	**Low**
TP43	0.4-2.0	D3 Basin	Sandy silty clay *Clayey (11) silty (20) sand (69)*	**27**	**9**	**4.5**	**SC**	**Low**
TP HO1	0.6 - 2.4	Horseshoe Area	Gravelly clayey silty sand *Clayey (10) silty (14) sand (78)*	24	8	4	SC	Low
TP HO1	2.4 - 4.1	Horseshoe Area	Gravelly silty sand *Clayey (13) silty (16) sand (71)*	**28**	**13**	**6.5**	**CL**	**Low**
TP HO2	0.5 - 2.0	Horseshoe Area	Clayey silty sand *Silty (12) clayey (38) sand (50)*	**35**	**15**	**7.5**	**CL**	**Low**
TP HO5	0.6 - 2.1	Horseshoe Area	Gravely silty sand *Silty (11) clayey (26) sand (63)*	**33**	**13**	**6.5**	**CL**	**Low**
TP HO5	2.1 - 3.2	Horseshoe Area	Gravelly silty sand *Silty (11) clayey (19) sand (70)*	**27**	**12**	**6**	**CL**	**Low**
TP HO6	0.4 - 1.2	Horseshoe Area	Clayey silty sand *Silty (11) clayey (42) sand (47)*	**37**	**17**	**8.5**	**CH**	**Low**
TP HO6	1.2 - 2.1	Horseshoe Area	Gravelly silty sand *Silty (11) clayey (26) sand (63)*	**32**	**12**	**6**	**CL**	**Low**
Legend	LL		=	Liquid Limit				
	PI		=	Plasticity Index				
	LS		=	Linear Shrinkage				
	PE		=	Potential Expansiveness				
	USCS		=	Unified Soil Classification System				
	SP		=	Not Plastic				

*** Bold text shows suitability as impervious core**

6.1.2.2 Proctor Compaction Test Results

Proctor compaction tests were done on 14 samples that are considered representative of the material encountered in the investigated areas. The test results are summarised in **Table 7** below. The detailed results are included in **Annexure C**.

Table 7: Summary of Proctor Compaction Results

Proctor Compaction Test Results						
Inspection Pit No.	**Depth (m)**	**Site**	**Soil Profile Description**	**USCS**	**Proctor Compaction**	
			(Lab result in italics)		**MDD (kg/m^3)**	**OMC (%)**
TP27	1.0-3.1	D2 Basin	Silty sand *Silty (6) clayey (12) sand (82)*	SM	1975	11
TP27	3.1-4.0	D2 Basin	Gravely silty sand *Silty (5) clayey (7) sand (88)*	SM	2021	9.8
TP33	1.6-2.4	D2 Basin	Sandy silty clay *Clayey (21) silty (21) sand (58)*	SC	1750	15.2
TP42	0.7-2.4	D3 Basin	Sandy silty clay *Clayey (22) silty (23) sand (55)*	CL	1675	18
TP42	2.4-4.0	D3 Basin	Sandy silty clay *Clayey (2) silty (39) sand (59)*	CL	1693	18.4
TPH01	0.6 - 2.4	Horseshoe Area	Gravelly silty sand *Clayey (13) silty (16) sand (71)*	SC	2039	9.1
TPH01	2.4 - 4.1	Horseshoe Area	Gravelly silty sand *Clayey (13) silty (16) sand (71)*	CL	1915	13.7
TPH02	0.5 - 2.0	Horseshoe Area	Clayey silty sand *Silty (12) clayey (38) sand (50)*	CL	1730	16.2
TPH05	0.6 - 2.1	Horseshoe Area	Gravely silty sand *Silty (11) clayey (26) sand (63)*	CL	1869	13.2
TPH05	2.1 - 3.2	Horseshoe Area	Gravelly silty sand *Silty (11) clayey (19) sand (70)*	CL	1927	11.2
TPH06	0.4 - 1.2	Horseshoe Area	Clayey silty sand *Silty (11) clayey (42) sand (47)*	CH	1626	19.6
TPH06	1.2 - 2.1	Horseshoe Area	Gravelly silty sand *Silty (11) clayey (26) sand (63)*	CL	1704	14
Legend:	USCS	=	Unified Soil Classification System			
	MDD	=	Maximum Dry Density			
	OMC	=	Optimum Moisture Content			

6.1.2.3 Permeability Test Results

Clayey soils from TP33, TP42 and TPH06 were sampled to determine the coefficient of permeability and thus the suitability of this material for use as impervious core. Permeability values of slower than 1 x 10^{-7} m/s are considered suitable for impervious core material. The results of the permeability testing that were carried out at Proctor density are tabulated in **Table 8**.

Table 8: Summary of Permeability Test Results

Sample No.	Test Pit ID.	Depth (m)	USCS	Max Proctor Dry Density (kg/m^3)	Optimum Moisture Content (%)	Head of Water (cm)	Coefficient of Permeability (m/s)
L/NO9472	TP33	1.6-2.4	SC	1750	15.2	102	$1.155x10^{-7}$
L/NO9475	TP42	0.7-2.4	CL	1675	18	falling	$1.650x10^{-10}$
L/NO9476	TP42	2.4-4.0	CL	1693	18.4	falling	$2.553x10^{-10}$
F957	TPH06	0.4-1.2	CH	1615	19.8	falling	$4.7x10^{-11}$
F958	TPH06	1.2-2.1	CL	1692	14.8	falling	$7.9x10^{-12}$

6.1.2.4 Consolidated Undrained Triaxial Test Results

One sample of sandy silty clay (CL) from TP42 (0.7-2.4 m) was taken for triaxial testing. The testing was done on a remoulded sample compacted to 90% Proctor at Optimum Moisture Content and then saturated. A summary of results for the tri-axial test carried out is provided in **Table 9**.

