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DIRECTORATE: OPTIONS ANALYSIS

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT

GEOTECHNICAL INVESTIGATIONS:

NTABELANGA, SOMABADI AND THABENG DAM SITES

OCTOBER 2014

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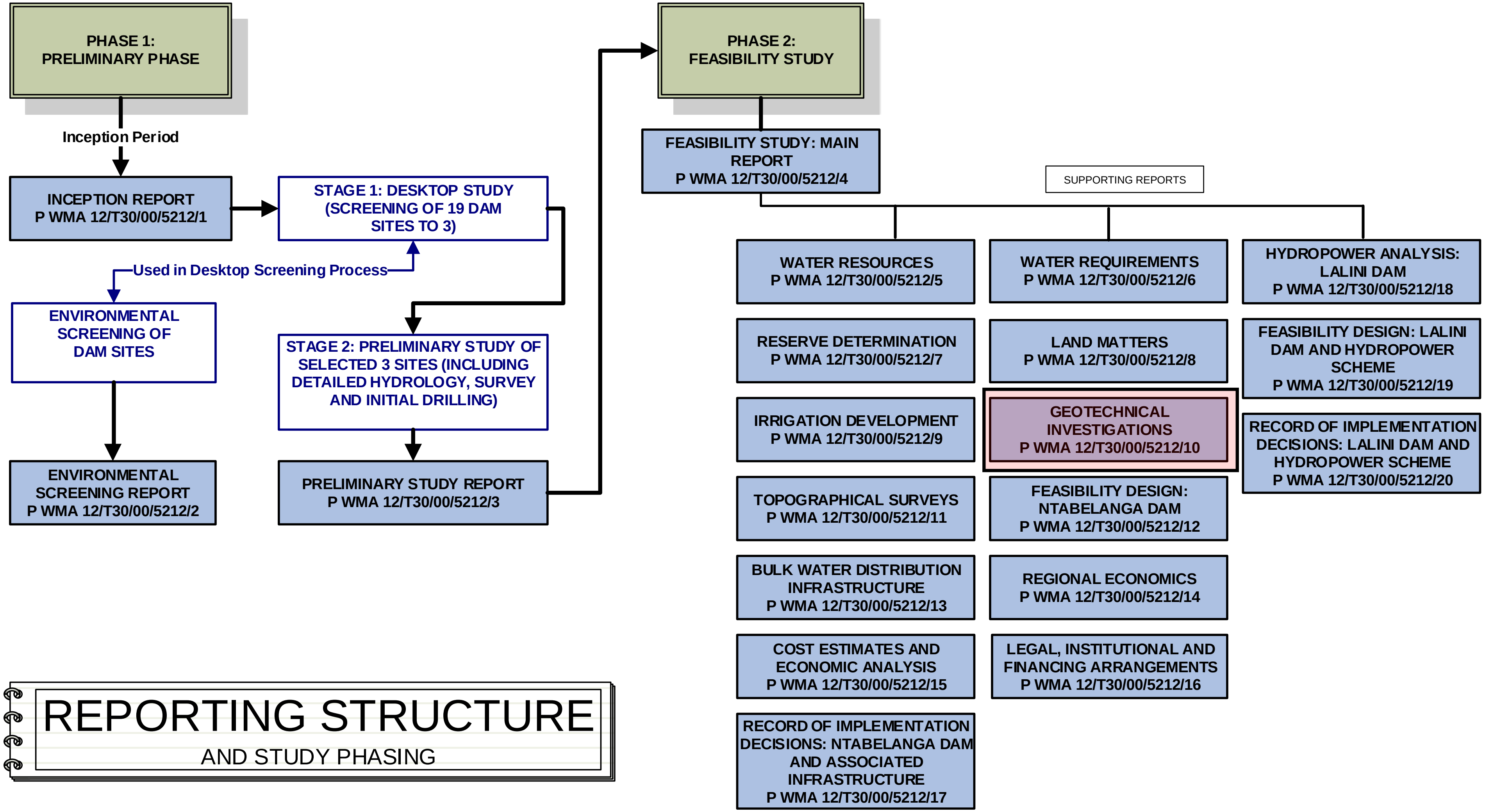

REPORTING STRUCTURE
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Note on Departmental Name Change:

In 2014, the Department of Water Affairs changed its name to the Department of Water and Sanitation, which happened during the course of this study. In some cases this was after some of the study reports had been finalized. The reader should therefore kindly note that references to the Department of Water Affairs and the Department of Water and Sanitation herein should be considered to be one and the same.

Note on Spelling of Laleni:

The settlement named Laleni on maps issued by the Surveyor General is locally known as Lalini and both names therefore refer to the same settlement.

EXECUTIVE SUMMARY

INTRODUCTION

The two-phase feasibility level dam site geotechnical investigations on the Umzimvubu Water Project commenced during October 2012. From reconnaissance level studies previously undertaken, three potential dam sites were shortlisted, namely the Thabeng, Somabadi and Ntabelanga sites. A two-phase geotechnical investigation approach was adopted.

The Phase 1 geotechnical and site selection feasibility investigations entailed an initial visual appraisal of each of the shortlisted sites considering a number of influence factors, followed by limited rotary core drilling, comprising one borehole on either side of the river at each of the three sites. The factors considered in the selection process included the following:

- *Topography and valley shape;*
- *Accessibility of equipment for investigation and construction purposes;*
- *Geology and founding conditions. This considered the influence of lithology and geological structure on the integrity of the foundation, stability and water tightness;*
- *The availability of construction materials for earthfill, rockfill and concrete dam types within the future impoundment basin of a dam constructed at the sites; and*
- *The effects of a dam constructed at the site on the local environment and infrastructure.*

At each of the three sites, rotary core boreholes were positioned above each river bank on the dam flanks and drilled to depths of about 40 m each. Water pressure tests were carried out in the boreholes, generally at 3 m intervals or such other stage lengths as deemed appropriate.

The visual appraisal and drilling investigation undertaken on the Thabeng Dam site did not identify fatal flaws or other problem areas that could preclude the construction of a dam at the site. The drilling indicated both flanks to be underlain by interlayered sandstone and dolerite. Rock quality was generally good and lugeon values low, indicating a strong, water tight foundation. The site lends itself to the construction of both embankment and concrete dam types. Whilst construction materials appear to be readily available in the overall project area, these were not visibly abundant within the future impoundment basin. In addition upstream structures and infrastructure would be effected by the construction of a dam at this site.

The assessments and drilling undertaken at the Somabadi site did not deduct any fatal flaws. Competent sandstone, with thin subordinate interbeds of siltstone and mudrock generally prevails from about 4m. Water pressure tests indicate low water losses. These conditions will provide good founding for earthfill, rockfill and concrete dam alternatives. Construction materials also appear to be readily available either from within the basin or within close proximity of the dam site. The Phase 1 geotechnical investigations indicated the Somabadi site to be suitable for dam construction.

At the Ntabelanga site both the visual appraisals and the subsequent drilling indicated a potentially good dam site. The drilling results indicated suitable founding conditions on dolerite below depths of between 4m to 6m on the chosen alignment. Water pressure tests gave generally low lugeon values indicating negligible water loss and hence relatively low grout takes. Generally high RQD and low fracture frequency values indicate good quality, competent dolerite. The drilling undertaken did not indicate any fatal flaws at the two positions drilled, in the form of faulting or other geological features that could compromise founding conditions or water-tightness of the foundation. The valley profile and founding conditions encountered appear to be equally suitable for the construction of earthfill, rockfill or concrete dam alternatives. Construction materials for alternative dam types also appear to be readily available within the future impoundment basin.

The Phase 1 investigations concluded that all three sites were suitable for the construction of earth embankment, rockfill or concrete dams. Following comparative suitability assessments the Ntabelanga site was considered to have the most consistent founding conditions, where the

foundation along the major proportion of the dam axis will be in dolerite, whereas at the other two sites there is interlayering and interbedding of different rock types, namely dolerite and sandstone at Thabeng and sandstone, mudrock and siltstone at Somabadi. Construction materials for alternative dam types also appeared to be more readily available within the future impoundment area of the Ntabelanga site. From the results of the Phase 1 assessment, the Ntabelanga site was selected as the preferred dam site for the more detailed Phase 2 Geotechnical Investigation.

The Phase 2 geotechnical investigation focussed on the preferred single dam site, namely the Ntabelanga site. It entailed the undertaking of further drilling, trial pitting, testing, geophysics and the investigation of potential construction materials sources.

The Phase 2 investigation of the Ntabelanga site initially considered two alternative dam alignments approximately 200m apart. The Line 1 or upstream alignment corresponds with that investigated during Phase 1. The centre-line for the Line 2 or downstream alignment coincides with the "nose" of the right abutment hill whence the valley immediately widens into a floodplain. This would allow a shorter side channel spillway discharge chute, would provide slightly easier access and more working space for construction, and would mean that the infrastructure immediately downstream (possibly hydropower house, pumping station, administration buildings, water treatment works) would be located closer to the dam wall but away from any potential backwater flooding effects below the dam. Based upon consideration of the results of the geotechnical investigation and other related factors such as avoidance of structural lineaments, Line 1 was selected as the preferred alignment. Subsequently, consideration was given to a possible third alignment a short distance upstream of the Line 1 alignment. This would require further verification during the detailed design investigations.

The Phase 2 investigations entailed the following:

- *The rotary core drilling of an additional 16 boreholes with a total drilling length of 458.8 m;*
- *The undertaking of 720 m of seismic refraction and 810 m of electrical resistivity surveys. The surveys were conducted parallel to and transverse to the Line 1 alignment;*
- *Trial pitting at the dam site and in identified borrow pits to assess founding conditions for the dam and appurtenant structures and undertake suitability assessments of potential construction material sources; and*
- *Sampling of materials for laboratory testing.*

The geotechnical and materials investigations undertaken during Phase 2 considered the following dam designs:

- *Concrete faced rock-fill (CFRD) dam;*
- *Earth core rock-fill (ECRD) dam;*
- *Earth core earthfill embankment dam (EF);*
- *Roller compacted concrete (RCC) dam; and*
- *Composite Central Bathtub Spillway (CCS)*

The construction materials requirements for the various dam types were calculated according to embankment or wall configuration and cross section. These are:

CFRD:	*1.3 million m³ of rock aggregate*
	100 000 m³ of sand
ECRD:	*1.1 million m³ of rock aggregate*
	260 000 m³ of core
	100 000 m³ of sand
EF:	*65 000 m³ of rock aggregate*
	2.1 million m³ of shell (general shoulder fill)
	50 000 m³ of core
	25 000 m³ of sand

RCC[#]:	*500 000 m³ of rock aggregate*
	200 000 m³ of sand
CCS:	*1 million m³ of rock aggregate*
	20 000 m³ of shell (general shoulder fill)
	200 000 m³ of core
	150 000 m³ of sand

[#] Majority of these quantities constitutes the concrete volume

Competent, hard dolerite rock underlies the middle to upper right flank, generally occurring near to the surface at depths of under 1m or as sporadic surface outcrop. Tests conducted on the core samples indicate high strength rock with a low degree of alteration. These demonstrate that the rock will provide good foundations and will be suitable for both the production of rock fill and concrete aggregate. The reserves of potentially good quality dolerite in the hill to the east and south east of the dam, of which the right flank is a part, are extensive and far in excess of the required quantities for any of the above listed dam alternatives. Drilling indicates that a quarry located on the right flank upstream of the dam and within the basin would yield adequate rock aggregate for the construction of both the dam and the appurtenant concrete structures. The spillway configuration could be designed to duplicate as a quarry.

Extensive sand deposits occur in the Tsitsa River upstream of the dam. The Tsitsa River in the project area generally flows in a relatively incised channel with sand deposits confined to the river channel. Therefore the deposits are relatively narrow and would require selective seasonal exploitation during the dry season. Indications are that in excess of the required volumes of sand for construction purposes for any of the dam alternatives can be acquired from the Tsitsa River within the future impoundment basin.

Reddish brown, clayey hill-wash deposits associated with dolerite occur in relative abundance throughout the project area. These were tested from within the basin and found to be suitable for use as core. Indications are that sufficient reserves of good quality core material will be available in the project area for the construction of an embankment dam.

The shell requirements for the earth embankment dam (EF) option are of the order of 2.1 million m³. Sedimentary rocks comprising mainly mudrock with intercalated sandstone are widely distributed within the basin and were tested for suitability as embankment shell. These materials tended to break down under compaction rendering them insufficiently permeable for use as pervious fill, and only marginally suitable for use as semi-pervious fill. Whilst the latter could be used as embankment fill material, this would mean designing the embankment with very shallow slopes, significantly increasing the cost of the earthworks and hence overall dam costs, above the values used to compare dam types.

Consideration could be given to the investigation of extensive sandstone deposits in the surrounding hills or weathered dolerite, but these occur well outside of the future impoundment basin and the exploitation of the large quantities required would have long haul distances (with significant cost implications) and could have significant environmental impacts. These factors have been allowed for in the rates used in the cost estimates, and significantly increase the cost of an earthfill dam option. The paucity of suitable shell material within the basin is thus viewed as a significant constraint to the construction of an earth embankment dam.

For an embankment dam, including the earthfill and rockfill options, two alternative side-channel spillway alignments on the upper right flank were initially proposed, and a third alternative was proposed on the left flank. All of these options required significant excavations to be undertaken and the investigations were structured to assess their suitability for being designed as unlined channels and suitability to duplicate as a rock quarry.

Spillway Option 1 proposes a spillway channel cut into the upper right flank and orientated south to north. The first approximately 330 m of the spillway axis along the hill crest display visible outcrop *and sub-outcrop. This coupled with the drilling results, which indicate competent, near surface dolerite along this section implies good potential as an aggregate source.*

Deeper soils and weathering profiles were apparently down the hill slope further along the spillway axis. The transported and residual soils are particularly deep towards the end of the spillway chute before the outfall into the river. This implies a need to concrete-line the spillway chute to provide protection against excessive erosion. Dolerite outcrop is visible in the river.

Spillway Option 2 proposes an excavation cutting through the hill upstream of the dam in an easterly direction. Dolerite outcrops and sub-outcrops are visible along the first approximately 190 m of the spillway axis and drilling also indicates a shallow rock head profile.

Spillway Option 2 offers better founding conditions along the alignment of the lower chute than spillway Option 1, but the large quantities of rock excavation would be far in excess of the quantities required for the embankment construction and concrete aggregates. This would create the problem of disposal and spoiling of the excess quantities.

Spillway Option 3 proposes a side channel cut into the left flank, perpendicular to the dam axis on the upper left flank, then curving just in front of the downstream dam toe to intersect the river. There is sub-outcrop of sandstone on the upper left flank, but the remainder of the spillway alignment is underlain by a relatively thick mantle of transported and residual soils.

This upper spillway side-channel would be excavated in sandstone. From mid-slope, the chute and stilling basin excavation would be in dolerite. Being located on the steeper left flank, the depth of excavation, particularly along the western face would be deeper than the corresponding spillway option on the right flank, namely spillway Option 1.

The sandstone cores derived from the boreholes failed some durability tests and would not be suitable for rock-fill purposes, and would also not suitable for use as crushed aggregate. Dolerite derived from excavation would be suitable for use as rock-fill and concrete aggregates, although it is doubtful that this option would provide sufficient hard rock dolerite for the project requirements, necessitating an additional hard rock source to supply the shortfall. This would ideally be located on the right flank, where two spillway options are situated.

An RCC or CCS dam alternative would be designed with a central in-channel spillway. The aggregate for the RCC dam and for the spillway of the CCS dam would require a separate rock aggregate source, again ideally located on the mid to upper right flank, where the other spillway options are sited.

The conclusions drawn following these geotechnical and materials investigations were that the founding conditions at the dam site and the materials availability within the impoundment basin would be suitable for the construction of all of the alternative dam types mentioned above.

The exception is the earthfill option for which large quantities of embankment shell material would possibly need to be sourced from outside of the basin, with significant haulage cost and potential environmental impacts. The alternative to this would be a very conservative design for the embankment which would also lead to significantly increase construction cost.

Further site and materials investigations will be required to properly inform the detailed design process.

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LIST OF ACRONYMS AND ABBREVIATIONS

ASGISA-EC	Accelerated and Shared Growth Initiative for South Africa – Eastern Cape
CAPEX	Capital Expenditure
CFRD	Concrete-faced rockfill dam
CMA	Catchment Management Agency
CTC	Cost to Company
CV	Coefficient of Variability
DAFF	Department of Agriculture, Forestry and Fisheries
DBSA	Development Bank of Southern Africa
DEA	Department of Environment Affairs
DM	District Municipality
DME	Department of Minerals and Energy
DoE	Department of Energy
DRDAR	Department of Rural Development and Agrarian Reform
DRDLR	Department of Rural Development and Land Reform
DWA	Department of Water Affairs
DWS	Department of Water and Sanitation
EA	Environmental Authorisation
EAP	Environmental Assessment Practitioner
EC	Eastern Cape
ECRD	Earth core rockfill dam
EF	Earthfill (dam)
EIA	Environmental Impact Assessment
EMP	Environmental Management Plan
EPWP	Expanded Public Works Programme
ESIA	Environmental and Social Impact Assessment
EWR	Environmental Water Requirements
FSL	Full Supply Level
GERCC	Grout enriched RCC
GN	Government Notices
GW	Gigawatt
GWh/a	Gigawatt hour per annum
IAPs	Invasive Alien Plants
IB	Irrigation Board
IFC	International Finance Corporation
IPP	Independent Power Producer
IRR	Internal Rate of Return
IVRCC	Internally vibrated RCC
ISO	International Standards Organisation
kW	Kilowatt
LM	Local Municipality
ℓ/s	Litres per second

MAR_{NAT}	Mean Annual Runoff (Naturalised Flows)
MAR_{PD}	Mean Annual Runoff (Present Day Flows)
MEC	Member of the Executive Council
MIG	Municipal Infrastructure Grant
million m^3	Million cubic metres
MW	Megawatt
NEMA	National Environmental Management Act
NERSA	National Energy Regulator of South Africa
NHRA	National Heritage Resources Act
NOCL	Non-overspill crest level
NWA	National Water Act
NWPR	National Water Policy Review
NWRMS	National Water Resources Management Strategy
O&M	Operations and Maintenance
OPEX	Operational Expenditure
PICC	Presidential Infrastructure Co-Ordinating Committee
PPA	Power Purchase Agreement
PPP	Public Private Partnership
PSC	Project Steering Committee
PSP	Professional Services Provider
RBIG	Regional Bulk Infrastructure Grant
RCC	Roller-compacted concrete
REIPPPP	Renewable Energy Independent Power Producer Procurement Programme
RWI	Regional Water Institution
RWU	Regional Water Utilities
SEZ	Special Economic Zone
SIP	Strategic Integrated Project
SMC	Study Management Committee
SPV	Special Purpose Vehicle
TCTA	Trans Caledon Tunnel Authority
ToR	Terms of Reference
UOS	Use of System
URV	Unit Reference Value
WEF	Water Energy Food
WRYM	Water Resources Yield Model
WSA	Water Services Authority
WSP	Water Services Provider
WTE	Water Trade Entity
WUA	Water User Association

LIST OF UNITS

Description	Standard unit	Description	Standard unit
Elevation	m a.s.l.	Velocity, speed	m/s, km/hr
Height	m	Discharge	m^3/s
Distance	m, km	Mass	kg, tonne
Dimension	mm, m	Force, weight	N
Area	m^2 , ha or km^2	Gradient (V:H)	%
Volume (storage)	m^3	Slope (H:V) or (V:H)	1:5 (H:V) or 5:1 (V:H)
Yield, Mean Annual Runoff	m^3/a	Volt	V
Rotational speed	rpm	Power	W
Head of Water	m	Energy used	kWh
Pressure	Pa	Acceleration	m/s^2
Diameter	mm or m	Density	kg/m^3
Temperature	ºC	Frequency	Hz

1. INTRODUCTION

The Mzimvubu River catchment in the Eastern Cape of South Africa is within one of the poorest and least developed regions of the country. Development of the area to accelerate the social and economic upliftment of the people was therefore identified as one of the priority initiatives of the Eastern Cape Provincial Government.

Harnessing the water resources of the Mzimvubu River, the only major river in the country which is still largely unutilised, is considered by the Eastern Cape Provincial Government as offering one of the best opportunities in the Province to achieve such development. In 2007, a special-purpose vehicle (SPV) called ASGISA-Eastem Cape (Pty) Ltd (ASGISA-EC) was formed in terms of the Companies Act to initiate planning and to facilitate and drive the Mzimvubu River Water Resources Development.

The five pillars on which the Eastern Cape Provincial Government and ASGISA-EC proposed to model the Mzimvubu River Water Resources Development are:

- Afforestation;
- Irrigation;
- Hydropower;
- Water transfer; and
- Tourism.

As a result of this the Department of Water and Sanitation (DWS) commissioned the Mzimvubu Water Project with the overarching aim of developing water resources schemes (dams) that can be multi-purpose reservoirs in order to provide benefits to the surrounding communities and to provide a stimulus for the regional economy, in terms of irrigation, forestry, domestic water supply and the potential for hydropower generation amongst others.

The study commenced in January 2012 and is to be completed by October 2014 in several stages as follows:

- Inception;
- Phase 1 – Preliminary Study; and
- Phase 2 – Feasibility Study.

The purpose of this study was not to repeat or restate the research and analyses undertaken on the several key previous studies described below, but to make use of that information previously collected, to update and add to this information, and to undertake more focussed and detailed investigations and feasibility level analyses on the dam site options that have then been identified as being the most promising and cost beneficial.

Report Nos. P WMA 12/T30/00/5212/2 to 20 describe the feasibility study processes undertaken to select a preferred dam site that would be developed to meet the development goals and social benefits described above.

1.1 Purpose of this Report

This report describes the geotechnical and materials investigation at potential sites for the construction of a multipurpose dam in the Mzimvubu catchment for irrigation, domestic and industrial water supply, as well as hydropower potential.

From reconnaissance level studies previously undertaken, three potential dam sites were shortlisted, namely the Thabeng, Somabadi and Ntabelanga sites, the locations of which are indicated on Figure 1-1. The three sites are located near to the towns of Matatiele / Mount Fletcher and Maclear / Tsolo respectively.

A two-phase geotechnical investigation approach was adopted. The Phase 1 geotechnical investigations entailed an initial visual appraisal of each site followed by the drilling of two rotary core boreholes at each of the three shortlisted sites. The boreholes were positioned above each bank on the dam flanks and drilled to depths of about 40m each.

The results of the Phase 1 investigations are described in Section 2 and the results are presented in Appendix B and Appendix C. From the results of the Phase 1 assessment, the Ntabelanga site was selected as the preferred dam site for the more detailed Phase 2 geotechnical investigation.

The Phase 2 geotechnical investigation involving the feasibility level study of the Ntabelanga site entailed the undertaking of further drilling, trial pitting, testing, geophysics and the investigation of potential construction material sources.

The Phase 2 investigation considered two alternative dam alignments, annotated Line 1 (upstream) and Line 2 (downstream) on Figure D3 in Appendix D. The investigation entailed the following:

- The rotary core drilling of an additional 16 boreholes with a total drilling length of 458.81m;
- The undertaking of 720m of seismic refraction and 810m of electrical resistivity surveys. The surveys were conducted parallel to and transverse to the Line 1 alignment;
- Trial pitting at the dam site and in identified borrow pits to assess founding conditions for the dam and appurtenant structures and undertake suitability assessments of potential construction material sources; and
- Sampling of materials for laboratory testing.

The geotechnical and materials investigations undertaken during Phase 2 considered the following dam designs:

- Roller compacted concrete (RCC) dam;
- Concrete faced rock-fill (CFRD) dam;
- Earth core rock-fill (ECRD) dam; and
- Earth embankment dam.