Table 9: Summary of CU Triaxial Test Results

Sample ID	TP ID	Depth (m)	USCS	Moisture State	Rate of Compression (mm/m)	Sample State	Cohesion (kPa)	Angle of Internal Friction (degrees)
L/No. 9475	TP42	0.7-2.4	CL	saturated	0.59	Remoulded to 90% Proctor MDD and OMC	0	26.6

The above results appear to be incorrect, since a CL material containing 22% clay must have some cohesion.

6.1.3 Available Quantities of Earth Fill Material

6.1.3.1 Gugamela Site (D3)

Impervious Core Material Only **10** of the 30 test pits excavated in the D3 basin encountered soils described as sandy silty **clay.** Laboratory tests were conducted on samples from four of these test pits, and only two test pits (TP 42 and TP 43) gave material that is marginally suitable as impervious core. The findings indicate that the D3 basin is underlain predominantly by gravelly silty sand that is considered unsuitable for use as impervious core. Based on the previous DWA investigations and the description of the soil profiles, this result was not expected since the DWA reported large quantities of suitable material.

The total volume of impervious material that could be proved in the D3 basin is about 96 000 m^3. This volume represents a shortfall of 224 000 m^3 from the 2 x 160 000 m^3 required for the clay core of an embankment dam (see **Table 3**).

Semi-Pervious Embankment Material

Thirty test pits dug in the D3 basin and surroundings showed variable sandy and gravelly material and occasional clayey material with an average thickness of about 2.7 m that is suitable as semi-pervious material. The estimated volume from this area is **400 000 m^3**. This represents a shortfall of 1,7 million m^3 from the 2 x 1 052 000 m^3 needed for an embankment dam (**Table 3**).

6.1.3.2 Ncwabeni Site (D2)

Impervious Core Material

In the D2 dam basin, only **two** test pits encountered marginally suitable core material based on laboratory tests and the application of the Department Water Affairs (DWA) specification. The volume of impervious material available from this area is about 3 000 m^3.

Fourteen test pits were dug in the horseshoe area outside the D2 dam basin, of which only four showed marginally suitable impervious material. The total available volume is about 5 000 m^3.

Should it be considered to use all the impervious material that could be proved in the D3 basin, the D2 basin and the horseshoe (about 96 000 m^3, 3 000 m^3 and 5 000 m^3 respectively) then at least another 216 000 m^3 of impervious material will have to be proved to obtain 2 x 160 000 m^3 for the clay core of an embankment dam (see **Table 3**).

Semi-Pervious Embankment Material

Four test pits and three boreholes in the **spillway approach area** showed clayey sand (completely weathered granite to depths of 3.6-20.6 m). This material was not tested, but is considered suitable as semi-pervious embankment material. Excavation of the material might be difficult due to the steep topography and variable thickness of material. The estimated volume from this area is **8 000 m^3**.

Nine test pits dug in the **D2 basin** showed an average thickness of 2.7 m of sand and gravelly sand that is considered suitable for use as semi-pervious embankment material. The estimated volume from this area is **105 000 m^3.**

Fourteen test pits dug in the **horseshoe area** showed variable sandy material to an average depth of about 2.7 m. Most of these materials may be suitable for use as semi-pervious embankment material. The estimated volume from this area is **280 000 m^3**.

Thirty test pits dug in the **D3 basin** and surroundings showed variable sandy and gravelly material and occasional clayey material with an average thickness of about 2.7 m that might be suitable as semi-pervious material. The estimated volume from this area is **400 000 m^3**.

If the total proven volumes (**793 000m^3**) of semi-pervious materials from the above three areas are added together, it will still be necessary to prove another 1 200 000m^3 of material to reach the 2 x 1 052 000 m^3 needed for an embankment dam (**Table 3**).

6.2 ROCK MATERIALS

6.2.1 Cored Boreholes

Seventeen boreholes (BQ7 – BQ20 and BQ22 – BQ24) were drilled in an area proposed for a rockfill quarry (see **Figure A6**). The borehole results summarised in **Table 10** may differ slightly from the detailed borehole logs contained in Annexure D because ranges of degrees of weathering (e.g. highly to moderately) as given in the logs have been changed to single descriptors, and depths have been rounded to the nearest 0.2 m. This was done to facilitate drafting of the geological longitudinal sections as shown on **Figure A7.** The core photographs are contained in Annexure E.

Table 10: Summary of Borehole Logs in the proposed quarry area in the D2 Basin

Borehole number	Topsoil and residual soil (Depth - m)	Completely Weathered (Depth - m)	Highly weathered (Depth - m)	Moderately weathered (Depth - m)	Slightly weathered (Depth - m)	Un-weathered (Depth - m)
BQ 7	0.0-3.6	3.6-4.4	4.4-6.4	6.4-7.2		7.2-12.36
BQ 8	0.0-1.0	1.0-6.6	6.6-12.6	12.6-13.8	13.8-18.4	18.4-28.51
BQ 9	0.0-2.0	2.0-26.4		26.4-27.6	27.6-28.0	28.0-29.91
BQ 10	0.0-0.6	0.6-10.0	10.0-17.4			17.4-29.70
BQ 11	0.0-3.4	3.4-6.0	6.0-7.2	7.2-8.0	8.0-18.0	18.0-25.48
BQ 12	0.0-0.80			2.4-3.2	0.80-2.4	3.2-25.41
BQ 13	0.0-3.0	3.0-3.6	3.6-4.4		4.4-6.2	6.2-29.94
BQ 14	0.0-3.8	3.8-5.6	5.6-9.4		9.4-10.0	10.0-30.09
BQ 15	0.0-0.8	0.8-7.6	7.6-8.6 10.0-10.4 10.6-12.0		8.6-10.0 10.4-10.6 12.0-13.0	13.0-18.25
BQ 16	0.0-0.2	0.2-1.4	1.4-2.8		2.8-3.6	3.6-9.32
BQ 17	0.0-3.2	3.2-4.2	4.2-8.4	8.4-9.6		9.6-16.33
BQ 18	0.0-1.2	1.4	4.0-19.0	19.0-21.4	21.4-27.01	-
BQ 19	0.0-2.2				2.2-10.97	
BQ 20	0.0-3.0		3.0-5.6		5.6-30.3	
BQ22	0.0-1.6	1.6-5.0	5.0-16.4	16.4-18.6	18.6-27.29	
BQ23	0.0-1.0		1.0-20.6	22.8-27.2	20.6-22.8 27.2-28.4	28.4-30.05
BQ24	0.0-0.6	0.6-2.8	2.8-18.6	18.6-20.4	20.4-21.6	21.6-27.35

6.2.2 Laboratory Testing

Two samples, one of which comprised highly weathered cores and the other completely weathered cores from various boreholes were submitted for laboratory testing. The samples were crushed to the minus 25 mm size and subjected to Grading, Atterberg, Proctor and Triaxial testing. For the Proctor and Triaxial tests, the minus 0.85 mm fraction was used, while for the Triaxial test, a mixture of the two weathered samples was used. The results are shown in **Table 11**.

Table 11: Summary of laboratory tests on crushed rock cores

Sample Origin	Completely weathered cores	Highly weathered cores
Laboratory Number	3493	3494
Grading description	Clayey (1%), silty (5%) sand (96%)	Clayey (1%), silty (3%) sand (96%)
LL (%)	29	Non plastic
PI (%)	6	Non plastic
LS (%)	3	1.5
PE	Low	Low
Proctor MDD (kg/m^3)	1897	1883
OMC (%)	6.5	6.4
Cohesion - total (kPa)	128.9	
Friction angle - total (degrees)	26.1	
Cohesion - effective (kPa)	107.8	
Friction angle - effective (degrees)	33.8	

The hard rockfill was not tested, but shear strength values of c = 0 and phi = 42 degrees can be assumed. The laboratory results for the soft rockfill gave values for c (effective) = 107 kPa and phi (effective) of 33.8 degrees. However, from the grading it appears that the rock, when broken down, comprises almost completely of sand-size particles. For this material, a cohesion of zero and friction angle of 38 degrees would seem more appropriate.

6.2.3 Available Volumes of Rockfill Material

Moderately weathered to unweathered granite occurs everywhere in the D2 basin, but in most areas it is covered by variable thicknesses of topsoil, residual granite soil, completely weathered granite and highly weathered granite.

The moderately to unweathered granite is suitable as conventional hard rockfill, and large quantities of this material are available once the overburden has been removed. The hard rock occurs mainly in and alongside the course of the Ncwabeni River, while the valley sides are underlain by variable thicknesses of soft rock.

Due to the presence of slightly weathered and unweathered granite in the riverbed, a quarry area adjacent to the Ncwabeni River was investigated. Development of a quarry in this area will require diversion of the river or extraction of material in wet conditions from the low-lying areas.

To avoid wastage of large volumes of overburden, consideration was given to the use of soft rockfill (completely and highly weathered granite) in parts (about 50%) of a rockfill embankment while conventional hard rockfill is to be used in critical zones (upstream shell and outside zones). The ratio of soft to hard material available from the proposed quarry depends on the volume requirements, i.e. a small quantity of hard rock can be obtained from areas close to the river, resulting in the production of a relatively small volume of soft rock, whereas a large volume of hard rock will require larger volumes of soft excavation.

A temporary diversion weir can be constructed in the Ncwabeni River, just north of BQ17, and a diversion canal can be excavated to pass between BQ13 and BQ14 and end in the river channel between BQ8 and BQ7 (see **Figure A6**).

Quarry area A (with a surface area of about 30 000 m^2) between BQ17, BQ11, BQ18 and a point about 100 m to the northwest of BQ20, can be developed to produce about 80 000 m^3 of soft rockfill.

If a bulking factor is taken into account, about 40 000 m^3 of hard rockfill can be obtained for every metre depth excavated below the soft zone. If the excavation is taken to a depth of 6m below river bed, 240 000 m^3 of hard rockfill is obtained here. (Based on the topography and the established weathering patterns, it appears possible to extend Quarry A towards the east, in which case the above volumes can be increased by about 20% to obtain a total volume of 288 000 m^3.) Provision will have to be made for the pumping of groundwater seeping into the quarry.

Quarry area B (area of about 20 000 m^2) around BQ9, BQ8, BQ10 and BQ21 can be quarried to produce about 250 000 m^3 of soft rockfill. If the excavation is taken to a depth of 6 m below river bed level, then about 400 000 m^3 of hard rockfill can be obtained. (Based on the topography and the established weathering patterns, it appears possible to extend Quarry B towards the south-west, in which case the volume of rockfill can be increased to 600 000 m^3.)