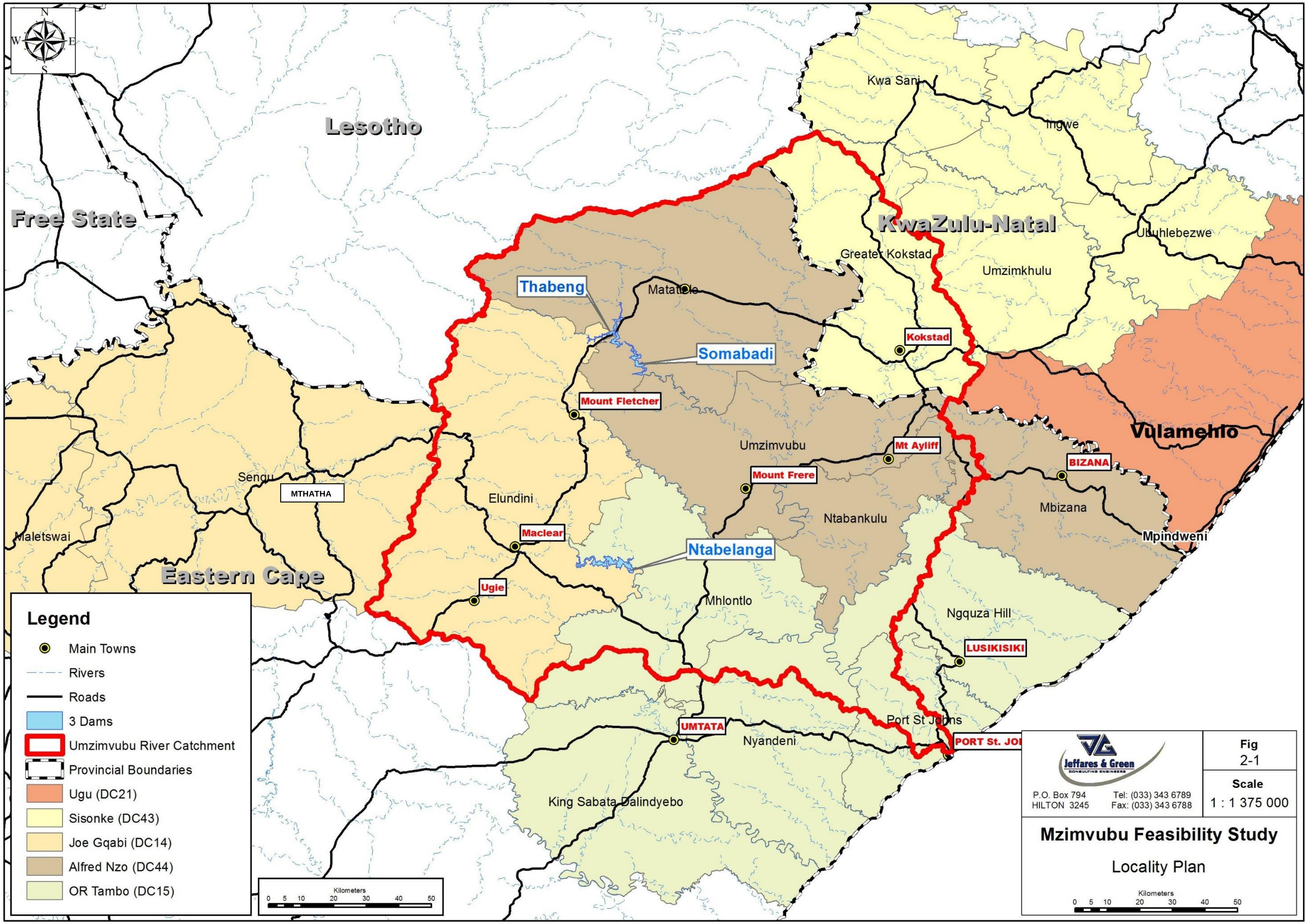


Figure 1-1: Locality Map of the Three Dam Sites in the Mzimvubu Catchment Area

2. PHASE 1 INVESTIGATIONS

2.1 Dam Site Field Reconnaissance

The dam site assessment field reconnaissance sheets are presented in Appendix B. As the preferred dam type was not defined at the time, the assessments considered a number of alternatives, namely earthfill, rockfill and concrete dam types. All three sites lend themselves to either of these options.

2.1.1 Thabeng Dam Site Assessment

The visual appraisal and subsequent drilling investigations undertaken did not detect any fatal flaws that would preclude the construction of a dam at this site.

The valley sides are steep and whilst this is conducive to a good area to storage ratio it renders mechanical access difficult. Visually, the site appears to offer good founding and cut-off conditions, which based upon observable surface features, will mainly by in dolerite on the right flank and a combination of dolerite and sedimentary rocks on the left flank. Dolerite outcrops across the river section. The nature of the rock and jointing across the major proportion of the dam axis appears to be conducive to watertight conditions, although more variability may be experienced on the left flank due to the intercalation and interbedding of various rock types.

From the initial assessment undertaken, no feasibly exploitable sources of good quality rock aggregate were identified in the impoundment basin upstream of the dam, but hard rock dolerite occurs in abundance a relatively short distance downstream of the site. Similarly no sources of core were identified in the basin, but again appear to occur downstream of the dam, in the form of weathered mudrock or red-brown colluvial clayey soils of doleritic derivation. As such areas would not be inundated by a dam constructed at this site they would incur more stringent environmental and rehabilitation constraints and restrictions. Had this site been selected for detailed Phase 2 investigations, mitigation of these constraints would have been required in the form of further detailed reconnaissance and investigations to identify potential sources of rock aggregate and core within the future impoundment basin. It is quite possible that given more time for reconnaissance, such materials would be located within the basin.

Sand deposits appear to be plentiful in the upstream river reaches near to Kinira Drift, but high flow at the time of the assessment prevented an accurate evaluation of quality or quantity.

A dam at this site would inundate some major infrastructural developments, including roads, pipelines, the upstream Kinira Drift river crossing and a water treatment works under construction at Kinira Drift.

The site is suitable for a number of alternative dam types, including earth embankment, rockfill or concrete. A concern at this stage is the apparent paucity of suitable sources of rock aggregate and core within the future impoundment basin.

2.1.2 Somabadi Dam Site Assessment

No fatal flaws were identified and there is good founding on sandstone. Whilst the pronounced bedding of the sandstone could lead to increased grout takes, the results of the water pressure tests carried out during the drilling investigation indicate low water losses.

The site occupies a steep U-shaped valley, which is particularly steep on the right flank. The steepness of the valley sides makes for difficult mechanical access along the dam axis.

Construction materials appear to occur in abundance within relatively short haulage distances of the site. Mudrock from within the basin could be considered for use as core. Clayey colluvial doleritic soils also occur to the north-east, but the area is outside the impoundment basin. Nevertheless the deposits appear to be extensive and to be of good quality for use as core.

Whilst no site was specifically identified as an aggregate source, dolerite occurs in relative abundance in the basin. Dolerite occupies a saddle in the left flank which could possibly be excavated to form an off-channel spillway and at the same time duplicate as a rock quarry. This would require further investigation in the form of rotary core drilling.

Whilst difficult to accurately assess due to high river flow at the time of the assessment, sand appears to be plentiful upstream of the dam site.

Inundation of roads and cultivation would occur in the basin. Whilst not accurately assessed as the full supply level had not been defined, such inundation of fields and roads appears to be fairly extensive.

The valley profile and founding conditions would suit a number of dam type alternatives, including earth embankment, rockfill and concrete.

2.1.3 Ntabelanga Dam Site Assessment

The assessment and subsequent drilling did not identify fatal flaws in the context of geological or geotechnical constraints.

The site occupies a steep sided, U-shaped valley with a low length to height ratio. There is good founding on dolerite, with visible outcrop in the river section and mid to upper right flank. The left flank is soil covered with indications of underlying dolerite bedrock. The top of the left flank is underlain by sandstone, which occurs as visible sub-outcrop. The nature of the dolerite and the jointing appears conducive to low seepage losses.

Conversely, the steep valley sides proved difficult to access during the drilling investigation, although access was easier than at the other two sites. The left hand side river bank a few hundred metres upstream of the dam shows minor evidence of sliding. Whilst this could be exacerbated during dam filling, it is localised and of a small scale, so that large scale instability is not considered likely. This would be further assessed during the detailed investigation.

Construction materials appear to be readily available in the basin within relatively short haulage distances. A purple mudrock that occurs in abundance within the impoundment basin could possibly be utilised as core, but appears more suited for use as shell material. A red clayey colluvial soil of doleritic origin occurs a short distance upstream of the dam site, as well as in other areas within the basin. It has good potential for use as core.

There also appear to be potential dolerite rock quarry sites within the basin a relatively short distance upstream of the dam. Alternatively there is extensive dolerite outcrop on the upper right flank and consideration could be given to channelling the spillway through this area and at the same time generating good quality dolerite rock.

Whilst there appear to be extensive sand deposits in the river, the extent of exploitable reserves was difficult to assess due to high flow at the time of the assessment.

The dam would bring about inundation of roads and agriculture in the basin, including an upstream river bridge. The site lends itself to alternative dam types, including earth embankment, rockfill and concrete.

2.2 Rotary Core Drilling Investigation

Following the field reconnaissance and findings thereof, a programme of core drilling was undertaken at each potential dam site in order to identify foundation conditions and to check whether there were any fatal flaws that could eliminate a site as being suitable for dam construction.

The drilling investigation was undertaken by Weppelmann Geotechnical Drilling cc. Two boreholes of approximately 40 m each were drilled vertically on both dam wall centreline flanks at each site. Drilling was by rotary core methods using an N-size, double-tube core barrel fitted with a diamond bit crown. Borehole logs and photographs are presented in Appendix C.

Water pressure tests were carried out in the boreholes, generally at 3 m intervals or such other stage lengths as deemed appropriate. The tests were carried out according to the 2010 South African National Roads Agency Limited (SANRAL) guideline, “Standard Specifications for Subsurface Investigations”. The test results were analysed and interpreted according to Houlsby, 1974.

2.2.1 Thabeng Dam Site

The results of the drilling investigation are summarised in Tables 2-1 and 2-2.

Table 2-1: Borehole T1: Lower Left Flank

Borehole T1: Lower Left Flank 30°29’42.9”S; 28°38’21.6”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 1.44	Cobbles and boulders of sandstone			
1.44 – 9.78	Unweathered, widely jointed, very hard rock, Dolerite with narrow, stained and calcite / chlorite coated joint planes	2 – 5 5 – 8	Total water loss 1	 Void filling
9.78 11.48	Unweathered, medium to widely jointed / thickly bedded, Sandstone with narrow, stained / coated joints and bedding planes.	8 – 11	3	Dilation
11.48 – 22.87	Unweathered, widely jointed, very hard rock, Dolerite with narrow stained / coated joints. Alteration / dislocation zone adjacent to contact aureole.	11 – 14 14 – 17 17 – 20 20 – 23	1 0 0 0	Laminar
22.87 – 37.87	Unweathered, very widely jointed / thickly bedded, hard rock, Sandstone with narrow, coated and healed discontinuity planes. Shaly interlaminations and inclusions.	23 – 26 26 – 29 29 – 32 32 – 35	0 0 0 3	 Dilation
37.87 – 40.01	Unweathered, medium to widely jointed, very hard rock, Dolerite with narrow, stained joints. End of borehole at 40.01m	35 – 40.01	0	

Table 2-2: Borehole T2: Lower Right Flank

Borehole T2: Lower Right Flank 30°29’46.6”S; 28°38’25.5”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.45	Cobbles of dolerite and sandstone			
0.45 – 5.5	Medium weathered becoming slightly weathered, widely jointed / thinly to medium bedded, medium hard to hard rock, Sandstone with wide becoming narrow, stained and gouge filled joints and bedding planes.	3 – 6	0	
5.5 – 15.43	Unweathered, widely jointed, very hard rock, Dolerite with narrow stained and chlorite coated joints.	6 – 9 9 – 12 12 – 15	0 0 0	
15.43 – 15.76	Thin shale interbed			
15.76 – 23.8	Unweathered, widely jointed, very hard rock, Dolerite with narrow stained and chlorite coated joints	15 – 18 18 – 21	0 0	
23.8 – 33.16	Unweathered, widely jointed, medium to thinly bedded, hard rock, Sandstone with narrow, calcite coated joints and bedding planes.	21 – 29 29 – 32	Grouted 5	No test Wash out
33.16 – 34.99	Unweathered, widely jointed / thinly bedded, hard rock, Siltstone with narrow, calcite coated joints and bedding planes.	32 – 35	4	Dilation
34.99 – 36.01	Unweathered, medium jointed / thinly bedded, hard rock, Sandstone with narrow, calcite coated joints and bedding planes.			
36.01 – 39.92	Unweathered, medium to widely jointed, very hard rock, Dolerite with narrow chlorite coated joints. End of borehole at 39.92m.	35 – 39.92	4	Dilation

The two boreholes drilled at the Thabeng site indicate both flanks to be underlain by interlayered sandstone and dolerite. This alternating arrangement of sedimentary and igneous rock is the result of concordant dolerite intrusions parallel to the sedimentary bedding, which have conformed to different bedding plane elevations.

The dolerite intrusion has resulted in peripheral alteration of both rock types, in the form of chilled margins brought about by the “baking” effect of the magma intrusion and subsequent rapid cooling. These aspects appear not to have compromised the integrity of the rock mass in respect of strength and water-tightness. Lugeon values are low and grouting requirements will be minimal.

The drilling undertaken does not indicate any fatal flaws at the two positions drilled, in the form of faulting, intense jointing or other geological features that could compromise the strength or water-tightness of the foundation.

The results of the Phase 1 geotechnical investigations indicated that the site is suitable for the construction of a dam. Whilst suitable for the construction of earthfill, rockfill and concrete dam types, the valley profile and founding conditions are possibly most suited to the construction of a gravity roller-compacted-concrete structure.

2.2.2 Somabadi Dam Site

The results of the drilling investigation are summarised in Tables 2-3 and 2-4.

Table 2-3: Borehole S1: Lower Left Flank

Borehole S1: Lower Left Flank 30°34'58.2"S; 28°41'37.5"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.18	Silty clay and medium gravel			
0.18 – 0.87	Silty clay and coarse gravel			
0.87 – 2.68	Completely to highly weathered very soft to soft rock, Mudstone			
2.68 – 3.6	Moderately weathered, very intensely laminated, highly fracture, medium hard rock, Mudstone with open silt filled joints and bedding planes.			
3.6 – 13.07	Slightly weathered to unweathered, intensely laminated, very slightly fractured, hard rock Sandstone with inter-bedded siltstone and mudstone (40mm interbeds)	5 – 8 8 – 11 11 – 14	0 0 0	
13.07 – 23.2	Slightly weathered to unweathered, intensely laminated, very slightly fractured, hard rock, Sandstone with inter-bedded mudstone lenses and clasts.	14 – 17 17 – 20 20 – 23	0 0 0	
23.2 – 30.29	Slightly weathered to unweathered, intensely laminated, slightly fractured, hard rock, Sandstone with inter-bedded mudstone lenses and clasts	23 – 26 26 – 29	0 0	
30.29 – 31.02	Slightly weathered to unweathered, very intensely laminated, moderately fractured, soft to medium hard rock, Mudstone.	29 – 32	0	
31.02 – 40.2	Slightly weathered to unweathered, very intensely laminated, moderately fractured, soft to medium hard rock, Mudstone. End of borehole at 40.2m.	32 – 35 35 – 40.m	0 0	

Table 2-4: Borehole S2: Lower Right Flank

Borehole S2: Lower Right Flank 30°35’02.2”S; 28°41’35.8”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 2.09	Highly weathered, soft rock, Sandstone			
2.09 – 4.61	Moderately, very thinly bedded, moderately fractured, soft rock, Sandstone with open discontinuities and inter-bedded mudstone.			
4.61 – 9.76	Moderately to slightly weathered, thinly bedded, slightly to moderately fractured, soft rock, Sandstone with inter-bedded mudstone lenses and open discontinuities.	5 – 8m 8 – 11m	1 1	Void filling Void filling
9.76 – 16.32	Slightly weathered, very thinly bedded, slightly fractured, soft rock, Sandstone with inter-bedded mudstone lenses and open, gouge filled discontinuities.	11 – 14m 14 – 17m	1 0	Void filling Void filling
16.32 – 18.93	Dark grey, slightly weathered, very thinly bedded / laminated, moderately fractured, soft rock, Mudstone with narrow, unaltered discontinuities.	17 – 20m	0	Void filling
18.93 – 36.24	Alternating maroon and grey, slightly weathered, very thinly bedded / intensely laminated, highly fractured, soft rock, Mudstone with inter-bedded sandstone and narrow, unaltered discontinuities.	20 – 23m 23 – 26m 26 – 29m 29 – 32m 32 – 35m	0 0 0 0 9	Void filling Void filling Void filling Void filling Dilation
36.24 – 37.58	Slightly weathered, intensely laminated, moderately fractured, soft to medium hard rock, Mudstone	35 – 40.18m	0	
37.58 – 40.18	Slightly weathered, very thinly bedded, slightly fractured, medium hard rock, Sandstone End of borehole at 40.18m.			

The results of the drilling undertaken at the two positions on the Somabadi site did not indicate any fatal flaws. Competent sandstone, with thin subordinate interbeds of siltstone and mudrock generally prevails from about 4 m. Water pressure tests indicate low water losses. These conditions will provide good founding for earthfill, rockfill and concrete dam alternatives.

The Phase 1 geotechnical investigations indicates the Somabadi site to be suitable for dam construction.

2.2.3 Ntabelanga Dam Site

The results of the drilling investigation are summarised in Tables 2-5 and 2-6.

Table 2-5: Borehole N1: Lower Left Flank

Borehole N1: Lower Left Flank 31°06'59.6"S; 28°40'18.3"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 1.91	Residual clayey gravel			
1.91 – 2.86	Completely weathered dolerite			
2.86 – 4.35	Highly weathered, closely jointed, soft rock, Dolerite			
4.35 – 9.3	Moderately weathered, closely jointed, medium hard rock, Dolerite, with wide gouge filled joints	5.96 – 8.98	62	Wash out
9.3 – 39.82	Slightly weathered, closely jointed, hard rock, Dolerite, with narrow, stained / coated joints. Becomes unweathered, medium to widely jointed, very hard rock, Dolerite. Joints narrow to occasionally wide, stained / coated, occasionally gouge filled with peripheral alteration. End of borehole at 39.82m	8.98 – 11.84 11.84 – 14.79 14.79 - 16.71 17 – 20 20.84 – 23.59 23.59 – 26.48 26.48 – 29.45 29.45– 32.81m 32.81 – 35.23 35.23 – 39.82	Aborted 5 0 1 5 0 1 4 7 5	 Dilation Void filling Void filling Dilation Dilation Dilation Turbulent Laminar

Table 2-6: Borehole N2: Lower Right Flank

Borehole N2: Lower Right Flank 31°07'02.0"S; 28°40'22.3"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 3.1	Colluvial gravely sandy clay with boulders			
3.1 – 4.2	Residual gravely clay with boulders			
4.2 – 5.8	Residual sandy clay			
5.8 – 6.39	Slightly weathered, closely jointed, medium hard to hard rock, Dolerite, with wide, gouge filled joints.			
6.39 – 40.03	Unweathered, widely jointed, very hard rock, Dolerite, with narrow, stained / coated joints. End of borehole at 40.03m	8 – 11m 11 – 1`4m 14 – 17m 17 – 20m 20 – 23m 23 – 26m 26 – 29m 29 – 32m 32 – 35m 35 – 38m 38 – 40.03m	0 0 0 0 0 0 0 7 8 14 2	 Dilation Dilation Dilation Dilation Dilation Dilation Dilation

The drilling results indicated suitable founding conditions on dolerite below depths of between 4 m to 6 m. Water pressure tests gave generally low lugeon values indicating negligible water loss and hence relatively low grout takes. Generally high Rock Quality Designation (RQD) and low fracture frequency values indicate good quality, competent dolerite.

The logs indicate a significant number of core recoveries above 100%. This is the result of core “stick-ups”, where the core at the end of a drill run is not broken off at the base of the hole and remains behind following retrieval of the core barrel. Visual evidence of this was apparent in the form of abrasions on the core sides resulting from the reseating of the core barrel over the core “stick-up”. This occurs as a result of a faulty core lifter not breaking off the core at the base of the hole or possibly because of very hard rock.

The drilling undertaken does not indicate any fatal flaws at the two positions drilled, in the form of faulting or other geological features that could compromise founding conditions or water-tightness of the foundation. The valley profile and founding conditions encountered appear to be equally suitable for the construction of earthfill, rockfill or concrete dam alternatives.

2.3 Conclusions Reached on the Phase 1 Investigations

The Phase 1 investigations have concluded that all three sites are suitable for the construction of earth embankment, rockfill or concrete dams.

In order to undertake comparative suitability assessments, a simple ranking matrix has been devised. This is based solely upon geotechnical parameters and does not consider other interrelated disciplines that need to be taken into consideration in optimising the most suitable dam site. The comparative suitability assessment is presented in Table 2-7.

Table 2-7: Comparative Suitability Assessment

Parameters			Dam Site		
			Thabeng	**Somabadi**	**Ntabelanga**
1.	Topography & Valley Profile		4	3	5
2.	Geology	Rock Type	4	3	4
		Structure	4	3	4
3.	Accessibility		2	2	3
4.	Founding conditions	Foundation	4	3	4
		Water-tightness	4	4	4
5.	Construction Materials	Earthfill	3	4	4
		Rockfill	3	4	5
		Concrete	2	3	5
6.	Environment & Infrastructure		1	3	3
Total			31	32	41

The matrix weighting values are on a scale of 1 to 5, where 1 is highly unsuitable and 5 is highly suitable. Based upon the parameters evaluated in Table 7 and subjective opinion based upon geotechnical criteria, the Ntabelanga site attained the highest score of 41 out of a possible 50.

The Ntabelanga site is considered to have the most consistent founding conditions, where the foundation along the major proportion of the dam axis will be in dolerite, whereas at the other two sites there is interlayering and interbedding of different rock types, namely dolerite and sandstone at Thabeng and sandstone, mudrock and siltstone at Somabadi.

Construction materials for alternative dam types are also more readily available within the future impoundment area of the Ntabelanga site.

The Thabeng site rates low on environment and infrastructure due to impacts on upstream infrastructural developments and potential environmental impacts should quarries and borrow pits need to be developed outside of the basin.

3. PHASE 2 FEASIBILITY LEVEL GEOTECHNICAL INVESTIGATIONS

As described in the Preliminary Study Report No. P WMA 12/T30/00/5212/3, and the Environmental Screening Report No. P WMA 12/T30/00/5212/2, the selected dam site based upon various decision criteria including technical, economic, environmental and social considerations was that the Ntabelanga Dam site was the preferred dam option, which was taken forward into a Phase 2 feasibility level study.

The Phase 2 feasibility level geotechnical investigations of the Ntabelanga site entailed rotary core drilling, geophysical seismic and electrical resistivity surveys, trial pitting, sampling and testing.

3.1 Physiography and Geology of the Project Area

3.1.1 General Description and Location

The location of the Ntabelanga site is indicated on Figure 3-1. The site is on the Tstitsa River, a tributary of the Mzimvubu River. It lies about 50 km, “as the crow flies” NNW of the city of Mthatha, about 30 km east of Maclear and about 22 km NW of Tsolo. Access into the site is possible along a number of gravel roads linking it to main road R396 and the N2, namely off the R396 at the top of Ntywenka Pass and at Somerville, as well as off the N2 at Qumbu.

The area is rural and the main land-use activity is pastoral stock and subsistence crop farming.