The drilling investigation proved about 330 000 m^3 of soft rockfill and 640 000 m^3 of hard rockfill, the total of 970 000 m^3, which is 130% of the required 754 000 m^3 shown in to **Table 3.** Revised quantities for hard rockfill (406 974 m^3) and soft rockfill (239 675 m^3) obtained from the Module 1:Technical Study, are slightly less than the provisional volumes assumed at the time of the materials investigation. Based on the revised figures, about 150% of the required volume had therefore been proved.

The practice to prove 200% of the required construction material volumes during a site investigation is an arbitrary safety factor that had been established for the investigation of soils borrrow areas to take account of the variability of soil properties over short distances. In these proposed rockfill quarries, the ratio of "soft" to "hard" rockfill may not be exactly as predicted from the borehole results, and might result in the production of slightly smaller or larger volumes of "soft" rock. However, the total volume of rockfill will not change significantly. A proven volume of 150% of the required volume is therefore considered to offer a sufficiently large margin of safety. However, an additional margin of safety is available by considering lateral extension of the quarries, and in so doing, obtains an additional volume of about 240 m^3 of hard rockfill.

7 GEOTECHNICAL INVESTIGATIONS

Geotechnical investigations were conducted at the following locations:

- along the D2 dam centre line;
- along the proposed alignment for a side spillway on the upper left flank;
- at two alternative weir sites on the Mzimkulu River; and
- at two alternative pumping station sites. Investigations aimed to provide founding levels for the various structures, including various types of dams.

Packer permeability tests were undertaken to determine the need for grouting below the dam.

7.1 DAM CENTRE LINE

The position for the dam centre line was selected based on the topography, particularly along the ridge on the left flank. The first step was to clear the bush along the centre line to allow access for the seismic investigation crew and the core drilling rigs. Due to the thick bush and dense undergrowth, the most economical method of bush clearing was by means of a 20-ton tracked excavator. While working along the centre line, the excavator was also used to dig test pits.

7.1.1 Seismic Refraction Survey

Due to the very steep topography of the right flank, the seismic survey could only be conducted along the 400 m length of the left flank (see seismic profile on **Figure A5** and the detailed report in **Annexure F)**.

A low seismic velocity surface layer is typically 2-3 m thick, and may be interpreted as colluvial materials or completely weathered granite. This is followed by material with a velocity in excess of 1000 m/s, that may be interpreted as a progression of completely to moderately weathered granite. This layer grades fairly sharply into seismic basement, comprising fractured, slightly weathered to unweathered granite where the velocity exceeds 2 500 m/s. However, this transition depth is highly variable from less than 5 m near the river and at the top of the left flank to more than 15 m in the middle parts of the left flank. These deep low velocity zones might represent fractured and deeply weathered zones along fault lines.

The seismic refraction surveys indicated the positions of anomalous areas where cored boreholes could be located, and assisted with the interpretation of geological conditions between boreholes.

7.1.2 Test Pits

Eight test pits were excavated along the proposed dam centre line on the left flank of the Ncwabeni River (see **Figure A3**). Five of these were excavated in depressions where it was expected that fault or shear zones may have caused deeper weathering that might be associated with thicker horizons of soft soil near ground surface. However, this was not the case. The soil profiles are summarised in **Table 12**.

Refusal depths of the 20-ton excavator were surprisingly shallow, indicating very stiff colluvial or residual soils at depths of 1.3-2.5 m. A ferruginised layer was encountered in TP02 and a hard zone of mylonitised material (possibly representing a re-cemented fault zone) was encountered in TP05.

Table 12: Summary of Soil Profiles along the Dam Centreline

Test Pit ID	TOPSOIL	COLLUVIUM	RESIDUAL GRANITE		GRANITE	Comments
	Loose slightly gravelly silty SAND (m)	Loose to medium dense silty SAND (m)	Very dense gravelly silty SAND (m)	Scattered cobbles in a loose slightly gravelly silty SAND (m)	Completely to highly weathered, soft to medium hard rock (m)	
TP01	0.0-0.7	0.7-2.2			>2.2	Refusal
TP02	0.0-0.5	0.5-2.5[1]				Refusal
TP03	0.0-0.5	0.5-1.4	1.4-1.6		>1.6	Refusal
TP04	0.0-0.5	0.5-1.6	1.6-1.8		>1.8	Refusal
TP05	0.0-0.5		0.5-1.3[3]		>1.3[4]	Refusal
TP06	0.0-1.3[2]		1.3-1.7		>1.7	Refusal
TP07	0.0-0.5			0.5-2.1		Digging stopped – material hard
TP08	0.0-0.5			0.5-1.5		Digging stopped – material hard

LEGEND:
1 Ferruginised dense to very dense gravelly sand.
2 From 0.5 m – 1.3 m ferruginised nodules in a loose gravelly silty sand.
3 Mylonitised scattered cobbles in a medium dense to dense gravelly sand.
4 Mylonitised as above.

7.1.3 Cored Boreholes

Ten boreholes were drilled along the length of the centreline with two holes (BR1 and BR2) on the right flank and eight holes (BL1-BL8) on the left flank. The borehole results are summarised in **Table 13** and may differ slightly from the detailed borehole logs in **Annexure D** because ranges of degrees of weathering (e.g. highly to moderately) given in the logs have been changed to single descriptors, and depths have been rounded to the nearest 0.2 m to facilitate drafting of the geological longitudinal section shown on **Figure A8**. The core photographs are contained in **Annexure E.**

The measured water levels on the left flank are erratic, with levels measured varying between 3 m in BL1 to 21.1 m in BL3, with the remainder of the boreholes along the centreline encountering water levels between these two extremes. On the right flank, water levels were encountered between 11.95 m and 18.3 m down the boreholes. Due to the very low permeability of the materials, the actual phreatic line might be lower (due to the removal of the drill string) or higher (as result of drilling water remaining in the hole).

The water pressure test (Lugeon test) results in **Annexure D** and summarised in **Table 14** indicate minimal water losses in most of the boreholes, with the exception of local higher water losses in some of the holes where poor core recovery and lower Rock Quality Designation (RQD) values were recorded.

Table 13: Summary of Borehole Logs along the Dam Centre line

Borehole Number (inclination)	Material type/ degree of weathering						
	Topsoil and residual granite	Completely weathered rock (m)	Highly weathered rock (m)	Moderately weathered rock (m)	Slightly weathered rock (m)	Un-weathered rock (m)	Water level (29/11/2011) (depth in m)
BL 1 (60)	0.0 –2.8			2.84-4.0	4.0-6.2	6.2-30.19	3.03
BL 2 (90)	0.0-2	2.0-4.8	4.8-6.2	6.2-11.0	11.0-14.20	14.20-18.0	16.6
BL 3 (60)	0.0-3.6	3.6-4.0	4.0-23.6	23.6-35.4			21.1
BL 4 (90)	0.0-0.8	0.8-1.8	1.8-14.6	14.6-16.2	16.2-18.2	18.2-21.61	16.7
BL 5 (60)	0.0-3.0	3.0-4.4	4.4-17.4		17.4-20.0	20.0-24.24	14.9
BL 6 (90)	0.0-2.0	2.0-3.4	3.4-11.2	14.4-14.8	11.2-14.4	14.8-23.8	15.4
BL 7 (90)	0.0-11.0		11.0-13.8 15.4-16.0		13.8-15.4	16.0-30.0	16.35
BL 8 (90)	0.0-1.0	1.0-3.2	3.2-14.8	14.8-15.0		15.0-20.59	15.5
BR 1 (90)	0.0-1.6	1.6-2.8 6.2-7.8	2.8-6.2 7.8-14.4 18.4-19.0		14.4-18.4	19.0-29.95	18.3
BR 2 (60)	0.0-2.0		2.0-19.6	19.6-26.6	26.6-30.2		11.95

Table 14: Summary of water test results

BH No.	LUGEON VALUES			
	>10 (actual lugeons)	10 - 4	4 - 1	<1
	Depth in m	Depth in m	Depth in m	Depth in m
BL1	6.00 – 9.03(31)			1.50 – 6.00 9.03 – 30.19
BL2		9.00 – 12.50		1.50 – 9.00
BL3	21.00 – 24.12 (11)	25.50 – 35.40		1.50 – 21.14 24.00 – 25.62
BL4				1.50 – 20.91
BL5		12.00 – 18.24	6.00 – 8.79	1.5 – 5.91 9.00 – 11.92 18.00 – 24.24
BL6	1.55 – 2.90 (165)			1.90 – 23.70
BL7		15.00 – 17.81		1.55 – 15.12 18.00 – 30.00
BL8				1.50 – 20.59
BR1		6.00 – 8.95 18.00 – 20.95	12.00 – 14.95	1.50 – 5.95 9.00 – 11.95 15.00 – 17.95 21.00 – 29.95
BR2	12.00 – 14.79 (23) 18.00 – 21.29 (13)			1.50 – 11.80 15.00 – 17.77 21.00 – 30.29

7.1.4 Founding Conditions for Alternative Dam Types

The shell of an earth embankment is typically founded on material with low organic content and low compressibility, and with shear strength similar to the dam wall material. This means that the ± 0.5 m thick layer of topsoil and zones of loose colluvium have to be removed and that founding will take place on medium-dense colluvium or on very dense residual granite at depths of 0.5-1.3 m. At some places it might be necessary to compact the upper layer of colluvium in order to obtain suitable founding conditions.

The shell of a rockfill embankment is typically founded on material with low organic content and low compressibility, and with shear strength similar to the dam wall material. This means that a 2.0-4.8 m thick layer of topsoil and colluvium has to be removed and that founding will take place on completely or highly weathered granite. In the case of a concrete faced rockfill (CFR) dam, founding of the plinth will require material with either low permeability or good groutability. Lugeon water pressure tests show that this depth corresponds with the depth of excavation for the rockfill shells.

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. Water pressure (Lugeon) tests in boreholes confirmed that the water loss is minimal below the depth at which the earthfill shells are to be founded.

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.

The recommended excavation depths at borehole positions are listed in **Table 15** and shown on **Figure A8** in **Annexure A**.

Table 15: Excavation Depths for Various Dam Components

BH No.	SECTION	SHELL		CUT-OFF		CONCRETE (m)
		EARTHFILL (m)	ROCKFILL (m)	EARTH CORE (m)	PLINTH (m)	
BL1	River section	2.2	2.84	2.84	2.84	4
BL2		2	4.84	2.0	4.84	11
BL3		2.2	4.05	3.5	4	26+
BL4	Left flank	1.8	1.8	1.8	1.8	16.2
BL5		2.2	4.4	3.0	4.4	17.4
BL6		2	3.3	2.1	3.3	14.8
BL7		1.8	4.0	1.7	11.0	13.8
BL8		2	3.3	2.2	3.3	15
BR1	Right flank	1.5	2.84	1.5	2.84	14.35
BR2		2	2.1	2.1	2.1	26.6

The layers of weathered rock below the recommended core trench depth contain joints that are generally tight, but five of the boreholes contain sections with Lugeon values of between 4 and 10, while four of the holes contain sections with Lugeon values of above 10. In the zone 6-24 m below ground level, Lugeon values of above 4 have been recorded. Curtain grouting will thus have to be done below the dam foundation to depths of 20 m on the flanks and about 40 m in the river section.