3.1.2 Climate

Climatic data for the nearby towns of Maclear and Tsolo are summarised in Table 3-1. There are variations between the two, which may be expected as Maclear lies at an elevation of 1 280 m.a.s.l. whereas Tsolo is at an elevation of 945 m.a.s.l.. As a result, Maclear displays generally lower temperature and higher rainfall figures than Tsolo. The elevation of Tsolo more closely resembles that at the dam site and therefore the climate at the Ntabelanga site is expected to be similar to that for Tsolo. This was apparent during the site investigation, where early morning winter temperatures at the dam site were appreciably warmer than those experienced in Maclear.

3.1.3 Topography

The proposed Ntabelanga dam site occupies a constricted, steep-sided valley displaying a pronounced geological influence, brought about by an interplay of sedimentary incision in variably resistant rock types and the positive relief features of a highly resistant dolerite sill. The valley constriction is localised and opens up both upstream and downstream of the site, although the general topography of the area is hilly, particularly on the northern side of the river. This will result in a relatively narrow and deep impoundment basin.

The elevation in the river section is about 897 m.a.s.l., rising steeply up both flanks to a maximum elevation of about 980 m.a.s.l.. Full supply level is proposed at an elevation of 947.3 m.a.s.l.

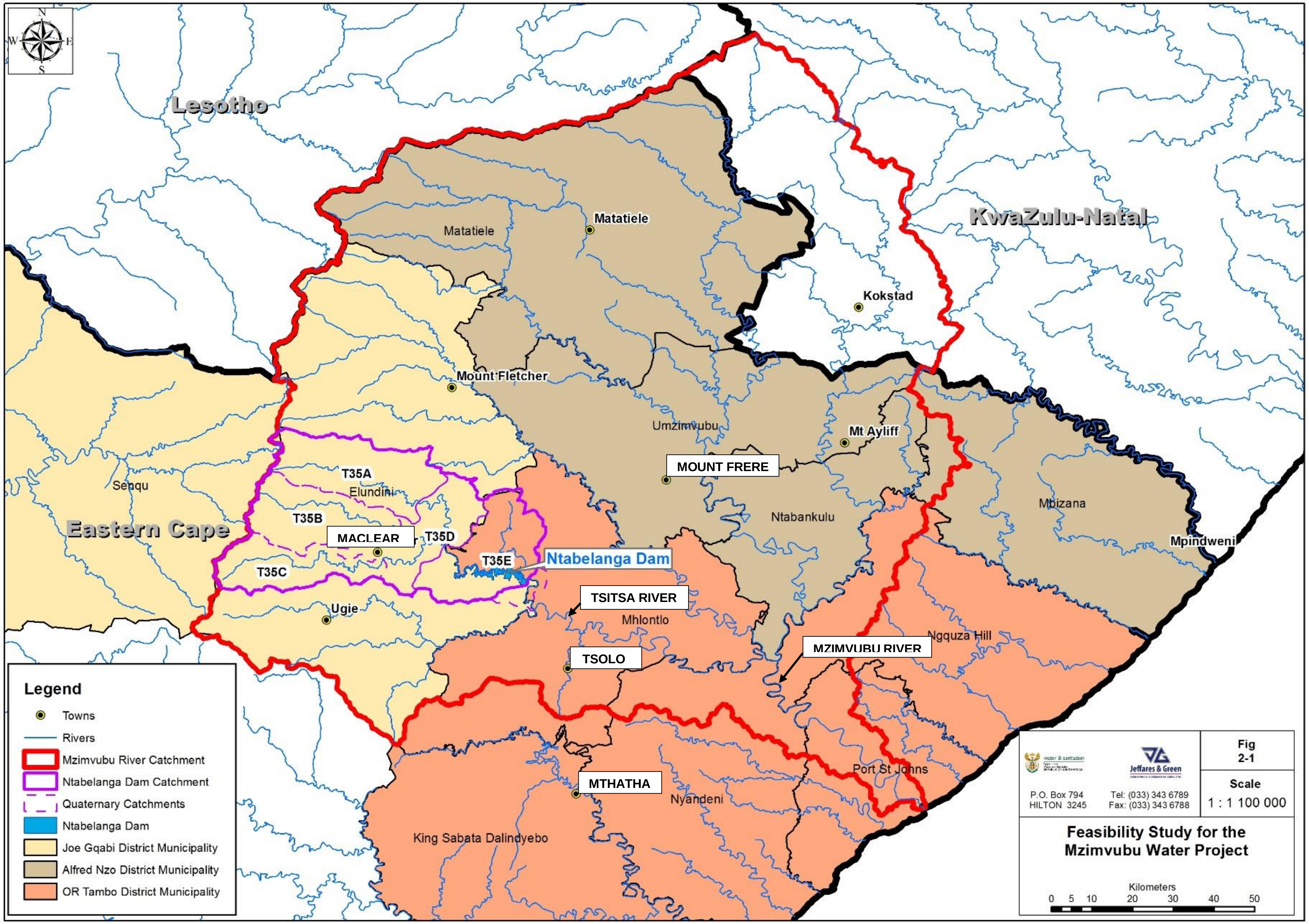


Figure 3-1: Locality Map of the Ntabelanga Dam

Table 3-1: Climatic Data for Maclear and Tsolo

Climatic Data for Maclear and Tsolo						
Month	**Maclear (Elevation 1 280 m.a.s.l.)**			**Tsolo (Elevation 945 m.a.s.l.)**		
	Average Rainfall (mm)	**Average Minimum Temperature (°C)**	**Average Maximum Temperature (°C)**	**Average Rainfall (mm)**	**Average Minimum Temperature (°C)**	**Average Maximum Temperature (°C)**
Jan	130	13.9	26.3	108	15.1	26.5
Feb	121	13.9	26	107	15.2	26.4
Mar	113	12.6	24.8	107	14.1	25.7
Apr	46	9.3	22.5	47	10.7	23.7
May	24	5.6	20	26	7.1	21.7
Jun	13	0.8	16.4	15	3.5	19.5
Jul	13	0	16.3	17	3.2	19.4
Aug	21	3.1	18.8	22	5.2	21
Sep	38	7.32	21.7	42	8.2	22.5
Oct	64	9.5	23	68	10.5	23.4
Nov	88	11.3	24.3	89	12.4	24.4
Dec	115	12.6	25.7	101	13.7	25.7
Annual	786			749		

Information extracted from Climate-Data.org

3.1.4 Geology

The 1:250 000 Geological Series map 3128 Umtata (1979), indicates the project area to be underlain by sedimentary rocks of the Tarkastad Subgroup of the Beaufort Group of the Karoo Supergroup and post-Karoo intrusive dolerite. An extract of the geological map is presented as Figure D2 in Appendix D. According to Johnson *et al* (2006), the sedimentary component consists of an undifferentiated, upward fining sequence of fluvial and braided stream deposits, comprising inter-bedded sandstone and mudrock, with a source area to the south east. The sedimentary sequence in the project area is about 1 000m thick and is predominantly argillaceous, with the mudrock component being about 70%.

The arenaceous rocks comprise light brownish grey to greenish grey, fine to medium grained sandstone. The mudrock displays a reddish colouration indicative of deposition in an arid environment.

Despite the bedded nature of sandstone, permeability is generally low, except where there is a prominence of open joints and bedding partings, in which case it is expected that grouting will be successful.

Dolerite is a dark grey, crystalline, medium grained, hypabyssal igneous rock composed mainly of plagioclase feldspar and pyroxene, with accessory amounts of olivine, biotite, amphibole, apatite and iron ore minerals. The drill core displayed an ophitic texture, which is the enclosure or partial enclosure of the feldspar minerals by pyroxene, brought about by the sequence of mineral crystallisation. The dolerite occurs as a network of dykes, sills and sheets intruded into the sedimentary strata. It occurs mainly in the form of sills, intruded concordantly into the host sediments and which may vary in thickness from a few metres to over 200m. Dykes are also common in the project area, being generally only a few metres wide, but extending for several kilometres.

The intrusions frequently result in contact aureoles of peripheral alteration in both the dolerite and the sedimentary rocks, due to the “baking” effects of the intrusion and subsequent rapid cooling. This has also created a pronounced joint pattern in the dolerite, comprising vertical shrinkage joints and horizontal stress relief joints. This arrangement of jointing, whilst pronounced and persistent, is not conducive to high seepage, and water losses through a competent dolerite foundation are generally low. The joints are filled or coated by secondary calcite and chlorite due to later circulation by magmatic fluids.

It is the resistant nature of the dolerite at the site that has created the base level in the river and the narrow, steep sided valley.

Study of Ntabelanga geological maps, aerial photographs and field observations did not detect any linear structural features such as faults. There is a linear stream channel running down the left flank between the Line 1 and Line 2 alignments, but it is considered to be an erosional rather than a tectonic feature. A small shear zone was intersected in Borehole NL2/5, but it is considered to be localised rather than a regional feature. Borehole SP2 intersected a number of secondary dolerite stringer dykes intruded into the main dolerite body, but these have not compromised the integrity of the rock quality.

The climatic N-value of the project area is less than 5, with a value of 2.3 for Umtata (Weinert, 1980). This implies that the weathering of primary minerals will be predominantly by chemical decomposition. As dolerite is composed entirely of primary minerals it will decompose to produce generally deep weathering profiles of residual soils with a mineralogical composition that is different to that of the parent rock.

The bedrock geology is overlain by a variably thick mantle of residual and colluvial soils, except on the upper right flank where any weathering products have been removed by erosion, leaving outcrop and sub-outcrop of hard dolerite. The properties of the transported colluvial soils are dependent upon their mode of origin. Those of mainly doleritic derivation comprise red or black clayey soils. The former of these has been investigated and found suitable for use as impervious core.

The colluvial deposits originating mainly from sedimentary rocks generally display more drab colouration of greyish brown, yellowish brown to brown and light reddish to reddish brown, and range in composition from gravely sand to silty sand, silty clay, silt and clay. There are extensive areas of severe gulley erosion on the inter-fluvial areas adjacent to stream channels. The erosional and piping characteristics are suggestive of the presence of dispersive soils.

Alluvial sand occurs in the course of the Tsitsa River and major tributary rivers and streams. Due to the steep and incised nature of the rivers, sand is mainly confined to the river channel, with few and only localised over-bank deposits.

The area has a low seismic hazard rating with a Modified Mercalli Scale (MMS) intensity of vi, equating to a horizontal peak ground acceleration of 66 $cm.s^{-2}$ and a vertical peak ground acceleration of 45 $cm.s^{-2}$, with a 90% probability of these figures not being exceeded during a period of 100years, for a maximum credible intensity (MCI) of xii (Fernandez and Guzman, 1979). A detailed seismic hazard analysis of the project area was conducted by Professor Kijko of the Natural Hazard Assessment Centre. His conclusion was that according to the applied guidelines the Ntabelanga dam site is rated as low risk. Please refer to the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/5212/12.

3.2 Rotary Core Drilling

The Phase 2 drilling investigation entailed the drilling of an additional sixteen boreholes with a total drilling length of 458.81m, the positions of which are indicated on Figure 3-2.

The Phase 2 drilling was undertaken by Weppelmann Geotechnical Drilling CC, as an extension of the Phase 1 contract. The logging of the boreholes was carried out according to the method prescribed by the Core Logging Committee (1976).

Two alternate dam alignments were proposed on the Ntabelanga site, annotated Line 1 and line 2 on Figure 3-2. Initially the downstream or Line 2 alignment was the preferred option and the Phase 2 boreholes were positioned to concentrate on that alignment. The drilling was programmed to start on the Line 2 left flank, so that any flaws could be identified at an early stage and the necessary revisions made to the investigation approach.

During the course of the drilling investigation the preference switched to the Line 1 alignment and the positions of certain of the undrilled boreholes were revised to accommodate this. Two of the boreholes on the Line 2 alignment were inclined at 60° and “stitched” beneath the river section to pick up high angle joints and other structural features in the foundations of the highest part of the dam structure.

Subsequently a third alignment was proposed, which is marginally upstream of the Line 1 alignment, as indicated by the boreholes located between line 1 and Line 2.

Optimisation of the dam axis will require further investigation during the detailed design stage of the project.

All drilling was by rotary core using N-size, double-tube core barrels fitted with diamond bit crowns. Water pressure tests were conducted in most of the boreholes at various depths and stage lengths. The borehole logs are presented in Appendix E.

The boreholes intersected dolerite with the exception of Borehole N3 on the upper left flank of the Line 1 alignment, which intersected sandstone. The boreholes show a distinct differentiation between founding conditions on the left flank and the right flank, with the right flank displaying consistently better conditions.

Boreholes SP1, SP2 and SP3 were drilled to the south east of the Line 1 upper right flank, as indicated on Figure D3 and D5, along a spillway alignment favoured at the time for earth-fill or rock-fill dam types.

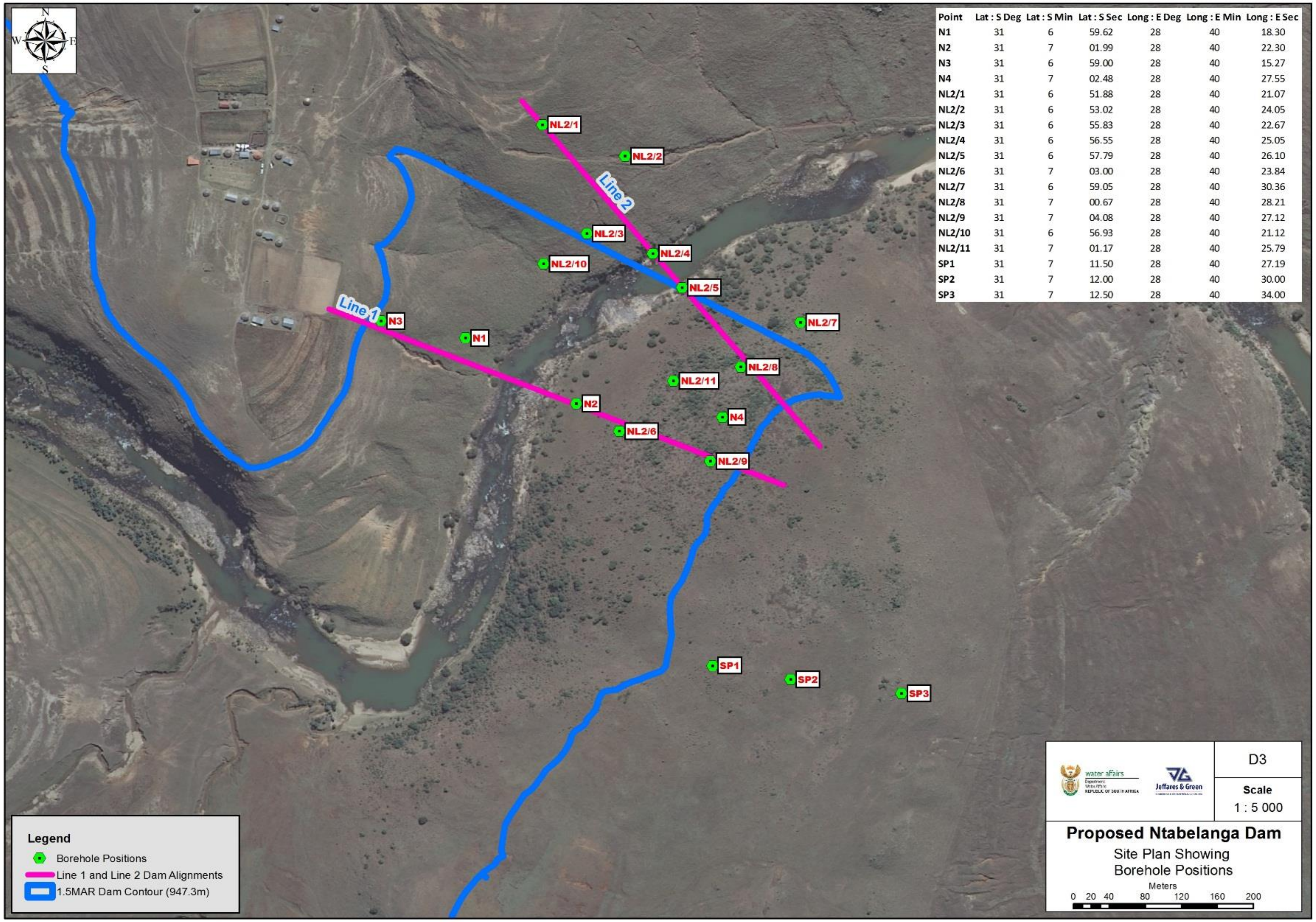
Locating the spillway on the upper right flank has the objective of duplicating its excavation as a rock quarry for the procurement of hard rock dolerite.

Subsequently three spillway alternatives were proposed, including one aligned on the upper left flank. Should a concrete gravity dam type be favoured, it is likely that a central, in-channel spillway option will be adopted.

3.2.1 Explanation of Terms

Core logging was carried out according to the prescribed methodology of the Core Logging Committee (1976).

- **Core Recovery**: Core recovery is the measured length of core recovered per drill run expressed as a percentage of the drill run, where the drill run is the length of drilling advance made in each drilling interval before the drill string is extracted to empty the core barrel. Values over 100% may indicate the recovery of core from a previous drill run in the drill run being described. Poor core recovery may be indicative of core loss in weak and highly fractured rock, but obviously depends upon the quality of the drilling. Annotated CR in the borehole summary tables on the following pages.



Point	Lat : S Deg	Lat : S Min	Lat : S Sec	Long : E Deg	Long : E Min	Long : E Sec
N1	31	6	59.62	28	40	18.30
N2	31	7	01.99	28	40	22.30
N3	31	6	59.00	28	40	15.27
N4	31	7	02.48	28	40	27.55
NL2/1	31	6	51.88	28	40	21.07
NL2/2	31	6	53.02	28	40	24.05
NL2/3	31	6	55.83	28	40	22.67
NL2/4	31	6	56.55	28	40	25.05
NL2/5	31	6	57.79	28	40	26.10
NL2/6	31	7	03.00	28	40	23.84
NL2/7	31	6	59.05	28	40	30.36
NL2/8	31	7	00.67	28	40	28.21
NL2/9	31	7	04.08	28	40	27.12
NL2/10	31	6	56.93	28	40	21.12
NL2/11	31	7	01.17	28	40	25.79
SP1	31	7	11.50	28	40	27.19
SP2	31	7	12.00	28	40	30.00
SP3	31	7	12.50	28	40	34.00

Figure 3-2: Alternative Dam Wall Alignments

- **RQD:** Rock Quality Designation (RQD) is the total length of individual core sticks exceeding 100mm in length expressed as a percentage of the drill run.
- **Fracture frequency:** Is the number of natural fractures / separations (excluding drilling breaks) that occur per metre of core recorded over the actual length of core over which that frequency occurs. Fracture frequency values are presented in the range of 0 to >20, as specific numbers greater than 20 are not considered significant. Annotated FF in the borehole summary tables on the following pages.

- **DMR:** DMR is the Dam Mass Rating according to Romana (2004) and is an adaptation of the rock mass rating (RMR) proposed by Bienawski (1973).

- **GSI**: The Geological Strength Index according to Hoek and Brown (1997), which is based upon the relationship between rockmass structure or discontinuity spacing and discontinuity condition. Values were derived using the chart for estimating GSI input values into the RocLab software program.

- **Water Pressure Test:** Is a pump-in test carried out to measure rock permeability. The test involves sealing off sections of a borehole with packers, after which five consecutive pump-in tests are done, each of 10 minutes duration. Test pressures are calculated according to the vertical depth to the top of the test stage. Pressures increase sequentially from the first to the third test and then decrease again by the same order of magnitude back to the fifth test. The results obtained were analysed according to the method of Houlsby (1974) and expressed as lugeon units, where 1 lugeon is equivalent to the water loss in litres per minute over a 1m length of borehole at a pressure of 1 MPa.

3.3 Left Flank Boreholes

3.3.1 Line 1

The results of the drilling investigation on the Left Flank, Line 1 alignment are summarised in Table 3-2. For Borehole N1 drilled during Phase 1, refer to Table 2-5.

Table 3-2: Borehole N3: Upper Left Flank, Line 1

Borehole N3: Upper Left Flank, Line 1, 31°06’59.0”S; 28°40’15.2”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 7.7	Colluvial clayey sand / sandy clay and boulders	6 – 9	145	Void filling
7.7 – 8.83	Highly to medium weathered, thinly bedded, closely to medium jointed, soft rock, Sandstone CR 65 – 100, RQD 0, FF >20			
8.83 – 14.64	Slightly weathered, moderately bedded and jointed, medium hard to hard rock, Sandstone with narrow discontinuity separations CR 95 – 100, RQD 47 – 88, FF 6 - 14	9 – 12	0	-
14.64 – 20.22	Unweathered, moderately bedded and jointed, hard rock, Sandstone with narrow, stained discontinuity planes. CR 92 – 100, RQD 64 – 97, FF 3 - 8	12 – 20.22	0	-

The two boreholes drilled on the Left Flank Line 1 alignment indicate the need for a relatively deep cut-off trench for a fill dam option and relatively deep foundation excavation for a concrete dam option.

Borehole N1 drilled on the lower flank indicated moderately weathered, closely jointed dolerite below a depth of 4.35m, with moderate values for core recovery, rock quality designation (RQD) and fracture frequency.

Water losses are also relatively high with a lugeon value of 62. From a depth of 9.3m the rock is slightly weathered and the values for core recovery, RQD and fracture frequency values improve significantly. The water pressure test over this section of the borehole was aborted as the packers could not be seated properly in the jointed rock formation and were leaking. Below a depth of 11.84 m the rock is good quality unweathered, widely jointed, very hard rock, dolerite with high core recovery and RQD values and correspondingly low fracture frequency and lugeon values.

Borehole N3 drilled on the upper left flank intersected transported materials and weak, weathered sandstone to a depth of 8.83m. Below a depth of 8.83 m the sandstone is slightly weathered, moderately fractured and bedded, moderate to hard rock with interlaminations of a more silty nature. The core recovery and RQD values are moderate to high and fracture frequency and lugeon values are low. At 14.64 m it becomes unweathered, moderately bedded, hard rock, sandstone with good values for core recovery, RQD and fracture frequency. Water losses are low, with zero lugeons recorded for the water pressure test.

3.3.2 Line 2

The results of the drilling investigation on the Left Flank, Line 2 alignment are summarised in Tables 3-3 to 3-7.