It must be noted that the above recommendations apply to the dam reference line that roughly corresponds with the alignment of a central earth core. However, the plinth for an upstream faced rockfill dam will be located a considerable distance upstream of the dam reference line, and for the design of this type of dam, additional geotechnical investigations will have to be conducted to determine the founding levels and grouting requirements.

7.2 SPILLWAY AND RISING MAIN

For an embankment dam alternative, a side spillway with chute leading to the Mzimkulu River is proposed. This alignment was investigated by means of a seismic refraction survey, test pits and cored boreholes.

It was assumed that the rising main will be located below or adjacent to the spillway chute and that the geotechnical information along the spillway line would apply.

7.2.1 Seismic Refraction Survey

A seismic refraction traverse was conducted along the upper 200 m length of the proposed spillway chute (see seismic profile on **Figure A5** and the detailed report in **Annexure F**). The last part to the Mzimkulu River was too steep for access.

A low seismic velocity surface layer is typically 2-3 m thick and may be interpreted as colluvial or alluvial materials or completely weathered granite. This is followed by material with a velocity in excess of 1000 m/s that may be interpreted as a progression of completely to moderately weathered granite. This layer grades fairly sharply into seismic basement comprising fractured, slightly weathered to unweathered granite where the velocity exceeds 2 500 m/s. However, this transition depth is highly variable. Deep low velocity zones might represent fractured and deeply weathered zones along fault lines.

The seismic refraction surveys indicated the positions of anomalous areas where cored boreholes must be located, and assisted with the interpretation of geological conditions between boreholes.

7.2.2 Test Pits

Four test pits were excavated along the spillway line to determine the founding conditions (see **Figure A3**). The profiles of the test pits are summarized in **Table 16**. No groundwater was encountered in any of the test pits excavated along the spillway. TP 11 encountered alluvial sand to a depth of more than 3 m indicating the presence of an old stream channel.

Table 16: Summary of Soil Profiles along the Spillway

Test Pit ID	TOPSOIL	COLLUVIUM	RESIDUAL GRANITE	Comments
	Loose slightly gravelly silty SAND (m)	Loose slightly gravelly silty SAND (m)	Scattered cobbles in a loose/loose to medium dense slightly gravelly silty SAND (m)	
TP09	0.0-0.5		0.5-1.2	Digging stopped – material hard
TP10	0.0-0.4		0.4-1.1	Digging stopped – material hard
TP11	0.0-0.5	0.5-2.9	2.9-3.0	Digging stopped – material hard
TP12	0.0-0.3	0.3-1.8	1.8-2.2	Digging stopped – material hard

7.2.3 Cored Boreholes

Five boreholes (BS1 – BS5) were drilled along the length of the spillway line, while two other boreholes (BL7 and BQ1) are in close proximity to the overspill section. The borehole results as summarised in **Table 17** may differ slightly from the detailed borehole logs presented in **Annexure D** because the ranges of degrees of weathering (e.g. highly to moderately) in the logs have been changed to single descriptors, and depths have been rounded to the nearest 0.2 m to facilitate drafting of the geological longitudinal sections shown on **Figure A9.** The core photographs are contained in **Annexure E.**

Table 17: Summary of Borehole Logs Along the Spillway line

Borehole Number	Material Type/ Degree of Weathering					
	Topsoil and residual granite (m)	Completely weathered rock (m)	Highly weathered rock (m)	Moderately weathered rock (m)	Slightly weathered rock (m)	Unweathered rock (m)
BS 1	0.0-7.6	7.6-8.0	8.0-11.0 11.8-12.6 15.8-18.6	11.0-11.80 12.6-15.8	18.6-20.2	20.2-25.43
BS 2	0.0-1.6	1.6-2.2	2.2-10.0 11.4-11.6 19.8-20.8	10.0-11.4	11.6-19.8	20.8-25.87
BS 3	0.0-1.8	1.8-4.4	4.4-9.6 10.8-12.6	9.6-10.8	12.6-20.0	20.0-25.87
BS4			0.6-9.0	9.0-12.8	12.8-15.3	
BS5		1.8-2.8	2.8-12.6 13.8-14.0		12.6-13.8	14.0-20.1
BL7	0.0-11.0	11.0-11.6	11.6-13.8 15.4-16.0		13.8-15.4	16.0-30.0
BQ1	0.0-19.6	19.6-20.6		20.6-21.6	21.6-23.0	23.0-28.43
BQ3	0.0-2.0	2.0-3.6	3.6-10.4		10.4- 12.4	12.4-17.23
BQ4	0.0-5.2	5.2-6.6	6.6-12.6 13.4-13.8	13.8-14.8	12.6-13.4	14.8-21.27

7.2.4 Founding Conditions

The spillway overflow section and return channel will have to be concrete-lined to prevent erosion of the deeply weathered granite. The concrete lining can be founded on highly weathered granite but must be designed to span across local zones of completely weathered rock that are likely to occur at or below the recommended founding levels. The recommended founding levels at the borehole positions are shown in **Table 18** and **Figure A9**.

For the rising main, the excavatibility can be derived from the excavator refusal depths (boundary between soft and hard excavation) given in **Table 16**. Blasting will be required from depths of 1-3 m along the upper 100 m length and from about 0.5 m along the lower steep section.

Table 18: Founding Depths for Spillway

BOREHOLE NUMBER	FOUNDING DEPTH (m)
BS1	8
BS2	2.5
BS3	4.5
BS4	2
BS5	4
BL7	11
BQ1	20.6

The estimated geotechnical properties of the rock at the above founding levels are as follows.

- E-modulus: 50 MPa
- Cohesion: 100 kPa
- Friction angle: 26 degrees.

At the end of the chute in the area next to the Mzimkulu River, a stilling basin can be founded on good quality granite with the following assumed properties:

- E-modulus: 15 GPa
- Cohesion: 1 500 kPa
- Friction angle: 40 degrees.

In the area of the spillway control structure at Boreholes BS1, BL7 and BQ1, highly weathered granite (founding quality rock) occurs at depths of 8 m, 11 m and 20.6 m respectively. These holes are located close to the proposed overspill section and these conditions may pose serious founding problems. It is recommended that additional investigations be conducted during the design stage to select the most favourable position for the spillway control structure and to determine the required founding levels.

7.3 ALTERNATIVE WEIR SITES

At the time of the drilling, the position of the abstraction weir in the Mzimkulu River had not been finalised, and still had to be determined on the basis of hydraulic and geotechnical considerations. Various sites from pre-feasibility study and initial hydraulic model runs had been identified. Further testing with the physical hydraulic model had narrowed the selection down to two sites for geological investigations. At the upper site, good quality rock outcrops are visible on the left bank and in the left part of the river channel, but a thick deposit of alluvium was anticipated on the right flank. About 200 m downstream of the upper site, the river valley narrows and becomes more symmetrical with rock outcrops across the full width of the river.

7.3.1 Cored Boreholes

Two boreholes (BW1 – BW2) were drilled on the right flank of the upper weir site, and one borehole (BW3) was drilled on the right bank about 200 m downstream of BW1 (see **Figure A5**). The borehole results summarised in **Table 19** may differ slightly from the detailed borehole logs in **Annexure D** because ranges of degrees of weathering (e.g. highly to moderately) in the logs have been changed to single descriptors, and depths have been rounded to the nearest 0.2 m to facilitate drafting of the geological longitudinal section as shown on **Figure A10**. The core photographs are contained in **Annexure E**.

Table 19: Summary of Borehole Logs at the Weir Sites

Borehole Number	Material type/ degree of weathering					
	Topsoil, alluvium and residual granite (m)	Completely weathered rock (m)	Highly weathered rock (m)	Moderately weathered rock (m)	Slightly weathered rock (m)	Unweathered rock (m)
BW1	0.0-16.4		16.4-17.6		17.6-23.4	
BW2	0.0-10.8				10.8-16.0	16.0-20.4
BW3	0.0-3.6					3.6-11.93

7.3.2 Founding Conditions

At the upper weir site, good quality rock is visible on the left flank and left part of the river channel. However, on the right bank the thickness of unconsolidated alluvial deposits varies between 10.8 m close to the river to 16.4 m further away.

At the lower weir site, there are sound granite outcrops on the left bank and BW3 on the right bank shows good quality founding rock at about 4 m depth. However, the flat-lying area to the right of BW3 is likely to be underlain by alluvium with an estimated thickness of 5-7 m.

Founding of the weir must be on slightly weathered or unweathered granite, for which the following geotechnical properties apply:

- E-modulus: 15 GPa

- Cohesion: 1 500 kPa
- Friction angle: 40 degrees.

7.4 PUMP STATION

At the time of the drilling, the position of the pumping station had not been finalised, and would depend on the position of the weir and desilting works.

7.4.1 Cored Boreholes

Boreholes BP1 and BP3 were drilled on the left bank of the Mzimkulu River to investigate founding conditions for alternative pump station positions (see **Figure A5**). The borehole results summarised in **Table 20** may differ slightly from the detailed borehole logs in **Annexure D** because ranges of degrees of weathering (e.g. highly to moderately) in the logs were changed to single descriptors, and depths have been rounded to the nearest 0.2 m to facilitate drafting of the geological columnar sections as shown on **Figure A11**. The core photographs are contained in **Annexure E.**

Table 20: Summary of Borehole Logs at the Pump Station Sites

Borehole Number	Material type/ degree of weathering					
	Topsoil, alluvium and residual granite (m)	Completely weathered rock (m)	Highly weathered rock (m)	Moderately weathered rock (m)	Slightly weathered rock (m)	Unweathered rock (m)
BP1	0.0-8.6				8.6-10.0	10.0-15.49
BP3	0.0-1.0				1.0-3.0	3.0-7.03

7.4.2 Founding Conditions

At the upper position for a pump station, the alluvium is 8.6 m thick, while it is only 1 m thick at the lower site. The selection of the most suitable site will depend on the depth required for the pump well.

Founding of the pump station must take place on slightly weathered or unweathered granite, for which the following geotechnical properties apply:

- E-modulus: 15 GPa
- Cohesion: 1 500 kPa
- Friction angle: 40 degrees.

8 RESERVOIR SLOPE STABILITY

Based on a 1 m interval contour plan of the dam basin, 15 areas with slopes higher than 20 m and steeper than 30 degrees were identified (see **Figure A12**). Average slope angles for these areas vary between 30 degrees and 42 degrees, with local steeper sections where there are rock outcrops. The maximum height of these slopes is 30 m. Areas occupied by these slopes vary in size from 1 000-3 000 m^2 and the maximum volume of slide material from a potentially unstable area is about 30 000 m^3.

As no access was available to the steep areas around the proposed dam basin, test pitting was not possible without occurring significant expense and environmental damage. As such test pitting was not conducted for the purposes of reservoir slope stability.