Table 3-3: Borehole NL2/1: Upper Left Flank, Line 2

Borehole NL2/1: Upper Left Flank, Line 2, 31°06’51.8”S; 28°40’21.0”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 1.5	Colluvial gravely and clayey sand with pebbles and boulders			
1.5 – 8.29	Medium weathered, very closely to closely jointed, medium hard rock, Dolerite with wide, filled joint separations. CR 29 – 97, RQD 0 – 39, FF 10 - >20			
8.29 – 15.15	Slightly weathered to in places medium weathered, medium jointed to closely jointed, medium hard rock, Dolerite with wide, stained and filled joints. CR 73 – 100, RQD 0 – 71, FF 11 - >20	9 – 15.15	78	Dilation

Table 3-4: Borehole NL2/2: Upper Left Flank, Line 2

Borehole NL2/2: Upper Left Flank, Line 2, 31°06'52.9"S; 28°40'24.2"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.8	Residual Dolerite			
0.8 – 19.99	Alternating zones ranging from completely weathered, friable / very closely jointed, very soft rock, to highly and moderately weathered, closely jointed, soft rock, Dolerite with a thin zone of unweathered, widely jointed, very hard rock, Dolerite. CR mainly material recovery to a core recovery of 64, RQD 0 – 55, FF >20			
19.99 – 25.25	Unweathered, widely jointed, very hard rock, Dolerite CR 100, RQD 80 – 100, FF1 - 7	No WPT		

Table 3-5: Borehole NL2/3: Lower Left Flank, Line 2

Borehole NL2/3: Lower Left Flank, Line 2, 31°06'55.8"S; 28°40'22.8"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 1.96	Colluvial clayey sand			
1.96 – 8.63	Residual becoming completely weathered, Dolerite	6 – 9	0	Void filling
8.63 – 11.19	Slightly weathered becoming unweathered, widely jointed with closely jointed zones, very hard rock, Dolerite with wide, gouge filled joints. CR 48 – 100, RQD 39 – 83, FF 2 - 6	9 – 12	0	Laminar
11.19 – 40.07	Unweathered, widely to very widely jointed with medium jointed zones, very hard rock, Dolerite with narrow, unaltered joints CR 92 – 100, RQD 73 – 100, FF0 – 9	12 – 15 15 – 18 18 – 21 21 – 24 24 – 27 27 – 30 30 – 33 33 – 36 36 – 40.07	0 0 0 4 10 0 36 73 31	- Void filling - Dilation Dilation Dilation/ Void filling Dilation Turbulent Dilation

Table 3-6: Borehole NL2/4: Left River Bank, Line 2

Borehole NL2/4: Left River Bank, Line 2, 31°06'56.5"S; 28°40'25.0"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 3.64	Alluvial sand, pebbles and boulders			
3.64 – 15.74	Unweathered, medium jointed, very hard rock, Dolerite with zones of more intense weathering and fracturing. CR 63 – 100, RQD 0 – 100, FF 0 - >20	7.8 – 10.4 10.4 – 13 13 – 15.6	0 0 0	- - -
15.74 – 31.39	Unweathered, medium to widely jointed, very hard rock, Dolerite with narrow, unaltered joints. CR 89 – 100, RQD 77 – 100, FF 0 – 7	15.6 – 18.2 18.2 – 20.8 20.8 – 23.4 23.4 – 26 26 – 31.39	0 0 0 0 1	- - - Dilation Dilation

Table 3-7: Borehole NL2/10: Lower Left Flank, Line 2

Borehole NL2/10: Lower Left Flank, Line 2 (towards 1), 31°06'56.9"S; 28°40'21.0"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 2.88	Colluvial gravely sandy clay with cobbles and boulders			
2.88 – 3.45	Completely weathered, friable, very soft rock, Dolerite			
3.45 – 5.05	Slightly weathered, closely to in places medium jointed, hard rock, Dolerite with wide, gouge filled joints. CR 91 – 100, RQD 39 – 47, FF 11 – 20	5 – 8	0	Void filling
5.05 – 12.32	Unweathered, zones of close, medium and very wide jointing, very hard rock, Dolerite with narrow separated, stained joint planes. CR 54 – 100, RQD 0 – 100, FF 1 - >20	8 – 11 11 – 14	0 0	- -
12.32 – 39.53	Unweathered, widely to very widely jointed, Dolerite with narrow, unaltered joints. Narrow zone of close jointing between 37.43 and 37.56m. CR 65 – 100, RQD 61 – 100, FF 0 - 4	14 – 17 17 – 20 20 – 23 23 – 26 26 – 29 29 – 39.53	0 0 4 10 0 10	Void filling - Dilation Dilation Dilation Dilation

Borehole NL2/10 was drilled between Line 1 and Line 2, but is located closer to Line 2. It indicates good quality dolerite below a depth of 5.05 m.

The drilling indicates the Line 2 alignment to be underlain entirely by dolerite. The upper left flank is underlain by a deep, intensely weathered profile, with poor quality rock extending to below the base depth of 15.15m in Borehole NL2/1 and to a depth of 19.99 m in Borehole NL2/2.

Borehole NL2/3 on the lower flank intersected good quality dolerite from a depth of 8.63 m. Borehole NL2/4 was drilled at the bottom of the Line 2 left flank on the riverbank. Good quality dolerite occurs from a depth of 3.6 m (4.2 m along the inclined length of the borehole).

The results of the drilling undertaken on the left flank justify the decision to revert back to Line 1 as the favoured alignment. As part of subsequent detailed investigations it is recommended that further drilling is undertaken to verify founding conditions along the Line 1 alignment.

On the left flank it is recommended that a combination of vertical and inclined boreholes are drilled to assess the dolerite / sandstone contact area at mid-slope, the left hand side river section and general infill drilling on the upstream and downstream dam footprint areas. Detailed assessment of the third proposed alignment would also be required in order to optimise the location of the dam axis.

3.4 Right Flank Boreholes

The results of the drilling undertaken on the right flank generally indicate more consistent and good founding conditions on both the Line 1 and Line 2 alignments.

3.4.1 Line 1

The drilling results along the Right Flank, Line 1 alignment are summarised in Tables 3-8 to 3-11. For Borehole N2, refer to Table 2-6.

Table 3-8: Borehole NL2/6: Lower to Mid Right Flank, Line 1

Borehole NL2/6: Lower to Mid Right Flank, Line 1, 31°07'03.0"S; 28°40'23.9"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.98	Colluvial clayey sand with boulders.			
0.98 – 34.65	Unweathered, wide to very widely jointed with localise medium to widely jointed, very hard rock, Dolerite with narrow separated, unaltered / coated joint planes. CR 89 – 100, RQD 85 – 100, FF 0 – 4	3 – 9 9 – 15 15 – 21 21 – 27 27 – 33 33 – 34.65	0 0 0 0 0 0	- - - - - -

Table 3-9: Borehole NL2/9: Upper Right Flank, Line 1

Borehole NL2/9: Upper Right Flank, Line 1, 31°07'04.1"S; 28°40'27.1"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.66	Colluvial boulders			
0.66 – 20.03	Unweathered, very widely jointed, very hard rock, Dolerite with narrow, unaltered / coated joint planes. CR 83 – 100, RQD 83 – 100, FF 0 – 2	3 – 9 9 – 15 15 – 20.03	0 0 0	- - -

Borehole N2 drilled on the Line 1 lower right flank intersected slightly weathered, closely jointed, moderately hard to hard rock, dolerite with widely separated, gouge-filled joints. From a depth of 6.39m the rock comprises competent, unweathered dolerite with high RQD, low fracture frequency and zero lugeon values.

The remaining boreholes drilled on this alignment and located on the mid to upper right flank intersected competent dolerite from shallow depths, which in boreholes NL2/6, NL2/9, N4 and NL2/11 are 0.98 m, 0.66 m, 2.43 m and 0.75 m respectively.

Table 3-10: Borehole NL2/11: Lower to Mid Right Flank, Line 1

Borehole NL2/11: Lower to Mid Right Flank, Line 1, 31°07'01.2"S; 28°40'25.7"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.75	Colluvial, clayey sand and boulders.			
0.75 – 3.48	Unweathered, medium to widely jointed, very hard rock, Dolerite with wide, gouge filled joints. CR 94 – 100, RQD 73 – 100, FF 2 – 6			
3.48 – 34.8	Unweathered, widely to very widely jointed, very hard rock, Dolerite with narrow, unaltered / coated joints. CR 97 – 100, RQD 83 – 100, FF 0 – 3	3 – 9 9 – 15 15 – 21 21 – 27 27 – 33 33 – 34.8	0 0 0 0 0 0	- - - - - -

Table 3-11: Borehole N4: upper Right Flank, Line 2

Borehole N4: Upper Right Flank, Line 1, 31°07'02.6"S; 28°40'27.5"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.33	Colluvila soil and pebbles			
0.33 – 2.43	Completely weathered, with corestones and boulders			
2.43 – 20.0	Unweathered, very widely jointed, very hard rock, Dolerite with narrow, unaltered / coated joints. CR 78 – 100, RQD 75 100, FF 0 – 2	3 – 6 8 – 14 14 – 20	0 0 0	- - Dilation

3.4.2 Line 2

The drilling results along the Right Flank, Line 2 alignment are summarised in Tables 3-12 to 3-14. Boreholes N4 and NL2/11 described below in the spillway options section are also applicable to this alignment.

Table 3-12: Borehole NL2/5: Right River Bank, Line 2

Borehole NL2/5: Right River Bank, Line 2, 31°06’58.0”S; 28°40’26.0”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 4.96	Alluvial pebbles and boulders.			
4.96 – 7.75	Unweathered, medium jointed, very hard rock, Dolerite with wide, stained joints.	6.1 – 8.7	68	Wash out
7.75 – 8.04	Shear zone			
8.04 – 11.93	Slightly weathered to unweathered, closely jointed, hard to very hard rock, Dolerite with wide to narrow separated, stained joints. CR 28 – 100, RQD 0 – 54, FF 5 - >20	8.7 – 11.3 11.3 – 13.9	112 299	Turbulent Void filling / Turbulent
11.93 – 35.27	Unweathered, medium to widely jointed, very hard rock, Dolerite with narrow zones of slightly weathered to unweathered, closely to very closely jointed, very hard rock, Dolerite. Joints narrow to occasionally wide, calcite coated. CR 91 – 100, RQD 56 – 100, FF 1 – 10	13.9 – 16.5 16.5 – 19.1 19.1 – 21.7 21.7 – 24.2 24.2 – 26.8 26.8 – 29.4 29.4 – 39.13	57 142 0 120 148 64 27	Void filling Turbulent Dilation Turbulent Wash out Dilation Laminar
35.27 – 39.13	Unweathered, very widely jointed, very hard rock, Dolerite with narrow, unaltered / coated joint planes. CR 97 – 100, RQD 92 – 100, FF 1 – 2			

Table 3-13: Borehole NL2/8: Mid Right Flank, Line 2

Borehole NL2/8: Mid Right Flank, Line 2, 31°07’00.8”S; 28°40’28.2”E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 0.1	Colluvial soil and pebbles			
0.1 – 8.21	Unweathered, widely to very widely jointed, very hard rock, Dolerite with narrow, unaltered / coated joint planes. CR 41 – 100, RQD 41 – 100, FF 0 – 1	2 – 5 5 – 8	0 0	Void filling -
8.21 – 10.96	Unweathered, medium jointed, very hard rock, Dolerite with narrow, coated joint planes. CR 98 – 100, RQD 93 – 100, FF 1 – 3	8 – 11	2	Turbulent
10.96 – 34.96	Unweathered, very widely jointed, very hard rock, Dolerite with narrow, coated joint planes. CR 97 – 100, RQD 97 – 100, FF 0 – 3	11 – 14 14 – 17 17 – 20 20 – 23 23 – 26 26 – 29 29 – 32 32 – 34.96	1 2 0 0 0 0 0 0	Void filling Dilation Void filling - - - - -

Table 3-14: Borehole NL2/7: Right Flank, Line 2 / Spillway Chute

Borehole NL2/7: Right Flank, Line 2 / Spillway Chute, 31°06'5695"S; 28°40'21.0"E				
Borehole Profile Summary		**Water Pressure Tests**		
Depth (m)	**Description**	**Depth (m)**	**Lugeons**	
			Value	**Flow**
0 – 2.7	Colluvial clayey sand / sandy clay			
2.7 – 9.12	Residual dolerite			
9.12 – 11.23	Completely weathered, friable to very closely jointed, very soft rock, Dolerite.			
11.23 – 14.1	Unweathered, medium to widely jointed, very hard rock, Dolerite with narrow stained joint planes. CR 86 – 100, RQD 40 – 100, FF 2 - >20			

Borehole NL2/5 was drilled at the bottom of the Line 2 right flank on the riverbank. Bedrock was intersected at a depth of 4.96 m (5.73 m along the inclined length of the borehole).

From here to a depth of 11.9 m (13.77 m along the inclined length of the borehole) the rock displays variable degrees of weathering and fracturing, with zones of more intense weathering and jointing characterised by low RQD values and high fracture frequencies, including a narrow shear zone between 7.75 and 8.04 m (8.95 and 9.28 m along the inclined length of the borehole). Below a depth of 11.9 m, even though the rock is competent, lugeon values remain relatively high.

As the rock quality is generally good, the relatively high water losses could possibly be attributable to leakage of packers. The literature (Geological Survey Manual – Geological Feasibility Investigations for Dam Sites) mentions one of the limitations of water pressure tests being the susceptibility for packers to sometimes not form a proper seal against the borehole sides, with resultant leakage. This can be very difficult or almost impossible to detect, especially when leakage occurs through the bottom packer in a double packer test.

Competent dolerite occurs from a depth of 0.01 m in borehole NL2/8. The intermediate boreholes, N4 and NL2/1, as previously mentioned indicate good quality dolerite prevailing from near-surface. Furthermore, good founding is apparent over the entire mid to upper right flank in the form of visible surface outcrop and sub-outcrop.

Borehole NL2/7 was drilled on the side-slope off the main spur forming the right flank, on the proposed side-channel spillway chute alignment (Spillway Option 1). The borehole displays a deep weathering profile, with competent dolerite only occurring below a depth of 11.23 m.

Whilst more consistent and better founding conditions for the two alignments are indicated on the right flank compared to the left flank, again based on the drilling undertaken the Line 1 alignment is considered superior in terms of founding and depth to good quality dolerite.

It is recommended that additional drilling on the right flank during the detailed investigation assess the right hand side river section, as well as infill drilling concentrating on the lower to mid right flank dam axis and footprint areas. In addition, assessment of the third proposed alignment would be necessary to optimise the location of the dam axis.

3.5 Spillway Investigations

At the time of the geotechnical investigation, no decision had been made on the most feasible spillway type, position or alignment, although a number of preliminary alternatives had been proposed.

In the case of earth-fill or rock-fill dam types the spillway design initially envisaged a spillway excavation on the upper right flank, which would also serve for the exploitation of competent dolerite rock for use in construction.

Two alternative spillway alignments (Spillway Option 1 and Spillway Option 2) were initially proposed on the upper right flank. Subsequently a third alternative (Spillway Option 3) was proposed on the left flank. All three alternatives are illustrated on Figure 3-3.

A RCC dam alternative would most probably be designed with a central in-channel spillway. This would require a separate rock aggregate source, again ideally located on the mid to upper right flank.

3.5.1 Spillway Option 1

Spillway option 1 proposes a spillway channel cut into the upper right flank and orientated south to north, as indicated on Figure 3-3.

Dolerite outcrop and sub-outcrop is visible along the first approximately 330 m of the spillway axis. Boreholes and trial pits in close proximity to the spillway axis indicate the following:

Table 3-15: Spillway Option 1: Boreholes and Trial Pits

Spillway Option 1: Boreholes and Trial Pits	
Borehole / Trial Pit No.	**Description**
SP1 (Borehole)	Competent dolerite from 0.41m
NL2/9 (Borehole)	Competent dolerite from 0.66m
N4 (Borehole)	Competent dolerite from 2.43m
NL2/8 (Borehole)	Competent dolerite from 0.01m
NL2/7 (Borehole)	Colluvial soil to 2.7m Residual and completely weathered dolerite to 11.23m Competent dolerite from 11.23m
SSP1 (Trial Pit)	Residual dolerite to 1.7m Highly weathered dolerite from 1.7m to below 3.5m (end of trial pit)
SSP2 (Trial Pit)	Weathered mudrock to 2.2m Weathered dolerite from 2.2m to below 3.9m (end of trial pit)
SSP3 (Trial Pit)	Colluvial soil to 4.9m Residual mudrock from 4.9m to below 5m (end of trial pit)
D42 (Trial Pit)	Colluvial soil to below 2.6m (end of trial pit)
Exposure	Dolerite outcrop in the river

The boreholes corroborate surface and near surface competent dolerite along the hill crest, with deeper soils and weathering profiles down the hill slope. The transported and residual soils are particularly deep towards the end of the spillway chute before the outfall into the river. This implies a need to concrete-line the spillway chute to counter against excessive scour and erosion. Dolerite outcrop is visible in the river.

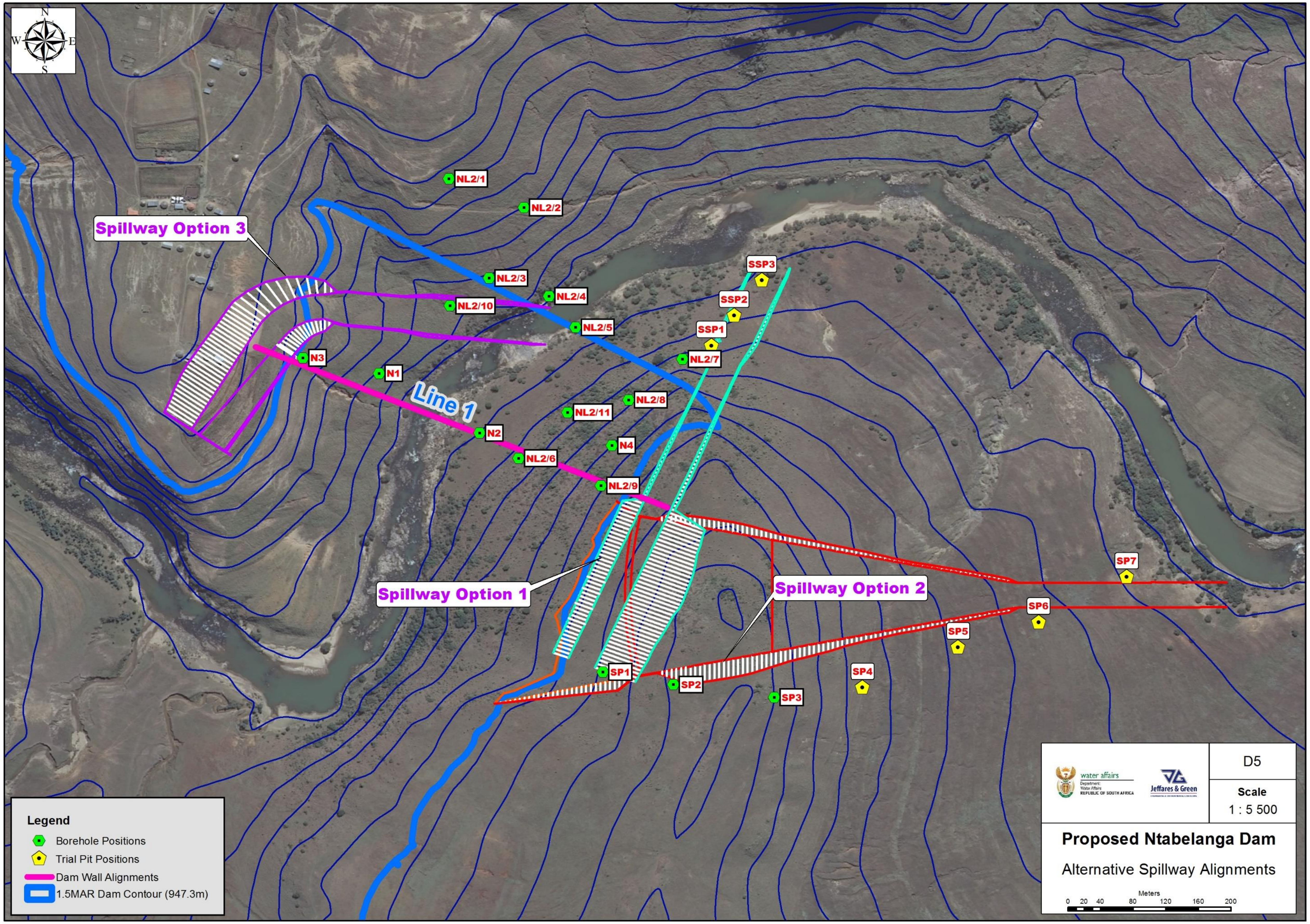


Figure 3-3: Ntabelanga Dam Alternative Spillway Alignments

3.5.2 Spillway Option 2

Spillway option 2 proposes an excavation through the hill upstream of the dam as indicated on Figure 3-3.

Dolerite outcrop and sub-outcrop is visible along the first approximately 190m of the spillway axis. Boreholes and trial pits in close proximity to the spillway axis indicate the following:

Table 3-16: Spillway Option 2: Boreholes and Trial Pits

Spillway Option 2: Boreholes and Trial Pits	
Borehole / Trial Pit No.	**Description**
SP1 (Borehole)	Competent dolerite from 0.41m
SP2 (Borehole)	Competent dolerite from 1m
SP3 (Borehole)	Completely to highly weathered dolerite to 5.5m Medium weathered dolerite to 8.5m Competent dolerite from 8.5m
SP4 (Trial Pit)	Weathered sandstone to 1.2m Weathered mudrock from 1.2m to below 1.7m (end of trial pit)
SP5 (Trial Pit)	Colluvial soil to 2.4m Weathered mudrock from 2.4m to below 3.3m (end of trial pit)
SP6 (Trial Pit)	Excavator refusal at 1m on slightly weathered sandstone
SP7 (Trial Pit)	Excavator refusal at 1.2m on slightly weathered sandstone
Exposure	Sandstone outcrop in the left hand side river terrace. Dolerite outcrop in the river

Spillway option 2 offers better founding conditions along the alignment of the lower chute than spillway option 1, but the large quantities of rock excavation are far in excess of the quantities required for embankment construction and the production of concrete aggregates. This would create the problem of disposal and spoiling of the excess materials excavated.

3.5.3 Spillway Option 3

Spillway option 3 (see Figure 3-3) proposes a side channel cut into the left flank, initially roughly perpendicular to the dam axis on the upper left flank, then curving downwards and running along the front of the downstream dam toe to intersect the river.

There is sub-outcrop of sandstone on the upper left flank, but the remainder of the spillway alignment is underlain by a relatively thick mantle of transported and residual soils. Boreholes in close proximity to the alignment indicate the following:

The upper spillway channel will be excavated in sandstone. From mid-slope, the chute and stilling basin excavation will be in dolerite. Being located on the steeper left flank, the depth of excavation, particularly along the western face will be deeper than that for the corresponding spillway option on the right flank, namely spillway option 1.

The sandstone derived from excavation is probably suitable for use as rock-fill, although durability tests would be required to verify this. It is not suitable for use as crushed aggregate.

Table 3-17: Spillway Option 3: Boreholes

Spillway Option 3: Boreholes	
Borehole No.	**Description**
N3	Unconsolidated materials to 7.7m Highly to medium weathered, soft rock, sandstone to 8.83m Below 8.83m slightly becoming unweathered, medium hard to hard rock, sandstone.
NL2/10	Unconsolidated to partially consolidated, transported and weathered dolerite to 3.45m. Slightly weathered, closely jointed, hard rock, dolerite to 5.05m. Competent dolerite below 5.05m.
NL2/3	Unconsolidated to partially consolidated transported and residual materials to 8.63m. Competent dolerite below 8.63m.
NL2/4	At river outfall. Competent dolerite below 3.6m

Dolerite derived from excavation will be suitable for use as rock-fill and concrete aggregates, although it is doubtful that this spillway option will provide sufficient hard rock dolerite for the project requirements, therefore necessitating an additional hard rock source to supply the shortfall. This additional source would ideally be located on the right flank, where the other two spillway options are proposed.

3.6 Geophysical Seismic Refraction and Electrical Resistivity Surveys

The geophysics was undertaken by the Council for Geoscience. It entailed the following:

- Four seismic refraction lines, each 180 m in length.
- Four multi-Electrode Resistivity Tomography (ERT) lines, of which three were 180m in length and one was 270 m in length.

The Council for Geoscience's report is presented in Appendix F.

3.6.1 Geophysical Line 1

The Line 1 geophysical traverses run roughly parallel and on the Line 1 dam alignment from the mid to upper right flank in a westerly direction to the right river bank, as indicated on Figure 1 of the Council for Geoscience (CfG) report in Appendix F. The lengths of both the seismic and ERT traverses are 180 m.

The seismic results indicate hard rock dolerite with seismic velocities of over 2 700 m/s occurring near to the surface or on surface and extending from the start on the mid to upper right flank down the slope to where the topography flattens out closer to the river.

This correlates well with boreholes NL2/9 and NL2/6. The seismic profile over this section is consistent and displays no pronounced velocity discontinuities or anomalies to suggest the presence of faults or other adverse features.

Along the lower right flank the seismic velocities indicate a deeper weathering profile. The 1 500 m/s seismic velocity contour extends to a maximum depth of about 6 m along this section. This is in close agreement with the rock-head depth intersected in borehole N2.

The electrical resistivity profile on the mid to upper right flank displays high resistivity values and mirrors that of the seismic profile. On the lower flank adjacent to the river, resistivity values have a more pronounced dip skewed closer to the river, possibly commensurate with saturated alluvial materials containing boulders.

There is a localised zone of lower resistivity between stake values 140 and 150, possibly indicative of more intense fracturing. This was not picked up by seismic, as seismic often does not detect lower velocity zones occurring beneath higher velocity zones. The proposed detailed investigation drilling on the right river bank should aim to verify conditions in this area.

3.6.2 Geophysical Line 2

- The Line 2 geophysical traverses run perpendicular to the Line 1 dam axis and alongside the right hand side of the river, as indicated on Figure 1 of the report. The length of the seismic traverse was 180 m and the ERT traverse was extended on the downstream side to a length of 270 m.
-
- The seismic results indicate a consistent profile comprising unconsolidated transported materials in the velocity range of 300 to 600 m/s, overlying partially consolidated residual and weathered materials in the velocity range of 900 to 1 200 m/s, in turn underlain by dolerite with seismic velocities exceeding 1 800 m/s. Rock head depths generally increase in a downstream direction. There is good correlation with borehole N2, at stake value 87 m.
-
- The ERT also indicates a consistent profile, as described above. The extended ERT section displays a shallower rock-head depth and this is consistent with borehole NL2/5 which intersected dolerite from a depth of 3.6 m.

-

3.6.3 Geophysical Line 3

The Line 3 geophysical traverses run roughly parallel and on the Line 1 dam alignment from the left hand side river bank to the upper left flank, as indicated in Appendix F. The lengths of both the seismic and ERT traverses are 180 m.

The seismic traverse indicates consistently thicker colluvial and residual soils over bedrock, which on the lower slope comprises dolerite and on the upper flank sandstone. Depths to dolerite bedrock on the lower flank are shallower than depths to competent sandstone higher up the slope, which are generally from about 7 m.

The ERT survey corroborates the results of the seismic refraction.

3.6.4 Geophysical Line 4

The Line 4 geophysical traverses run perpendicular to the Line 1 dam axis, on the lower to mid left flank and roughly parallel to the river. The lengths of both the seismic and ERT traverses are 180 m.

The seismic results show a bedrock profile over the major proportion of the line at a depth of about 5 m, which correlates with the depth in borehole N1 of 4.35 m to medium weathered dolerite. There is a section from a stake value of 20 to 50 m where bedrock depth is significantly shallower. This is not as readily apparent on the ERT section.

The geophysical seismic and electrical resistivity surveys do not show any anomalies suggesting the presence of faults or other structural discontinuities.

3.7 Trial Pitting

Trial pitting was carried out by means of a Caterpillar 428F tractor-loader-backhoe (TLB) and a Caterpillar 320D track-mounted excavator, at the dam site, the site of appurtenant works, spillway chute alignments, the possible saddle dam and for the investigation of potential construction material sources.

The trial pits were profiled according to the prescribed methodologies of Jennings *et al* (1973) and the Core Logging Committee (1976). The trial pit profile descriptions are presented in Appendix G and are described in the following sub-sections of the report. The positions of trial pits are indicated on Figure D4 in Appendix D.

3.7.1 Dam Site and Appurtenant Works

Trial pits were excavated on the footprint of the preferred dam alignment and in the area downstream of the dam, on the right hand or southern side of the Tsitsa River. As no conceptual layout design of the appurtenant works had been formulated, trial pits were randomly positioned to gain an overall assessment of the subsurface conditions in the area.

It had also been proposed to undertake trial pitting on the left flank, but at the time of trial pitting there was sustained rainfall and the steep, slippery conditions on the left flank slope were deemed too treacherous to put a machine on the steep slope. Trial pit D8 involved the profiling and sampling of a deep erosion channel.

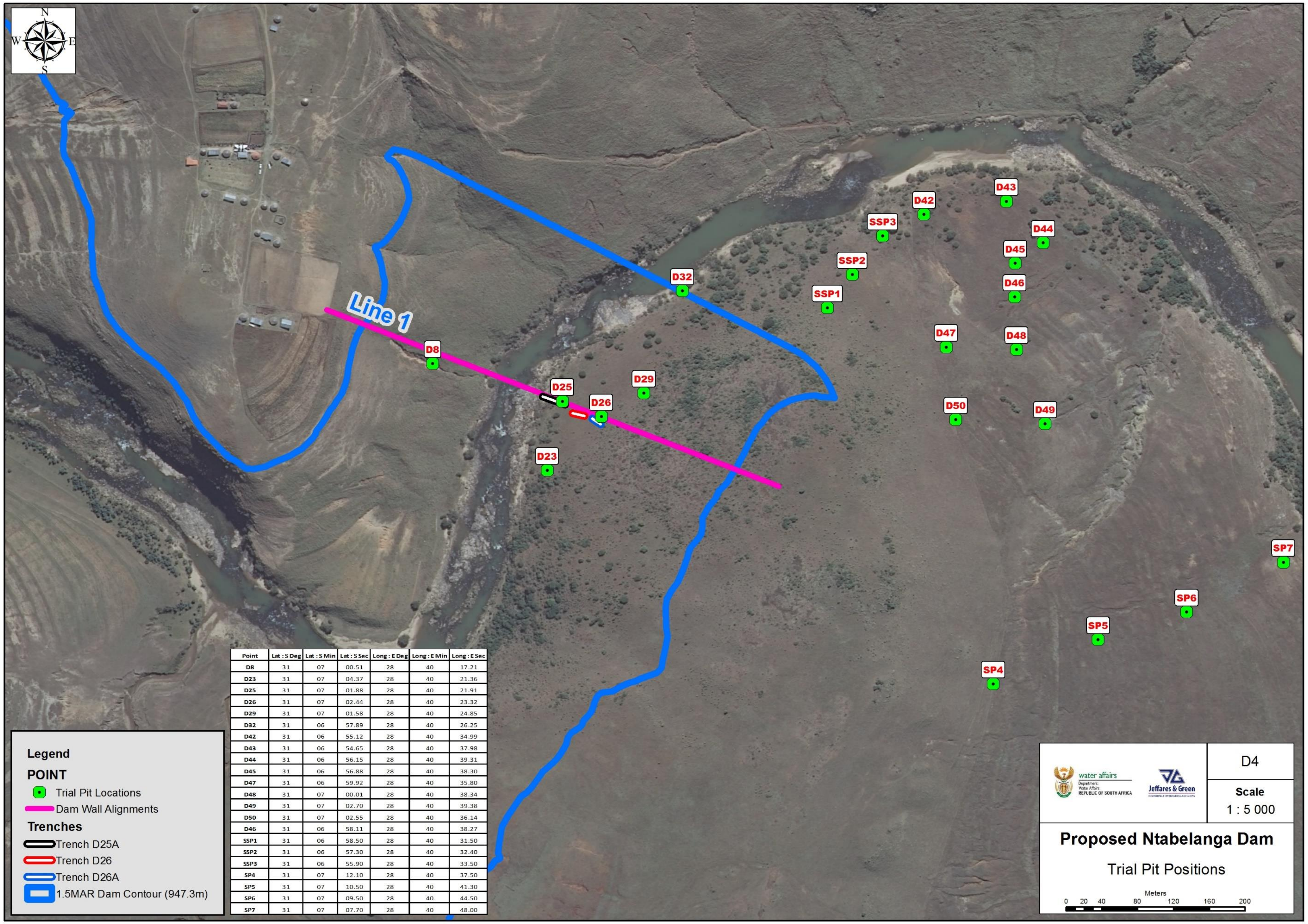
The positions of the trial pits are indicated on Figure 3-4 and the trial pit logs are presented in Appendix G.

Trial pits D23, D25 and D29 were excavated with a TLB on the lower right flank of the Line 1 alignment. Trial pit D23 intersected clayey colluvial and residual soils containing boulders to a depth of 1.6 m, where TLB refusal occurred on dolerite corestones. Trial pits D25 and D29 intersected similar profiles comprising sandy colluvial soils containing boulders, with refusal in both cases at a depth of 1m on boulders.

The trial pits were augmented by trenching along the dam centre-line by means of a 20 ton excavator. The trenches are annotated D25A, D26 and D26A. The traverse profile along the length of trench D25A is presented in Table 3-18. The zero traverse distance corresponds to the edge of the river.

As the lateral subsurface profiles encountered in trenches D26 and D26A were more consistent, they have been described as trial pits and the logs are presented in Appendix G1.

Both indicate a deep unconsolidated profile of colluvial soils underlain by residual dolerite soils.



Point	Lat : S Deg	Lat : S Min	Lat : S Sec	Long : E Deg	Long : E Min	Long : E Sec
D8	31	07	00.51	28	40	17.21
D23	31	07	04.37	28	40	21.36
D25	31	07	01.88	28	40	21.91
D26	31	07	02.44	28	40	23.32
D29	31	07	01.58	28	40	24.85
D32	31	06	57.89	28	40	26.25
D42	31	06	55.12	28	40	34.99
D43	31	06	54.65	28	40	37.98
D44	31	06	56.15	28	40	39.31
D45	31	06	56.88	28	40	38.30
D47	31	06	59.92	28	40	35.80
D48	31	07	00.01	28	40	38.34
D49	31	07	02.70	28	40	39.38
D50	31	07	02.55	28	40	36.14
D46	31	06	58.11	28	40	38.27
SSP1	31	06	58.50	28	40	31.50
SSP2	31	06	57.30	28	40	32.40
SSP3	31	06	55.90	28	40	33.50
SP4	31	07	12.10	28	40	37.50
SP5	31	07	10.50	28	40	41.30
SP6	31	07	09.50	28	40	44.50
SP7	31	07	07.70	28	40	48.00

Figure 3-4: Trial Pit Positions

Table 3-18: Description of D25A Trench Excavation

<table>
<tr><td colspan="12">D25A Centre-line Trench Excavation</td></tr>
<tr><td colspan="6">CO-ORDINATES
31°07'01.7"S
28°40'21.2"E</td><td colspan="6">31°07'02.0"S
28°40'22.0"E</td></tr>
<tr><td colspan="12">TRAVERSE DISTANCE (m)</td></tr>
<tr><td colspan="2">0</td><td colspan="2">5</td><td colspan="2">10</td><td colspan="2">15</td><td colspan="2">20</td><td colspan="2">25 26</td></tr>
<tr><td colspan="3">Depth Range 0.3 – 0.9m.</td><td colspan="3">Depth Range 0 – 1.1m.</td><td colspan="3">Depth Range 1.0 – 1.5m.</td><td colspan="3">Depth Range 0.5 – 1.2m</td></tr>
<tr><td colspan="3">Slightly moist to moist, light brown, loose, intact, sand containing numerous pebbles of dolerite and sandstone and boulders of mainly dolerite up to 1.5m diameter, alluvium. Roots.</td><td colspan="3">Slightly moist to moist, light reddish brown, loose, intact, silty sand with numerous, densely packed, matrix supported, mainly dolerite boulders 0.5 to 1.5m diameter, alluvium.</td><td colspan="3">Moist, dark reddish brown to reddish brown, loose, intact, silty clayey sand with numerous densely packed, mainly dolerite boulders 0.3 – 1.5m diameter. Matrix supported becoming clast supported at bottom, colluvium.</td><td colspan="3">Moist, red-brown, loose, intact, clayey sand / sandy clay with numerous densely packed dolerite boulders up to 1.5m diameter. Matrix supported becoming clast supported at bottom, colluvium.</td></tr>
<tr><td colspan="12">Difficult excavation at the bottom of the trench due to dense boulders</td></tr>
</table>

The profiles encountered in trial pits D42 to D50 are related to topographic position. Trial pits D42 and D43 near to the bottom of the slope just above the river intersected deep clayey colluvial soils with scattered boulder horizons.

At higher elevations on the slope the transported soils display evidence of ferruginisation. In addition at the higher elevations, weathered bedrock was intersected, namely residual mudrock in D44, weathered sandstone in D48 and D49, and at still higher elevations, weathered dolerite in D47 and D50.

3.7.2 Spillway Alignments

- Trial pit numbers SSP1, SSP2 and SSP3 were excavated along the proposed lower chute alignment, of Spillway Option 1. Subsequently, design changes have moved the proposed alignment westwards. The positions of the trial pits are indicated on Figure 3-4.
- Trial pit SSP1 intersected colluvial and residual dolerite soils to a depth of 1.7 m. Below this to beyond the base of the trial pit at 3.5m the material is highly weathered, soft rock, dolerite. The particle size analysis categorises the material as sandy gravel with a grading modulus of 2.35. The results indicate highly plastic fines with a plasticity index of 19 and linear shrinkage of 9.5, but because the fines component is only 20% the material has a low potential expansiveness.
- Trial pit SSP2 intersected a dolerite / mudrock contact. Mudrock was intersected in the upper part of the trial pit to a depth of 2.2 m. This was underlain by completely weathered, very soft rock, dolerite to 3.2 m, in turn underlain by highly weathered, soft rock, dolerite to beyond the base of the trial pit at 3.9 m.
- Trial pit SSP3 was excavated above the bank of the river, but is offset an appreciable distance off the currently proposed spillway alignment. It intersected clayey colluvial soil to a depth of 4.9 m, underlain by soft residual mudrock to below 5 m. The test result on the thick colluvial soil horizon indicates a plastic clayey silt with a potential expansiveness of medium. Such material would require undercutting and removal below spillway invert.
- Based upon the results of the trial pitting it is likely that the channel of the discharge chute of Spillway Option 1 would require lining.

- Trial pits SP4 to SP7 were excavated along the proposed alignment of the discharge chute of Spillway Option 2. Ongoing design changes have subsequently moved the alignment northwards. The trial pit positions are indicated on Figure 3-4.
- The profile in trial pit SP4 comprises 0.4 m of colluvial soil, underlain by medium to slightly weathered, medium hard rock, sandstone. Excavation of the sandstone was easily accomplished due to the presence of open joints and bedding planes. The sandstone is underlain at a depth of 1.2 m, by weathered mudrock, which extends to below the base of the trial pit.
- Trial pit SP5 intersected deep colluvial soils to a depth of 2.4 m, underlain by weathered mudrock, extending to beyond the base of the hole at 3.3 m. No machine refusal was experienced at the base of excavation.
- Trial pits SP6 and SP7 encountered excavator refusal at depths of 1 m and 1.2 m respectively. Sandstone outcrop beyond SP7 forms a cliff face above the bank of the river.

Spillway Option 2 offers better founding conditions along the alignment of the lower chute than Spillway Option 1.

The spillway design for a RCC dam alternative is a central, in-channel spillway founded on dolerite bedrock.

3.7.3 Saddle Dam

- It was noted that for larger dam storage capacities (i.e. FSL greater than 947.3 m.a.s.l.) there would be a possible need to construct a saddle dam. A plan of this saddle dam and the trial pit positions is shown Figure 3-5.
- Three trial pits were excavated along the axis of the saddle dam by means of a Caterpillar 320D excavator. The trial pit positions are indicated on Figure 3-5 and the profile logs are presented in Appendix G. No sampling of the materials was undertaken.
- Trial pit Saddle 1 was excavated on the south west extremity of the saddle dam axis. The trial pit intersected highly weathered, soft rock, mudrock at a depth of 1.2 m, which progressively became medium hard. Whilst absolute refusal was not experienced, at a depth of 1.4 m excavation was difficult and the rock was emitting smoke under the action of the excavator tines.
- Trial pit Saddle 2 was excavated in the central part of the saddle dam axis. Similar rock to that in trial pit Saddle 1 was intersected from a depth of 0.5 m and at 0.8 m difficult excavation was experienced, with the rock smoking under the action of excavation.
- Trial pit Saddle 3 was excavated on the north eastern extremity of the saddle dam axis. The trial pit intersected highly weathered, soft rock, sandstone at a depth of 1.3 m. The rock progressively became medium hard and difficult excavation was experienced at a depth of 1.6 m.
- The three trial pits excavated for the saddle dam indicate good founding at shallow depth. As design criteria pertaining to the saddle dam are not known, the need or otherwise for grouting cannot be commented on at this stage.

Figure 3-5: Possible Saddle Dam and Trial Pit Positions

3.8 Sampling and Testing

3.8.1 Unconfined Compressive Strength (UCS) Testing of Rock Core

The UCS test results are presented in Appendix H1 and are summarised in Table 3-19.

Table 3-19: Results of Unconfined Compressive Strength Tests

Results of Unconfined Compressive Strength Tests			
Borehole No.	**Depth (m)**	**UCS (MPa)**	**Rock Hardness**
N1	7.82 – 7.95	46.7	Hard rock
N2	7.75 – 7.9	166.8	Very hard rock
N2	12.31 – 12.49	136.6	Very hard rock
NL2/6	1.95 – 2.25	131.1	Very hard rock
NL2/9	4.89 – 5.09	203.2	Extremely hard rock

Rock hardness according to the Core Logging Committee (1976), where:

UCS range of 1 to 3MPa:	Very soft rock
UCS range of 3 to 10MPa:	Soft rock
UCS range of 10 to 25MPa:	Medium hard rock
UCS range of 25 to 70MPa:	Hard rock
UCS range of 70 to 200MPa:	Very hard rock
UCS >200MPa:	Extremely hard rock

All the above UCS tests were carried out on dolerite cores. The core sample from borehole N1 (7.82 – 7.95m) was retrieved from a section of the borehole between 4.35 m and 9.3 m, over which the rock has been described as brownish grey, medium weathered, medium hard rock, dolerite.

The intact rock strength of the sample, according to the UCS test, is hard rock. All other intact rock strengths are generally consistent with the descriptions given in the respective borehole logs.

3.8.2 Petrographic Analyses on Rock Core

The report on the petrographic analyses by the University of Pretoria is presented in Appendix H2.

The core sample from borehole N1 between a depth of 7.7 m and 7.82 m sample was retrieved from a section of the borehole described in the borehole log as being medium weathered. The thin section description indicates a low degree of alteration, confined to the margins of individual feldspar and pyroxene minerals, with the secondary alteration by-products comprising mainly chlorite and sericite.

The remainder of the core samples were retrieved from unweathered parts of boreholes. The degree of alteration is low and the secondary alteration products are generally not deleterious. Where smectite was identified, it occurs only in trace amounts.

From a mineralogical perspective, the dolerite is considered suitable for use in the construction of the dam and the production of crushed rock aggregates.

3.8.3 Borehole Water Pressure Tests

- The water pressure test results are reproduced in Table 3-20.

Table 3-20: Water Pressure Test Results

Water Pressure Test Results							
BH No	**Position**	**Stage Depth (m)**	**Fract Freq**	**Permeability**			**Comments**
				Lugeons per Test	**Repres. Lugeon**	**Flow Gp**	
N1	L1 LLF	5.96-8.98	12-13	13, 74, 48, 28, 62	48/62	Turbulent /Washout	
		8.98-11.84	11-20+	Test Aborted			Could not seat packer (leaking)
		11.84-14.79	3-8	3, 5, 7, 6, 4	5	Dilation	
		14.79-16.71	8-10	0, 2, 1, 0, 0	0	Void filling	
		17-20	9-19	5, 13, 10,0, 1	1	Void filling	
		20.84-23.59	2-3	0, 1, 7, 9, 5	5	Dilation	
		23.59-26.48	4-5	0, 0, 0, 0, 0	0	-	
		26.48-29.45	3-5	0, 1, 4, 4, 0	1	Dilation	
		29.45-32.81	3-9	0, 4, 5, 7, 0	4	Dilation	Zone of closer jointing
		32.81-35.23	0-6	1, 8, 7, 9, 0	7	Turbulent	
		35.23-39.82	2-6	5, 4, 5, 6, 5	5	Laminar	
N2	L1 ULF	8-11	0-3	0, 0, 0, 2, 0	0	-	
		11-14	1-3	0, 0, 7, 0, 0	0	Dilation	
		14-17	1-20+	1, 0, 0, 0, 0	0	-	Narrow fracture zone
		17-20	3-8	0, 0, 0, 0, 0	0	-	
		20-23	3-6	0, 0, 1, 0, 0	0	-	
		23-26	4-5	0, 0, 14, 0, 0	0	Dilation	
		26-29	3-5	0, 0, 8, 0, 0	0	Dilation	
		29-32	1	0, 2, 15, 7, 0	7	Dilation	Possible packer leakage
		32-35	1-2	0, 5, 15, 8, 0	8	Dilation	Possible packer leakage
		35-38	1-2	0, 12, 14, 14, 0	14	Dilation / Wash out	Possible packer leakage
		38-40.03	2	0, 2, 2, 3, 0	2	Dilation	
N3	L1 ULF	6-9	20+	245,162,152, 117, 145	145	Void filling	
		9-12	6-9	0, 0, 0, 0, 0	0	-	
		12-20.22	3-8	0, 0, 0, 0, 0	0	-	
N4	L1 URF	3-6	0-2	0, 0, 0, 0, 0	0	-	
		8-14	0-1	0, 0, 1, 0, 0	0	-	
		14-20	1-2	0, 0, 6, 0, 0	0	Dilation	
NL2/1	L2 ULF	9-15.15	11-20+	5, 6, 90, 89, 78	78	Dilation	

Water Pressure Test Results							
BH No	Position	Stage Depth (m)	Fract Freq	Permeability			Comments
				Lugeons per Test	Repres. Lugeon	Flow Gp	
NL2/2	L2 U-MLF						No WPT Poor rock quality
NL2/3	L2 LLF	6-9	20-20+	141, 4, 76, 38, 0	0	Void filling	
		9-12	4-6	0, 1, 0, 0, 0	0	Laminar	Residual / Completely weathered dolerite
		12-15	0	0, 0, 0, 0, 0	0	-	
		15-18	1	0, 73, 11, 0, 0	0	Void filling	
		18-21	1-2	0, 0, 0, 0, 0	0	-	
		21-24	2	0, 71, 83, 45, 4	4	Dilation	
		24-27	1	5, 0, 46, 33, 10	10	Dilation	Possible packer leakage
		27-30	2-9	0, 44, 41, 37, 0	0	Dilation / void filling	Flow pattern difficult to define
		30-33	1-2	31, 70, 41, 41, 36	36	Dilation	Possible packer leakage
		33-36	0-8	77, 76, 73, 69, 85	73	Turbulent	
		36-40.07	1-3	2, 10, 30, 31, 4	31	Dilation	Possible packer leakage
NL 2/4 Inclined b/hole. Stage depths are vertical	L2 L River bank	7.8-10.4	1-10	0, 0, 0, 0, 0	0	-	
		10.4-13	7-20+	0, 0, 0, 0, 0	0	-	
		13-15.6	5-14	0, 0, 0, 0, 0	0	-	
		15.6-18.2	2-3	0, 0, 0, 0, 0	0	-	
		18.2-20.8	1-3	0, 0, 0, 0, 0	0	-	
		20.8-23.4	0-1	0, 0, 0, 0, 0	0	-	
		23.4-26.0	2-5	0, 0, 0, 0, 0	0	Dilation	
		26.0-31.4	3-6	0, 1, 4, 0, 1	1	Dilation	
NL2/5 Inclined b/hole. Stage depths are vertical	L2 R River bank	6.1-8.7	3-20+	5, 50, 65, 68, 29	68	Wash out	
		8.7-11.3	8-20+	168, 125, 112, 113, 181	112	Turbulent	
		11.3-13.9	3-5	583, 225, 188, 205, 299	299	Void filling / Turbulent	Possible packer leakage
		13.9-16.5	2-4	80, 64, 52, 58, 57	57	Void filling	
		16.5-19.1	1-7	249, 188, 142, 183, 237	142	Turbulent	Possible packer leakage
		19.1-21.7	1-3	0, 0, 48, 0, 0	0	Dilation	