The circular failure charts of Hoek and Bray (1977) were used for soil (completely to highly weathered granite) slopes with angles of 30 and 50 degrees and a height of 30 m, and the properties of the completely and highly weathered granite were tested for use as soft rockfill. These properties are as follows:

- Unit weight: 1916 kg/m3
- Effective cohesion: 107 kPa
- Effective friction angle: 34 degrees.

For the worst (rapid drawdown) water conditions, Factors of Safety of between 1.7 and 2.2 were obtained for slope angles of 30 degrees and 50 degrees respectively.

In the case of rock slopes, the potential failure planes are stress relief joints that dip at smaller angles than the surface topography. These joints are generally wavy and not continuous over long distances. The estimated shear strength parameters of these joints are C = 150 kPa and phi = 35 degrees. The estimated Factor of Safety against failure of a stress relief joint under full hydrostatic pressure and dipping at 25 degrees is 2.3.

Thus, the risk of failure is low for the actual slopes with maximum slope angles of 42 degrees and heights of less than 30 m. In the extremely unlikely event that all 15 areas fail simultaneously, the volume of mobilised material will be very small (less than 0.3% of the reservoir volume) and the effect on the water level will be minimal.

9 CONCLUSIONS

9.1 GEOLOGY

All components of the project (the dam centre line, the spillway line, weir sites, pump station site, quarry site and most of the borrow areas) are underlain by granitic bedrock of the Oribi Gorge Suite. This rock is weathered to various degrees and depths across the area, with slightly weathered to unweathered rock on surface or at shallow depth near the rivers and deeper weathering in the higher lying areas.

The bedrock is generally covered by layers of colluvial clayey silty sand in the higher lying areas, while there are well-developed layers of alluvial silt, sand and gravel closer to the rivers.

9.2 POTENTIAL FATAL FLAW

At Site D2, no fatal flaw that would rule out the construction of all types of dams was identified. Very deep founding levels might rule out a concrete dam, while the lack of sufficient impervious soils might be considered a fatal flaw for the construction of a clay core embankment dam.

9.3 CONSTRUCTION MATERIALS

For an earth embankment, sufficient quantities of neither semi-pervious earthfill nor impervious core material could be sourced within the D2 basin or elsewhere in the area.

A rockfill embankment comprising of “soft” and “hard” rockfill zones can be constructed from completely and highly weathered granite (soft rockfill) and moderately weathered to unweathered granite (hard rockfill) that can be obtained from the proposed quarry.

As an impervious zone, an upstream concrete face might be considered. Good quality concrete aggregate can be obtained from the bottom part of the rockfill quarry.

9.4 FOUNDING CONDITIONS

9.4.1 Dam

Very deep weathering at Borehole BL3 that is also associated with a zone of low seismic velocity material between the middle of the left flank and the river indicates the possible occurrence of a wide fault zone in this area. This has a major effect on the founding level of a concrete structure, but since the weathered material contains enough clay, it is relatively impervious and suitable as foundation for an embankment dam.

Founding levels for a concrete gravity (RCC) dam are so deep that its cost will far exceed those for an embankment dam or composite dam.

If a composite dam is to be considered, an 80 m long concrete section can be located between Boreholes BL2 and the top of the right flank, in which case the average excavation depth will be about 8 m. The embankment on the left side will have to be supported by a long flank wall, the founding levels of which have not been investigated, but are expected to be about 10 m deep.

Founding conditions for earth-fill or rock-fill embankments are favourable. The shells can be founded at depths of 1.5-3.5 m, while the core or plinths can be founded at depths of 1.6-5 m. The completely and highly weathered rock on which the core/plinth will be founded is impervious, but the deeper-lying jointed rock will have to be curtain grouted to depths of between 20 m on the upper flanks and about 40 m in the river section.

It must be noted that the above recommendations apply to the dam reference line that roughly corresponds with the alignment of a central earth core. However, the plinth for an upstream faced rockfill dam will be located a considerable distance upstream of the dam reference line, and for the design of this type of dam, additional geotechnical investigations will have to be conducted to determine the founding levels and grouting requirements.

9.4.2 Spillway and Rising Main

Founding conditions for the proposed side spillway on the upper left flank are not favourable because of very deep, intense and irregular weathering, resulting in 8-20 m of residual granite (soil) in the area of the control structure. It might be necessary to relocate the position of the spillway control structure. The proposed return channel will need to be concrete-lined and will be founded on highly weathered (soft rock) granite with local zones of completely weathered granite (very soft rock).

It is recommended that additional investigations be conducted during the design stage to select the most favourable position for the spillway control structure and to determine the required founding levels.

Blasting will be required for construction of the pipe trench for the rising main if it has to be constructed along or below the spillway chute.

9.4.3 Weir

At the lower weir site, there are sound granite outcrops on the left bank and BW3 on the right bank shows good quality founding rock at about 4 m depth. However, the flat-lying area to the right of BW3 is likely to be underlain by alluvium with an estimated thickness of 5-7 m.

9.4.4 Pump Station

At the upper position for a pump station, the alluvium is 8.6 m thick, while at the lower site it is only 1 m thick. At these depths, founding will be on good quality slightly weathered granite. The selection of the most suitable site will depend on the depth required for the pump well.

9.5 RESERVOIR SLOPE STABILITY

In the unlikely event of a slope failure along the rim of the reservoir, the volume of mobilised material will be very small (less than 0.3% of the reservoir volume) and the effect on the dam level will be minimal.

10 REFERENCES

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