Water Pressure Test Results

BH No	Position	Stage Depth (m)	Fract Freq	Permeability			Comments
				Lugeons per Test	Repres. Lugeon	Flow Gp	
		21.7-24.2	3-10	142, 119, 120, 112, 157	120	Turbulent	Possible packer leakage
		24.2-26.8	4-8	0, 116, 109, 144, 148	148	Wash out	Possible packer leakage
		26.8-29.4	4-7	0, 64, 63, 65, 0	64	Dialtion	Possible packer leakage
		29.4-39.1	1-6	25, 25, 23, 35, 26	27	Laminar	
NL2/6	L1 MRF	3-9	0-1	0, 0, 0, 0, 0	0	-	
		9-15	0-4	0, 0, 0, 0, 0	0	--	
		15-21	1-3	0, 0, 0, 0, 0	0	-	
		21-27	1-2	0, 0, 0, 0, 0	0	-	
		27-33	2-3	0, 0, 0, 0, 0	0	-	
		33-34.65	1	0, 0, 0, 0, 0	0	-	
NL2/7	Spillway Chute						No WPT
NL 2/8	L2 MRF	2-5	0-1	123, 4, 1, 0, 0	0	Void filling	
		5-8	1	0, 0, 0, 0, 0	0	-	
		8-11	3	1, 6, 2, 5, 1	2	Turbulent	
		11-14	1-2	2, 5, 0, 0, 1	1	Void filling	
		14-17	1	2, 1, 11, 1, 2	2	Dilation	
		17-20	0-1	57, 1, 0, 0, 0	0	Void filling	
		20-23	1-2	1, 0, 0, 0, 0	0	-	
		23-26	1-3	0, 0, 0, 0, 0	0	-	
		26-29	1-2	0, 0, 0, 1, 0	0	-	
		29-32	1-3	0, 0, 0, 0, 0	0	-	
		32-34.96	1	0, 0, 0, 0, 0	0	-	
NL 2/9	L1 URF	3-9	0-2	0, 0, 0, 0, 0	0	-	
		9-15	0-2	0, 0, 0, 0, 0	0	-	
		15-20.03	1	0, 0, 0, 0, 0	0	-	
NL 2/10	L2 LLF	5-8	8-20+	141, 4, 76, 38, 0	0	Void filling	
		8-11	1-7	0, 0, 0, 0, 0	0	-	
		11-14	1-7	0, 0, 0, 0, 0	0	-	
		14-17	1-3	0, 73, 11, 0, 0	0	Void filling	
		17-20	0-3	0, 0, 0, 0, 0	0	-	
		20-23	1-2	0, 71, 83, 45, 4	4	Dilation	Possible packer leakage
		23-26	1-2	5, 0, 46, 33, 10	10	Dilation	Possible packer leakage
NL2/10 Contd.		26-29	1-2	0, 44, 41, 37, 0	0	Dilation	Possible packer leakage

Water Pressure Test Results							
BH No	**Position**	**Stage Depth (m)**	**Fract Freq**	**Permeability**			**Comments**
				Lugeons per Test	**Repres. Lugeon**	**Flow Gp**	
		29-39.53	1-4	9, 20, 12, 12, 10	10	Dilation	Possible packer leakage
N 2/11	L1 MRF	3-9	1-3	0, 0, 0, 0, 0	0	-	
		9-15	1-3	0, 0, 0, 0, 0	0	-	
		15-21	1-3	0, 0, 0, 0, 0	0	-	
		21-27	1-3	0, 0, 0, 0, 0	0	-	
		27-33	1-3	0, 0, 0, 0, 0	0	-	
		33-34.8	0-1	0, 0, 0, 0, 0	0	-	
SP1	Spillway						No WPT
SP2	Spillway						No WPT
SP3	Spillway						No WPT

3.8.4 Soil Properties and Compaction Tests

- The results of particle size analysis, Atterberg limits and standard Proctor moisture / density tests are presented in Appendix H3, Appendix H4 and are summarised in Table 3-21.

Table 3-21: Summary of Indicator and Standard Proctor Moisture / Density Tests

Summary of Indicator and Standard Proctor Moisture / Density Tests												
TP No.	**Depth (m)**	**Grv (%)**	**Snd (%)**	**Slt (%)**	**Cly (%)**	**GM**	**LL (%)**	**PI**	**LS (%)**	**USC**	**Proc. MDD (kg/m³)**	**Proc. OMC (%)**
C1A	0.4-1.0	10	13	31	46	0.44	41	13	9.5	ML		
C2	0.4-2.6	0	29	24	47	0.24	52	23	10.5	CH		
C2A	0.5-1.9	0	36	23	41	0.34	35	19	9.5	CL		
C4A	0.4-2.1	0	8	21	71	0.06	49	20	11.5	ML		
C5	0.6-2.7	3	19	40	38	0.21	39	20	10	CL		
C6	0.5-2.3	10	18	35	37	0.46	42	20	9.5	CL		
C7	0.9-2.9	3	14	33	50	0.19	41	23	13.5	CL		
C8	0.4-2.5	0	19	45	36	0.15	44	20	11.5	CL		
C9	0.5-2.2	3	15	30	52	0.24	48	26	12.5	CL		
C10	0.4-2.1	4	28	28	40	0.44	37	20	10	CL		
Mx*	-	3	19	28	50	0.28	46	23	11.5	CL	1523	26.2
C12	0.5-2.6	1	27	22	50	0.29	37	17	10	CL		
C15	0.4-1.8	3	34	45	18	0.34	34	12	7.5	CL		
C16	0.5-1.6	2	26	38	34	0.29	35	15	7.5	CL		
C18	0.3-2.9	1	39	28	32	0.38	22	10	5.5	CL		
C20	0.3-2.2	0	39	24	37	0.36	33	16	8	CL		
C22	0.3-2.6	0	9	49	42	0.03	59	25	10.5	ML		
C23	.25-1.7	4	29	28	39	0.39	36	16	8.5	CL		
C25	0.3-0.9	3	17	39	41	0.23	53	22	10.5	ML		
C28	0.5-2.6	0	7	52	41	0.02	61	27	15.5	ML		
C30	0.5-2.1	2	37	27	34	0.39	37	19	8	CL		
Mx**	-	0	27	34	39	0.24	43	21	11.5	CL	1563	23.9
F5	.25-1.2	31	44	13	12	1.6	25	11	5.5	SC		
F6	0.4-1.2	46	29	13	12	1.82	31	16	6.5	SC		
F8	0.6-1.6	16	58	16	10	1.05	21	9	3.5	SC		
F10	Chan'l	41	31	13	15	1.7	33	16	8.5	SC		
F12	0.3-1.6	44	36	15	5	1.75	20	5	3.5	SC		
F13	.25-.95	29	52	11	8	1.47	23	9	4	SC		
Mx#	-	64	22	8	6	2.19	22	8	4	GC	2057	10.5
F25	0.9-1.2	29	43	13	15	1.38	23	10	4	SC		
F28	0-1.5	18	47	26	9	1.16	23	6	4	SC/SM		
F29	0-0.6	23	49	15	13	1.46	25	9	6	SC		
F31	0.4-1	43	32	20	5	1.75	20	5	3.5	SC/SM		
F33	0.4-1	57	26	7	10	2.15	31	14	7.5	SC		
F34	0.1-0.8	32	37	23	8	1.39	21	6	4	SC/SM		
F37	0-1.5	28	43	17	12	1.36	22	7	4	SC/SM		
F37A	Chan"l	5	33	44	18	0.53	27	12	6	CL		
F39	0.2-1	43	32	15	10	1.74	23	9	5.5	SC		
Mx##	-	72	17	7	4	2.42	23	8	5.5	GP/GC	2060	11.3
D42	0.5-2.6	N	O	T		T	E	S	T	E	D	
D8/LF01	1.0-1.2	0	11	40	49	0.03	54	21	11.5	MH		
D26/RF01	1.0-1.3	0	26	23	51	0.24	41	23	10	CL		
SSP1	1.7-3.5	68	21	7	4	2.35	39	19	9.5	GC		
SSP3	0.7-4.9	1	22	45	32	0.2	40	18	10	CL		
SP5	0.3-2.4	0	16	26	58	0.14	47	22	11.5	CL		
SP7	.15-1.2	60	31	6	3	2.14	21	8	2.5	GP/GC		

Note:

- Grv is gravel component; Snd is sand component
- Slt is silt component; Cly is clay component
- GM is the grading modulus; LL is the liquid limit
- PI is the plasticity index; LS is the linear shrinkage
- USC is the Unified Soil Class
- Proc. MDD Standard Proctor maximum dry density
- Proc. OMC Standard Proctor optimum moisture content
- * Core Borrow Pit 1: equal proportions of C2+C4A+C7+C8+C10
- ** Core Borrow Pit 2: equal proportions of C12+C18+C22+C28+C30
- # Fill Borrow Pit 2: equal proportions of F5+F8+F10
- ## Fill Borrow Pit 1: equal proportions of F25+F29+F34+F37+F39

3.8.5 Dispersion Tests

Dispersion occurs in cohesive soils when the forces of repulsion between clay particles exceed the forces of attraction (Jermy and Walker, 1999). In the presence of water deflocculation occurs as the clay particles repel one another and go into suspension, resulting in a susceptibility to undergo erosion and piping. The tendency for dispersion is pronounced in soils with a high exchangeable sodium percentage or ESP (Elges, 1985).

The prominent gulley erosion in the project area is indicative of the presence of potentially dispersive soil. The gulley erosion is confined to the colluvial soils overlying sedimentary rocks, represented in the samples tested by sample F37A.

The dispersiveness of a soil can be determined by a number of laboratory tests, both physical and chemical. In this study two tests were undertaken, namely the double hydrometer test and the pinhole test. The tests were carried out on the materials proposed for use in the dam construction, namely from the proposed core and shell borrow pits.

The double hydrometer test involves two parallel tests, one being the standard hydrometer test and the second a hydrometer test in which no chemical dispersant is added. The proportion of the five micron size fraction that goes into suspension in the second test is expressed as a percentage of the five micron size fraction measured in the standard test. There are a number of different interpretations of the results, but generally a value below 30% is regarded as being non-dispersive.

The results of the dispersion tests are presented in Appendix H5. The materials from the proposed core and shell borrow pits categorise as non-dispersive to intermediate (slightly) dispersive according to the double hydrometer tests.

The pinhole test involves flushing water through a 1mm pinhole formed in a remoulded soil sample. Dispersiveness is evaluated according to the degree of erosion of the pinhole and the turbidity of the test water.

The pinhole test results undertaken on the borrow pit samples corroborate those of the double hydrometer tests on corresponding materials, generally categorising as non-dispersive to slightly dispersive.

The results are summarised in Table 3-22.

Table 3-22: Summary of Double Hydrometer and Pinhole Tests

Summary of Double Hydrometer and Pinhole Tests				
Trial Pit No.	**Depth (m)**	**Test Description**	**Result**	**Dispersivity**
C2	0.4 – 2.6	Double Hydrometer	28	Non-dispersive
C4A	0.4 – 2.1	Double Hydrometer	9	Non-dispersive
C7	0.9 – 2.9	Double Hydrometer	9	Non-dispersive
C8	0.4 – 2.5	Double Hydrometer	27	Non-dispersive
Mix*		Double Hydrometer	34	Intermediate
		Pinhole (98%Proc. MDD)	ND3	Intermediate
C12	0.5 – 2.6	Double Hydrometer	14	Non-dispersive
C18	0.3 – 2.9	Double Hydrometer	31	Intermediate
C28	0.5 – 2.6	Double Hydrometer	31	Intermediate
C30	0.5 – 2.1	Double Hydrometer	46	Intermediate
Mix**		Double Hydrometer	25	Non-dispersive
		Pinhole (98% Proc. MDD)	ND3	Intermediate
F37A	Channel	Double Hydrometer	48	Intermediate

- Note:
- * Core Borrow Pit 1: equal proportions of C2+C4A+C7+C8+C10
- ** Core Borrow Pit 2: equal proportions of C12+C18+C22+C28+C30

3.8.6 *Shear Strength and Consolidation Tests*

The results of the tri-axial and consolidation tests are presented in Appendix H7 and are summarised in Tables 3-23 and 3-24.

An earthfill dam will experience steady state conditions during its design life time. Consolidated undrained triaxial tests with pore pressure measurements were carried out to determine the effective shear strength parameters of disturbed and undisturbed samples. Duncan (1994), demonstrated that these effective shear strength parameters will be similar to the effective shear strength parameters determined by a consolidated drained triaxial test. The preference of the consolidated undrained triaxial test over the consolidated drained test is due to the relatively shorter time period to conduct a test as well as the graphical illustration of the stress path that a sample will experience during the testing procedure.

Table 3-23: Summary of Consolidated Undrained Triaxial Tests

Consolidated Undrained Triaxial Tests			
Trial Pit No.	**Depth (m)**	**Cohesion (kPa)**	**Friction Angle (degrees)**
D26 (RF 01)	1.0 - 1.3	26.5	23
D8 (LF 01)	1.0 - 1.2	17.4	27.7
Mix*		15.1	26.6
Mix**		0.2	27.3
Mix#		14.1	27.6
Mix##		6.9	32.3

Note:
- * Core Borrow Pit 1; equal proportions of C2+C4A+C7+C8+C10 (98% Proc. MDD)
- ** Core Borrow Pit 2: equal proportions of C12+C18+C22+C28+C30 (98% Proc. MDD)
- # Fill Borrow Pit 2: equal proportions of F5+F8+F10 (95% Proc. MDD)
- ## Fill Borrow Pit 1: equal proportions of F25+F29+F34+F37+F39 (95% Proc. MDD)

Table 3-24: Summary of Consolidation Tests

Test Pit No.						
	D26 (RF01) At in-situ density	**Effective Stress (kPa)**	**200**	**400**	**800**	**1600**
		m_v (1/Mpa)	0.223	0.1856	0.1124	0.0588
	D8 (LF01) At in-situ density	**Effective Stress (kPa)**	**200**	**400**	**800**	**1600**
		m_v (1/Mpa)	0.0862	0.0749	0.0966	0.0682
	Mix C2+C4A+C7+C8+C10 at 98% Proc. MDD	**Effective Stress (kPa)**	**200**	**400**	**800**	**1600**
		m_v (1/Mpa)	0.2643	0.2183	0.0846	0.0377
	Mix C12+C18+C22+C28+C30 At 98% Proc. MDD	**Effective Stress (kPa)**	**200**	**400**	**800**	**1600**
		m_v (1/Mpa)	0.2432	0.23	0.1068	0.0532
	Mix F5+F8+F10 At 95% Proc. MDD	**Effective Stress (kPa)**	**200**	**400**	**800**	**1600**
		m_v (1/Mpa)	0.1092	0.0697	0.0551	0.0368
	Mix F25+F29+F34+F37+F39 At 95% Proc. MDD	**Effective Stress (kPa)**	**200**	**400**	**800**	**1600**
		m_v (1/Mpa)	0.1063	0.0885	0.0566	0.0323

- Where m_v is the coefficient of volume compressibility
-

3.8.7 Permeability

The results of falling head permeability tests are presented in Appendix H8 and are summarised in Table 3-25.

Table 3-25: Summary of Falling Head Permeability Tests

Summary of Falling Head Permeability Tests			
Trial Pit No.	**Depth**	**Coefficient of Permeability**	**Remarks**
Mix*		$1.0x10^{-9}$ m/s	Remoulded to 98% of Proctor MDD
Mix**		$8.4x10^{-9}$ m/s	Remoulded to 98% of Proctor MDD
Mix#		2.1×10^{-9}m/s	Remoulded to 95% of Proctor MDD
Mix##		$2.6x10^{-9}$ m/s	Remoulded to 95% of Proctor MDD
D8 (LF01)	1.0 – 1.2m	1.4×10^{-8} m/s	In-situ density
D26 (RF01)	1.0 – 1.3m	2.3×10^{-8} m/s	In-situ density

Note:

- * Core Borrow Pit 1; equal proportions of C2+C4A+C7+C8+C10
- ** Core Borrow Pit 2: equal proportions of C12+C18+C22+C28+C30
- # Fill Borrow Pit 2: equal proportions of F5+F8+F10
- ## Fill Borrow Pit 1: equal proportions of F25+F29+F34+F37+F39
-

4. CONSTRUCTION MATERIALS

- The feasibility level geotechnical investigations of the Ntabelanga Dam site were undertaken before any decisions had been made on dam type or any volume calculations made on the various material requirements.
- The investigations undertaken were therefore non-specific to any dam type or material requirements and were structured to provide an overview of founding conditions and materials availability. The geotechnical and materials investigations undertaken during Phase 2 considered the following alternative dam designs:

  - Roller compacted concrete (RCC) dam;
  - Concrete faced rock-fill dam (CFRD);
  - Earth core rock-fill dam (ECRD); and
  - Earth embankment dam.

Estimated volumes of the various material categories for the alternative dam options are listed in Table 4-1.

Table 4-1: Material Volumes for Alternative Dam Types

Material Volumes for Alternative Dam Types				
Dam Type	**Crushed Rock**	**Shell (General Fill**	**Core**	**Sand**
RCC	500 000m³	n/a	n/a	200 000 m³
CFRD	1 300 000 m³	n/a	n/a	100 000 m³
ECRD	1 100 000 m³	n/a	260 000 m³	100 000 m³
Earth Embankment	65 000 m³	2 100 000 m³	500 000 m³	25 000 m³

Where:

Concrete aggregate
Concrete aggregate + rock-fill
Concrete aggregate + rock-fill + filters
Concrete aggregate + filters
Concrete aggregate + rip-rap + filters

4.1 Hard Rock

A quarry or quarries will be required for the production of concrete aggregate, rock-fill, rip-rap, and coarse filters / drainage medium.

Competent, hard rock dolerite underlies the middle to upper right flank, occurring either close to the surface or as surface outcrop.

The positions of boreholes drilled for the evaluation of dam foundations and spillway excavations are indicated on Figures 3-2 and 3-3. The depths to competent dolerite, as encountered in the boreholes drilled on the middle to upper right flank are summarised in Table 4-2.

Table 4-2: Middle and Upper Right Flank Boreholes

Middle and Upper right Flank Boreholes		
Borehole Number	**Depth to Competent Dolerite**	**Comments**
N4	2.43 m	Spillway option 1 channel
NL2/6	0.98 m	
NL2/7	11.23 m	Spillway option 1. Drilled on the side of the hill to the west of the spillway chute
NL2/8	0.01 m	Spillway option 1 channel
NL2/9	0.66 m	Immediately to the west of the Spillway option 1 alignment
NL2/11	0.75 m	To the west of the Spillway Option 1 alignment
SP1	0.41 m	Spillway option 2
SP2	1.00 m	Spillway option 2
SP3	8.50 m	Spillway option 2. Drilled on the side of the hill

Pieces of drilled rock core were retrieved from the core boxes and submitted for petrographic analyses and unconfined compressive strength (UCS) testing. The petrographic analyses indicate a relatively low degree of alteration and insignificant amounts of deleterious alteration products, such as smectite clay minerals.

UCS tests on core from the upper right flank indicate competent, high strength dolerite. The rock is suitable for use as crushed rock aggregates, rock-fill, rip-rap and as a coarse filter / drainage medium.

Visually the reserves of potentially good quality dolerite in the right flank to the east and south east of the dam are vast and are potentially far in excess of the required quantities for any of the dam alternatives listed above. Drilling indicates that a quarry located on the right flank upstream of the dam and within the basin would yield adequate rock aggregate for construction purposes, either solely from the spillway excavation or from additional excavation upstream of the spillway.

4.2 Sand for Concrete Aggregate and Filters

Sand along a section of the Tsitsa River upstream of the dam was sampled, as indicated by the yellow hatching on Figure 4-1.

The Tsitsa River in the project area generally flows in a relatively incised channel with sand deposits confined to the river channel. Over-bank deposits on inside meanders are of a restricted and localised nature.

Therefore sand deposits in the Tsitsa River are relatively narrow and will require selective seasonal exploitation during the dry season. Screening will be required to remove gravel (mudrock fragments), pebbles and boulders.

The laboratory test results are presented in Appendix H and are summarised in Table 4-3.

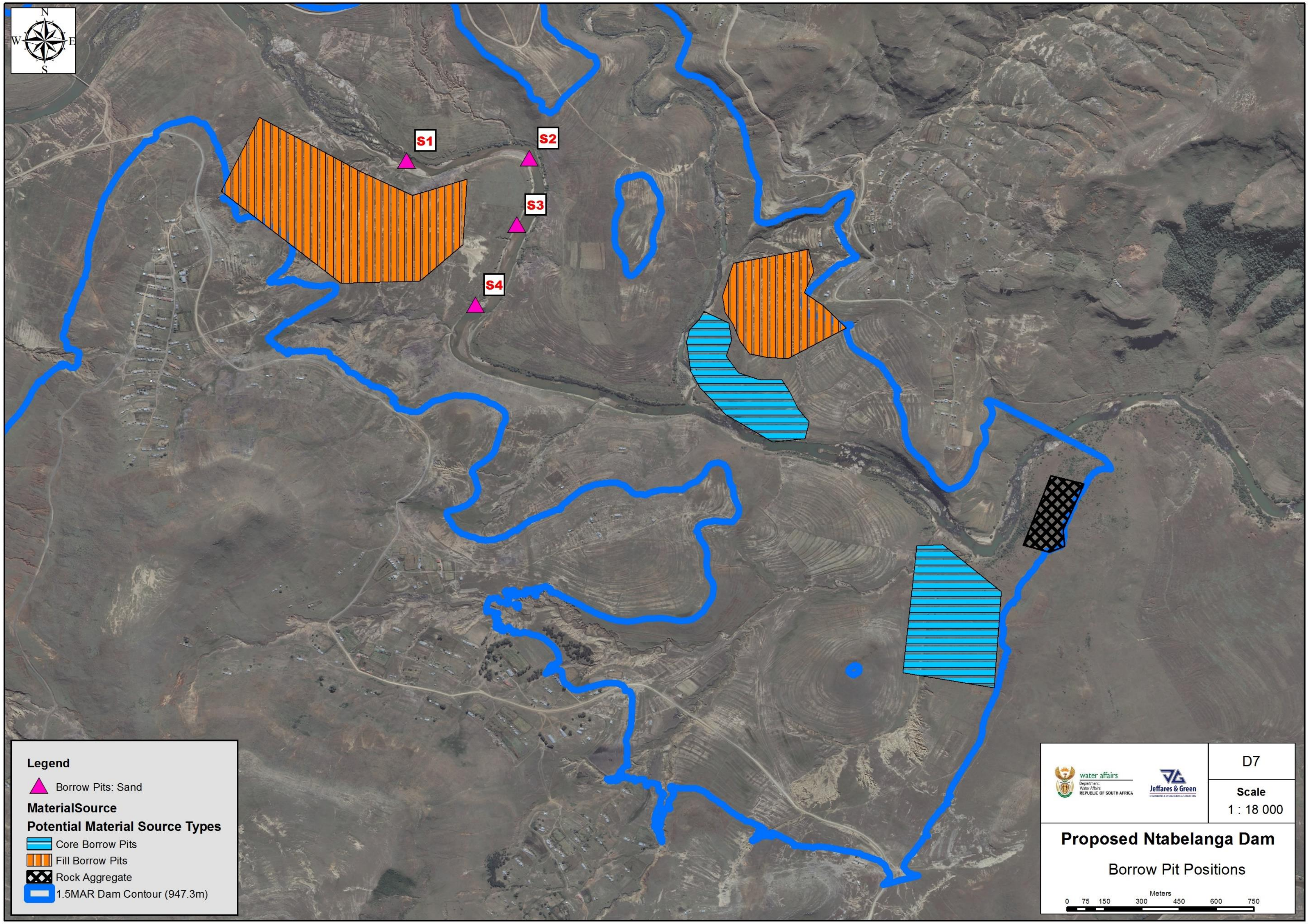


Figure 4-1: Positions of Potential Borrow Areas and Quarries

Table 4-3: Summary of Test Results on Tsitsa River Sand

Summary of Test Results on Tsitsa River Sand					
Parameter	**Specification**	**Sample Number**			
		S1	**S2**	**S3**	**S4**
% Passing 4.75mm	90 – 100	94	99	100	100
% Passing 0.150mm	5 – 25	2	5	11	4
Dust (% passing 0.075mm)	5 maximum	0.9	2.5	4.2	2
Fineness Modulus	1.2 – 3.5	2.55	2.35	1.39	1.42
Chloride (mass % maximum)	Pre-stressed 0.01 Reinforced 0.03 Non-reinforced 0.08		2.66×10^{-4}		1.26×10^{-4}
Soluble Deleterious Impurities	85% minimum		85*		82*

* Interpolated result. Concrete strength comparisons on unwashed samples after 10 days and washed samples after 18 days.

Specifications according to SANS 1083 (2006).

The average particle size distribution of the sand, from samples S1, S2, S3 and S4, was assessed to determine whether the sand will be a suitable filter material to achieve sufficient drainage control from the base clay material and to prevent internal soil movement. Guidelines published by the Federal Emergency Management Agency (FEMA), an agency of the United States Department of Homeland Security, were used to assess the suitability of the sand material to act as a filter drain.

The results of the assessment are illustrated in Figure 4-2. The base soil was taken as a mixed sample obtained from test pits C2, C4a, C7, C8 and C10. The average sand particle size distribution is shown in Figure 4-2 as the purple line. Based on the clayey base soil material, the sand filter is required to fall within the lower and upper limits of particle size distribution shown by the red and blue lines.

Figure 4-2 indicates that the sand falls outside of the lower limit set by the FEMA design guidelines at approximately 60 % passing the 0.4 mm sieve size.

This implies a need for further filter design during the detailed design stage entailing blending with coarser material to bring the filter grading within the upper and lower limits.

Estimated reserves within the area investigated are approximately 130 000 m³, but visually the actual feasibly exploitable reserves in the Tsitsa River, available within the impoundment basin and within economic haulage distance of the dam, will be far in excess of this and will easily meet the maximum required volume of 200 000 m³ of sand for a RCC dam alternative, given in Table 4-1.

If suitable regional commercial sources are not found to be economic at the detailed design or construction stages, blending with crusher sand could also be considered.

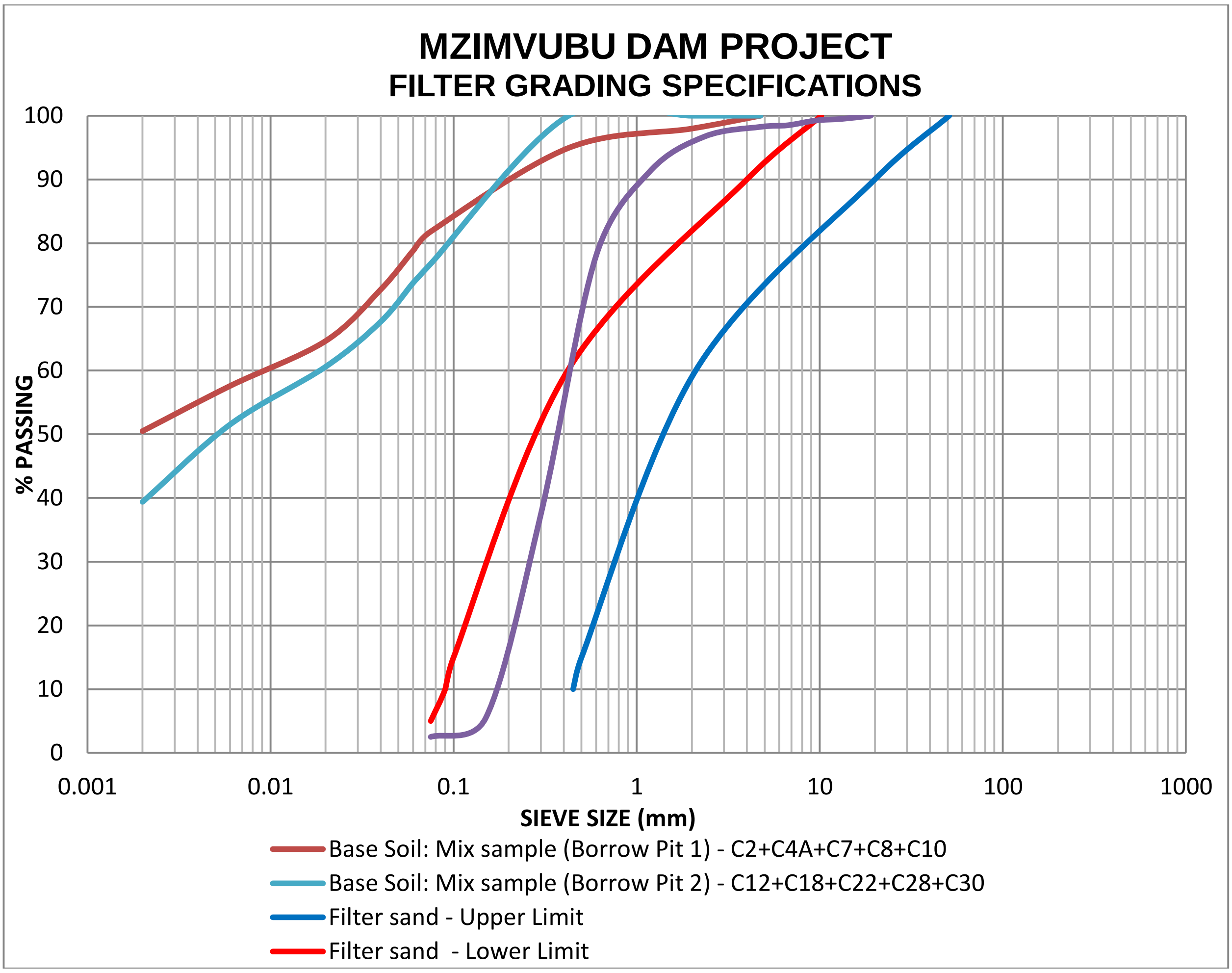


Figure 4-2: Filter Material Grading Specifications (FEMA Design Guidelines)

4.3 Core Material

Reddish brown, clayey hill-wash deposits generally of doleritic origin, but also associated with mudrock occur in relative abundance throughout the project area. The investigation targeted two areas cross-hatched blue on Figure 4-1, both within the impoundment basin and within close haulage distance to the dam. The combined estimated volume of core material available from the two areas is approximately 220 000 m³. The laboratory test results on the materials sampled are presented in Appendix H.

At the time that the investigation was undertaken dam type and material requirements had not been decided upon. As a result, whilst this figure is less than the volumes indicated in Table 4-1 of 260 000 m³ required for an ECRD and 500 000 m³ for an earth embankment dam, other areas identified during the reconnaissance are expected to more than triple the volume proved above.

In addition, small amounts of core quality material can be procured from the dam foundation excavations on the lower flanks. The procurement of adequate volumes of suitable core material from within the impoundment basin for any of the dam alternatives is not expected to present a problem.

Specifications for core material have been extracted from the literature [Elges *et al* (1994)*, Melvill (2002)**, and Mouton (2010)***] and are presented in Table 4-4.

Table 4-4: Specification for Core Material

Specification for Core Material	
Material Property	**Specification**
% Passing 4.75mm	90 – 100 (***)
% Passing 0.425mm	60 – 100 (*)
% Passing 0.002mm (Clay Content)	10 – 30 (*) 10 – 40 (**)
Liquid Limit	25 – 60% (*)
Plasticity Index	10 – 30 (*)
Linear Shrinkage	6 – 14% (*)
Standard Proctor Optimum Moisture Content	12 – 25% (*)
Standard Proctor Maximum Dry Density	1350 – 1700kg/m^3 (*)
Friction angle (saturated, drained triaxial)	20 - 30° (*)
Cohesion (saturated, drained triaxial)	15 – 24kPa (*)
Permeability	1 x 10^{-9}m/s (*)
Organic Content	<2% (**)

4.3.1 Core Borrow Pit 1

Core Borrow Pit 1 is located about 600 m to the south of the dam and on the southern side of the Tsitsa River. It is therefore proposed as the core source to mainly serve the right flank embankment construction.

The area investigated occupies the lower slope of an extensive N-S trending dolerite spur. The underlying bedrock is mudrock, but the material investigated comprises colluvial, hillwash deposits derived mainly from the dolerite on the upper hill-slope. The area of exploitation can be increased significantly by extending in an easterly direction up the slope to the full supply level contour.

Test results on the sampled materials are summarised in Table 4-5.

The materials tested from Core Borrow Pit 1 generally comply with the specification parameters in Table 4-4, although clay content is generally higher than the maximum limit specified, but the material is generally non-expansive and will not present workability problems during construction nor will it be susceptible to heave, although linear shrinkage values above 8 indicated a tendency for shrinkage on drying out.

A further suitability evaluation, according to the USBR (1974), is based upon the relationship between liquid limit and plasticity index. Figure 4-3 shows the samples tested from Core Borrow Pit 1 plotted on the Casagrande Plasticity Chart. Samples that plot within the elliptical area are considered suitable for use as core. Figure 1 confirms that all the samples tested from Core Borrow Pit 1 plot within the suitability zone and may therefore be considered suitable for use as core.

Table 4-5: Summary of Core Borrow Pit 1 Test Results

Summary of Core Borrow Pit 1 Test Results											
Property	**Trial Pit No.**										
	C1A	**C2**	**C2A**	**C4A**	**C5**	**C6**	**C7**	**C8**	**C9**	**C10**	**Mix**
% Passing 4.75mm	94	100	100	100	98	96	99	100	98	99	99
% Passing 0.425mm	85	100	99	100	95	85	95	97	94	91	94
% Passing 0.002mm	46	47	41	71	38	37	50	36	52	40	50
Liquid Limit	41	52	35	49	39	42	41	44	48	37	46
Plasticity Index	13	23	19	20	20	20	23	20	26	20	23
Linear Shrinkage	9.5	10.5	9.5	11.5	10	9.5	13.5	11.5	12.5	10	11.5
Pot. Expansiveness	L	L	L	L	L	L	L	M	L	L	L
Permeability (m/s)#											$1x10^{-9}$
Dispersion (dbl hydrm)		28		9			9	27			34
Dispersion (pinhole) #											ND3
Friction angle #											26.6
Cohesion #											15.1

Note:
Mix is a mixed sample comprising equal proportions of C2, C4A, C7, C8, and C10.
Pot. Expansiveness is the potential expansiveness according to van der Merwe (1964), where L is low and M is medium.
Superscript# indicates testing on a sample remoulded to 98% of Standard Proctor maximum dry density.

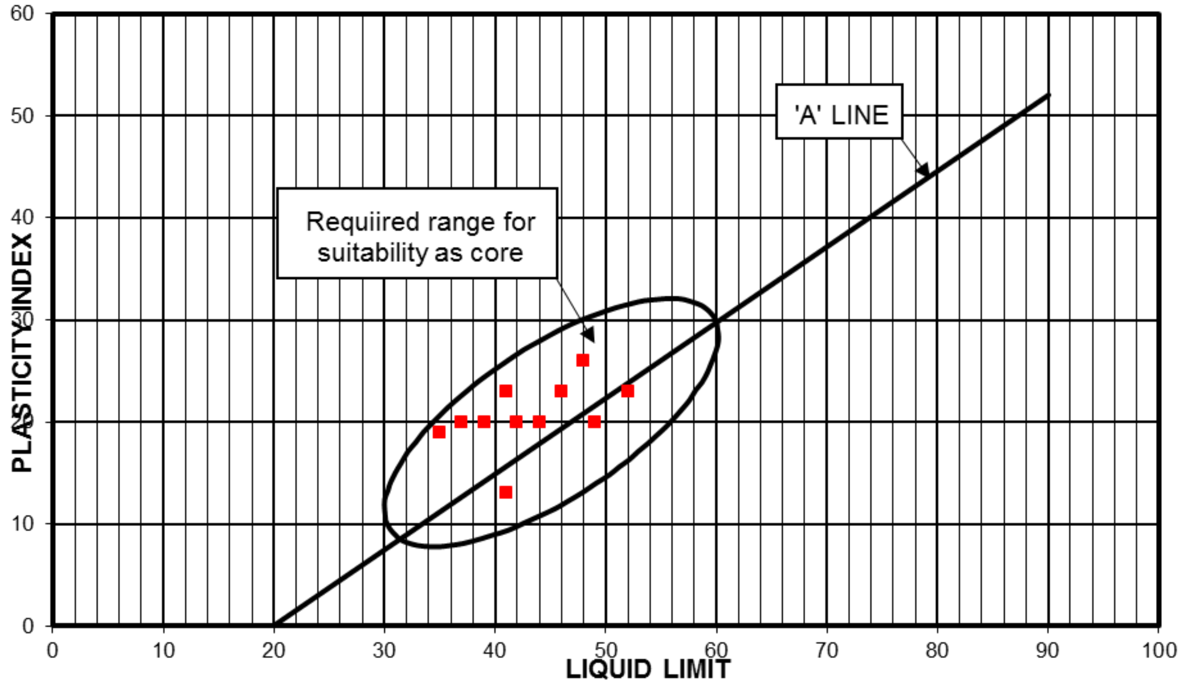


Figure 4-3: Plasticity Chart for Core Borrow Pit 1

4.3.2 Core Borrow Pit 2

Core Borrow Pit 2 is located about 1.2 km west south of the dam and on the northern side of the Tsitsa River. It has therefore been proposed as a core source to serve the left flank embankment construction. The area investigated extends from a low lying area adjacent to a tributary river of the Tsitsa, eastwards onto a southerly facing spur. The underlying bedrock geology comprises both dolerite and mudrock. The material investigated comprises colluvial deposits and residual to weathered mudrock.

Test results on the sampled materials are summarised in Table 4-6.

Table 4-6: Summary of Core Borrow Pit 2 Test Results

Summary of Core Borrow Pit 2 Test Results											
Property	**Trial Pit No.**										
	C12	**C15**	**C16**	**C18**	**C20**	**C22**	**C23**	**C25**	**C28**	**C30**	**Mix**
% Passing 4.75mm	100	98	99	100	100	100	97	99	100	99	100
% Passing 0.425mm	95	93	95	98	98	100	93	93	100	98	100
% Passing 0.002mm	50	18	34	32	37	42	39	41	41	34	39
Liquid Limit	37	34	35	22	33	59	36	53	61	37	43
Plasticity Index	17	12	15	10	16	25	16	22	27	19	21
Linear Shrinkage	10	7.5	7.5	5.5	8	10.5	8.5	10.5	15.5	8	11.5
Pot. Expansiveness	L	L	L	L	L	H	L	L	H	M	M
Permeability (m/s)#		-	-	-	-	-	-	-	-	-	8.4×10^{-9}
Dispersion (double hydrometer)	14	-	-	31	-	-	-	-	31	46	25
Dispersion (pinhole) #	-	-	-	-	-	-	-	-	-	-	ND3
Friction angle #	-	-	-	-	-	-	-	-	-	-	27.3
Cohesion #	-	-	-	-	-	-	-	-	-	-	0.2

Note:
Mix is a mixed sample comprising equal proportions of C1, C18, C22, C28 and C30
Pot. Expansiveness is potential expansiveness according to van der Merwe (1964), where L is low and M is medium.
Superscript# indicates testing on a sample remoulded to 98% of Standard Proctor maximum dry density.

The results of the tests undertaken on the materials sampled from Core Borrow Pit 2 indicate a higher degree of variability than those for Core Borrow Pit 1. Most of the samples are in compliance with the specification parameters in Table 4-4, but in the case of C18 the material is of insufficient plasticity, whereas in the case of C22, C28 and C30 plasticity is too high and the materials are potentially expansive.

On the Plasticity Chart (Figure 4-4) samples C18 and C28 plot outside of the suitability zone for core. The mixed sample plots in the centre of the suitability zone.

The majority of the materials tested are non-expansive, although C22 and C28 are highly expansive and C30 and the mixed sample are moderately expansive. Core Borrow Pit 2 is suitable for use as core, but the materials will require more selective usage and processing than the materials from Core Borrow Pit 1.

The effective cohesion of the mixed sample of Core Borrow pit 2 is significantly lower than the effective cohesion determined from the mixed sample of Core Borrow pit 1.

In order to interpret the result correctly it should be understood that the cohesion parameter *c'* is only a mathematical line-fitting constant used to model peak states, and should not be used to imply that a material has a shear strength at zero normal effective stress. Once soils have been sheared the effect of cohesion are destroyed. Therefore the effective cohesion are of no importance in design as critical state conditions are assumed.

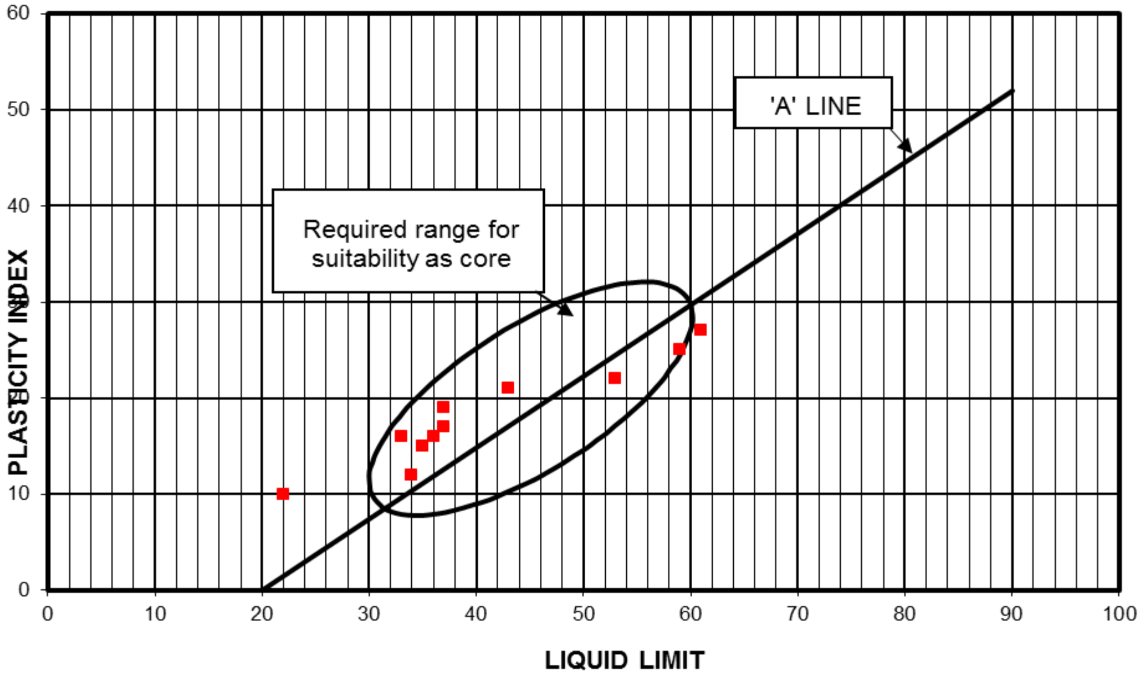


Figure 4-4: Plasticity Chart for Core Borrow Pit 2

4.4 Embankment Fill (Shell) Material

Sedimentary rocks comprising purple mudrock with subordinate thin intercalations of sandstone and siltstone occur in abundance in the impoundment basin. Two areas were investigated within the impoundment basin, cross-hatched orange on Figure 4-1. The laboratory test results on the samples retrieved in the investigation are presented in Appendix H.

Whilst the investigation undertaken, using a TLB did not prove the required 2 million m³ required for an earth embankment dam, the use of more powerful excavating equipment and extending the area of exploitation into adjacent areas with visible evidence of mudrock and sandstone would easily provide the shell requirements for earth embankment dam construction. Specifications for shell material, according to Elges *et al* (1994) and Mouton (2010) are presented in Table 4-7.

4.4.1 Fill Borrow Pit 1

Fill Borrow Pit 1 is located on the southern side of the Tsitsa River, approximately 2.6 km directly WNW of the dam. The materials comprise predominantly weathered mudrock with intercalated sandstone layers. This material occurs in abundance within the future impoundment basin and well beyond the area investigated.

The test results on the sampled materials are summarised in Table 4-8.

Table 4-7: Specification for Shell Material

Specifications for Shell Material		
Material Property	**Specification**	
	Semi-Pervious	**Pervious**
% Passing 4.75mm	60 - 100	-
% Passing 0.425mm	30 - 100	Min 20
% Passing 0.002mm (Clay Content)	<25	<20
Liquid Limit	<25%	<20%
Plasticity Index	<10	<5
Linear Shrinkage	<5%	<2%
Standard Proctor Optimum Moisture Content	10 – 15%	8 – 12%
Standard Proctor Maximum Dry Density	1600 – 1850kg/m³	1700 – 2000kg/m³
Friction angle (saturated, drained triaxial)	30 – 35°	>35°
Cohesion (saturated, drained triaxial)	10 – 15kPa	<10kPa
Permeability	1 x 10^{-7}m/s	1 x 10^{-5}m/s

Table 4-8: Summary of Fill Borrow Pit 1 Test results

Summary of Fill Borrow Pit 1 Test Results										
Property	**Trial Pit No.**									
	F25	**F28**	**F29**	**F31**	**F33**	**F34**	**F37**	**F37A**	**F39**	**Mix**
% Passing 4.75mm	82	96	94	76	75	85	85	99	73	44
% Passing 0.425mm	61	57	45	38	25	55	59	86	40	18
% Passing 0.002mm	15	9	13	5	10	8	12	18	10	4
Liquid Limit	23	23	25	20	31	21	22	27	23	23
Plasticity Index	10	6	9	5	14	6	7	12	9	8
Linear Shrinkage	4	4	6	3.5	7.5	4	4	6	5.5	5.5
Potential Expansiveness	L	L	L	L	L	L	L	L	L	L
Permeability (m/s)#	-	-	-	-	-	-	-	-	-	2.6x10^{-9}
Dispersion (double hydrometer)	-	-	-	-	-	-	-	48	-	
Proctor MDD	-	-	-	-	-	-	-	-	-	2069
Proctor OMC	-	-	-	-	-	-	-	-	-	11.3
Friction angle #	-	-	-	-	-	-	-	-	-	32.3
Cohesion #	-	-	-	-	-	-	-	-	-	6.9

Note:
Mix is a mixed sample comprising equal proportions of F25, F29, F34, F37 and F39.
Sample F37A is overburden and was not tested to evaluate its suitability as shell construction material, but rather determine the potential dispersiveness of the colluvial sediments.

Table 4-8 indicates that the materials are marginally suitable for use as semi-pervious fill. They tend to break down under compaction, which impairs their grading, shear strength and renders them insufficiently permeability for semi-pervious and pervious fill.

It is apparent that the results for the mix sample are incorrect as the percentage of particles passing the 4.75 mm and 0.425 mm are lesser compared to the samples it comprises of.

The Proctor MDD does however fall in the range of a highly weathered, soft rock, mudrock.

4.4.2 Fill Borrow Pit 2

Fill Borrow Pit 2 is located on the northern side of the Tsitsa River and approximately 1 km to the north west of the dam. The materials comprise mudrock with interlayered sandstone.

Test results on the materials are summarised in Table 4-9.

Again the materials are insufficiently pervious for use as semi-pervious and pervious fill. It is apparent that the results for the mix sample is incorrect as the percentage of particles passing the 4.75mm and 0.425mm are lesser compared to the samples it comprises of. The Proctor MDD does however fall in the range of a highly weathered, soft rock, mudrock.

Table 4-9: Summary of Fill Borrow Pit 2 Test Results

Summary of Fill Borrow Pit 2 Test Results							
Property	**Trial Pit No.**						
	F5	**F6**	**F8**	**F10**	**F12**	**F13**	**Mix**
% Passing 4.75mm	90	71	87	81	71	79	45
% Passing 0.425mm	42	36	80	39	43	59	30
% Passing 0.002mm	12	12	10	15	5	8	6
Liquid Limit	25	31	21	33	-	23	22
Plasticity Index	11	16	9	16	-	9	8
Linear Shrinkage	5.5	6.5	3.5	8.5	-	4	4
Potential Expansiveness	L	L	L	L	L	L	L
Permeability (m/s)[#]	-	-	-	-	-	-	2.1×10^{-9}
Proctor MDD	-	-	-	-	-	-	2057
Proctor OMC	-	-	-	-	-	-	10.5
Friction angle [#]	-	-	-	-	-	-	27.6
Cohesion [#]	-	-	-	-	-	-	14.1

Note:
Mix is a mixed sample comprising equal proportions of F5, F8 and F10

4.5 Water

Water was sampled from the Tsitsa River immediately upstream of the dam. The laboratory test results are presented in Appendix H9 and are summarised in Table 4-10.

Table 4-10: Summary of Test Results on Tsitsa River Water

Summary of Test Results on Tsitsa River Water		
Property	**Specification**	**Tsitsa River Water**
Total Dissolved Solids (mg/l)	2000 max	105.5
Chloride Content (mg/l)	500 max	5.3
Calcium Hardness as $CaCO_3$ (mg/l)	400 max	101.6
pH	6 – 8	7.75
Sulphate Content (mg/l)	1000 max	Not detected
7 Day Comparative Cube Strength (as % of control sample)	90% min	105%

Note: Specifications according to the Portland Cement Institute, 1994.

The results indicate that the water is suitable for use in the manufacture of concrete.

4.6 Conclusion of Materials Availability for Dam Alternatives

The various material requirements for the alternative dam types are tabulated in Table 4-1.

4.6.1 Roller Compacted Concrete Dam

Rock and sand occur in abundance within the future impoundment basin and the availability of suitable construction materials presents no constraints to the construction of an RCC dam.

4.6.2 Concrete Faced Rock-fill Dam

As for 4.6.1, there are large reserves of rock and sand in the future dam basin and again the availability of construction materials imposes no constraints on the construction of a CFRD alternative.

4.6.3 Earth Core Rock-fill Dam

As previously mentioned rock and sand are freely available from within the basin. Whilst the investigations undertaken did not prove the required volume of core for an ECRD, extending Core Borrow Pit 1 and exploiting other areas within the basin where the presence of suitable core material was visually apparent, is envisaged to provide far in excess of the required volume of good quality core material. The availability of suitable construction materials is not expected to present a constraint to the construction of an ECRD alternative.

4.6.4 Earth Embankment Dam

Rock and sand is readily available and will meet the volume requirements for an earth-fill dam. The investigations undertaken did not prove the required 500 000 m³ of core material, but visually suitable core material of sufficient quantity appears to be available from within the basin and in the general project area, within relatively close proximity to the dam. The shell requirements for an earth embankment dam are of the order of 2.1 million m³. Sedimentary rocks comprising mainly mudrock with intercalated sandstone are widely distributed within the basin and were tested for suitability as embankment shell.

The materials tested for possible use as shell are considered unsuitable for use as pervious fill and marginally suitable for use as semi-pervious fill. Consideration could be given to the investigation of extensive sandstone deposits in the surrounding hills or weathered dolerite, but these occur outside of the future impoundment basin and the exploitation of the large quantities required would lead to significant environmental impacts. The paucity of suitable shell material within the basin is viewed as a constraint to the construction of an earth-fill dam alternative.

4.6.5 Site Investigations and Materials Requirements Conclusion

The conclusions drawn are that the founding conditions at the dam site and the materials availability within the impoundment basin would be suitable for the construction of most of the alternative dam types mentioned above. The exception is the earth fill option for which large quantities of embankment shell material would have to be sourced from outside of the basin, with significant haulage cost and potential environmental impacts.

Further site and materials investigations will be required to properly inform the detailed design process. A draft scope of work has been prepared for DWS, and is included in Appendix C of the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/521/12.

5. GEOTECHNICAL APPRAISAL

5.1 Dam Foundations and Cut-off

The dam foundation excavation profile for the cut-off is based upon the results of the rotary core drilling and seismic refraction survey. This produces a cut-off trench terminating in a rock hardness category of medium hard to hard rock and equating to a seismic velocity of about 1 500 m/s. This places the foundation in an intermediate to hard excavation category and approximately equivalent to the rippablilty limits of a D9 bulldozer.

Blasting in the excavation of the cut-off trench is not recommended, unless absolutely necessary, to avoid excess blast fracturing that could compromise the integrity of the foundation rock. All foundation excavations must be verified by a geotechnical professional during construction.

The longitudinal profile for Line 1 on Figure 5-1 indicates an excavation depth of about 8 m on the upper left flank decreasing to about 6m on the lower left flank, about 4 m through the river section to again about 6 m on the lower right flank, decreasing to about 1 m on the mid to upper right flank. It is recommended that the profile is amended as more drilling information becomes available during the detailed design investigations. As mentioned above, excavation depths and foundation treatment must be verified during construction.

The excavation profile for Line 2, as indicated on Figure 5-2, is significantly deeper and justifies the decision to favour the Line 1 alignment.

In the case of a RCC dam type the following foundation criteria are recommended (van den Berg and Parrock, 2009).

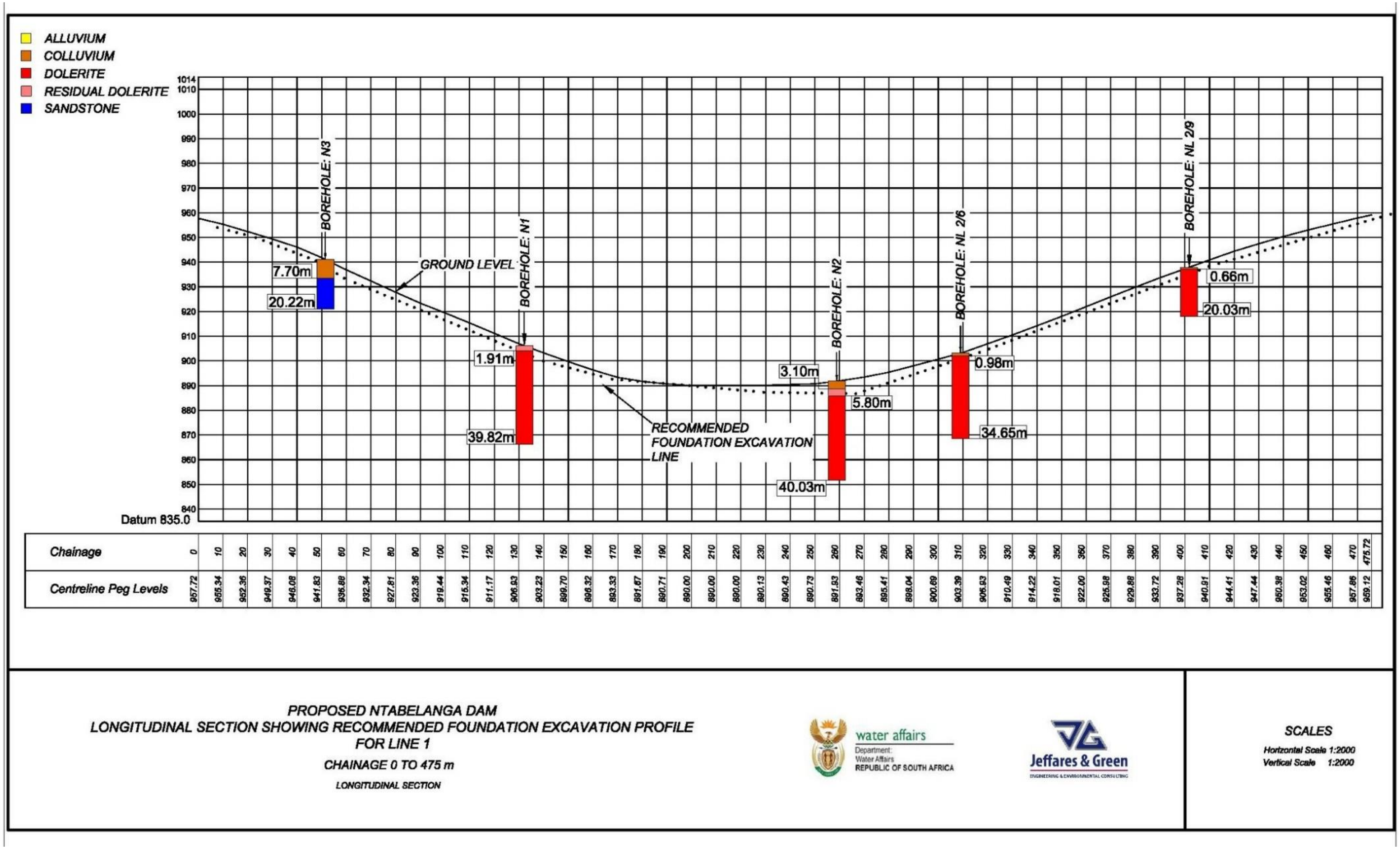
Table 5-1: RCC Foundation Design Criteria

RCC Foundation Design Criteria						
E_{mod} (GPa)	**RMR**	**Weathering**	**UCS**	**RQD**	**Joint Spacing**	**Joint Condition**
>4.5GPa	>40	Medium to Slightly Weathered	>20MPa	>30%	>300mm	Rough, Unaltered

The above criteria are based upon founding in a rock hardness of medium hard to hard rock, complying with the parameters recommended in Table 5-1 and the 1 500 m/s to 2 000 m/s seismic velocity profile. This would place the foundation in an intermediate to hard excavation category and it is likely that blasting would be required to achieve excavation to good quality foundation rock.

Blasting must be minimised in order to avoid excessive blast fracturing, which would compromise the integrity of the foundation rock and van Schalkwyk *et al*, 2009, recommend that bulk blasting be terminated at least 1m above the expected foundation level, proceeding below this by means of controlled blasting or the use of powerful excavating equipment.

As the foundation is the key element of the dam design it is recommended that the excavation profile is amended as more drilling information becomes available during detailed design investigations and that excavation depths and foundation treatment are verified by a geotechnical professional during construction.



Chainage	0	10	20	30	40	50	60	70	80	90	100	110	120	130	140	150
Centreline Peg Levels	957.72	955.34	952.36	949.37	946.08	941.83	936.88	932.34	927.81	923.36	919.44	915.34	911.17	906.93	903.23	899.70

Chainage	160	170	180	190	200	210	220	230	240	250	260	270	280	290	300	310
Centreline Peg Levels	896.32	893.33	891.67	890.71	890.00	890.00	890.00	890.13	890.43	890.73	891.93	893.46	895.41	898.04	900.69	903.39

Chainage	320	330	340	350	360	370	380	390	400	410	420	430	440	450	460	470	475.72
Centreline Peg Levels	906.93	910.49	914.22	918.01	922.00	925.98	929.88	933.72	937.28	940.91	944.41	947.44	950.38	953.02	955.46	957.86	959.12

Figure 5-1: Line 1 – Core Log Summary and Recommended Foundation Profile

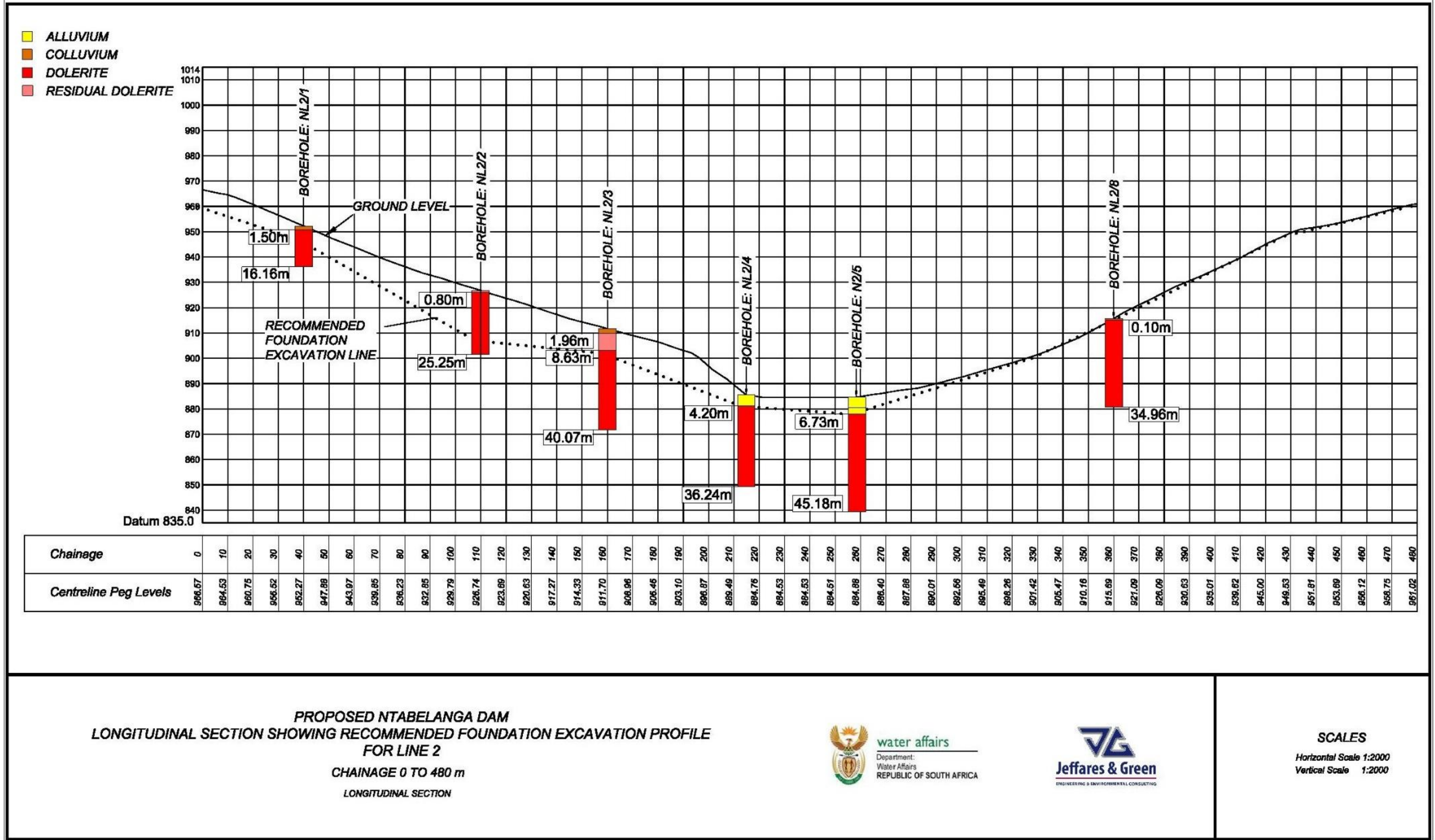


Figure 5-2: Line 2 – Core Log Summary and Recommended Foundation Profile

5.2 Preliminary Stability Assessment

This was undertaken primarily to determine if there were any fatal flaws with the use of the materials found in the vicinity of the proposed dam site, for any of the dam types investigated. Having passed the fatal flaw screening, the stability analysis was then used to determine viable embankment slopes and cross-sectional materials components so that accurate quantities and cost estimates could be estimated for each dam type.

5.2.1 Embankment Stability and Seepage Analyses

As part of both the geotechnical investigation and the dam type analyses, feasibility level assessments of dam stability and seepage were undertaken for the following three possible dam types, earth fill embankment with a clay core (EF), earth core rock fill dam (ECRD) and a concrete faced rock fill (CFRD) dam.

The roller compacted concrete (RCC) dam option has been checked for safety factors against overturning and sliding under SEF conditions, but in the case of seepage analyses of a concrete dam built on competent dolerite, the methodology relates more to the presence of seepage paths through weathered or jointed materials.

In this case, the foundations of the RCC dam are likely to be on competent dolerite, but the amount of jointing can only be determined by undertaking the additional geotechnical investigations recommended for the detailed design stage, and would then be fully dealt with by curtain grouting and drainage.

At the 2004 World Conference on Earthquake Engineering in Vancouver, Paper No. 3399 entitled: *Earthquake Aspects of Roller Compacted Concrete and Concrete-Face Rock fill Dams, by Martin Wieland and R. Peter Brenner* was presented. The conclusion was as follows:

"The main disadvantages (of RCC) are the following:

i) Water tightness: Due to the construction of the dam in thin horizontal layers, in the case of high hydraulic gradients, water may percolate along the horizontal construction interfaces. Special measures may be needed at the upstream face of the dam to improve the water tightness, i.e. layer of high paste monolithic mass concrete or a surface sealing by a geomembrane.

ii) Limited experience of engineers and contractors: Few designers and contractors have extensive experience with the design and construction of RCC dams. The design and construction practice are still in development. It should be noted that, at this feasibility study level, these analyses were undertaken with the main objective of determining if there are any fatal flaws with the use of the materials as found in the vicinity of the proposed dam site, for any of the dam types investigated, as well as determining the cross-sectional shape of the dam embankments for feasibility design purposes.

iii) Limited experience with safety and long-term performance: No large RCC dam has been exposed to extreme loadings like strong ground shaking during an earthquake or large floods.

iv) Galleries: Placement of RCC around formwork, which is needed for access galleries in the dam body, is tedious and slows down the construction process.

The main weaknesses of RCC dams are the water tightness under high hydraulic gradients, ageing mechanisms and the unknown performance under seismic loading."

The intervening years have shown an upsurge in the construction of several large RCC dams around the world, as well as significant research into overcoming the perceived disadvantages listed above.

More experience has been gained by engineers and contractors in this period, (including the DWS in-house design and construction divisions themselves), and improvements in RCC construction methodology, resilience to earthquake stresses and movement, mix design, and special treatment of surfaces to improve water tightness, have all combined to improve the confidence in RCC as a dam type, as recently demonstrated at the De Hoop dam, Spring Grove dam in South Africa, and Metolong dam in Lesotho.

The stability analysis of the roller compacted concrete (RCC) dam option was also undertaken is discussed in more detail in the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/521/12.

The stability and leakage analyses undertaken on other dam types have made use of the available information on the geotechnical properties of the available materials, as has been derived through the geotechnical investigations, but should be reviewed again with a more in-depth analyses as more information becomes available during the detailed design phase.

For all dam types it has been assumed that the foundations would be grouted. Grouting quantities have been adjusted to take into account the likely requirements of each dam type, which have different seepage cut-off arrangements.

The stability scenarios that have been analysed are:

a) Rapid Drawdown

This is when the reservoir level is rapidly reduced from the Full Supply Level (FSL) to the minimum operating level, and is generally only used in an emergency case when there may be some initial signs of failure or distress to the embankment.

It is not possible to ‘instantaneously drawdown the reservoir level as the outlet works would usually be designed to empty the dam over a period of 4 weeks. In terms of stability, rapid drawdown (RDD) is deemed to be a critical case, as it is assumed that with the rapid reduction in reservoir water level pore water pressures within the upstream shoulder of the embankment do not have sufficient time to dissipate, yet the shoulder loses the support in terms of loading of the reservoir water itself.

b) Seismic Event

An earthquake event would cause cyclic dynamic loading of the embankment, predominantly in the horizontal direction and may cause damage to the embankment but must not cause a total failure of the dam.

According to the seismic hazard map published in 2003 by the South African Council for Geoscience, Figure 5-3 (contained in draft SANS 10160-4), a peak horizontal ground acceleration of 50-100 cm/s^2 has been recorded, with a 10% probability of this being exceeded at least once in a 50 year period.

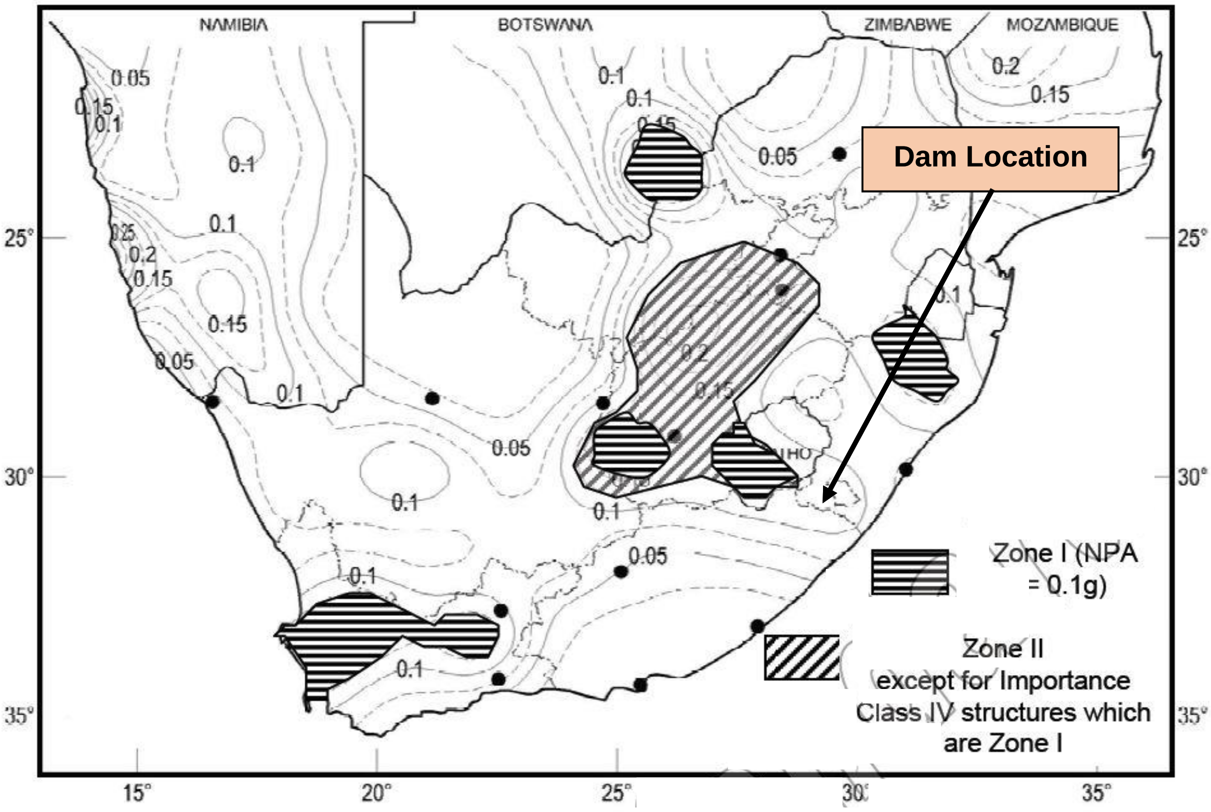


Figure 5-3: Peak horizontal ground acceleration

Taking only this guideline into consideration, this would be considered as a low risk zone, and a value of 0.1g would therefore have normally be applied as a horizontal loading in the design of the embankment.

Prof Andrzej Kijko, the Director of the University of Pretoria Natural Hazard Centre, was assigned to perform a detailed earthquake hazard assessment. From the research undertaken, indications are that there have been some historical earthquake events in the area of influence of this dam, which could merit the consideration of analysis using higher risk factors than those published in SANS 10160-4,2009.

The report and results of the above seismicity study are included as Appendix D in the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/521/12.

A short extract of the findings of this study is as follows:

"For frequency of ground motion exceeding 1 Hz, the analysis used 1,574 records from 58 earthquakes in the distance range of 0 km to 400 km. (Boore and Atkinson, 2008).

The probabilistic seismic hazard analysis (PSHA) was performed using conventional, Cornell-McGuire procedure (Cornell, 1968; McGuire, 1976; 1978), where the integration across the uncertainty in the peak ground acceleration (PGA) prediction equation is an integral part of the methodology.

In accordance to the current seismic guidelines such as Euro code 8 (2004) and ASCE (2005), three seismic design levels were considered:

- *Operating Basis Earthquake (OBE),*
- *Maximum Design Earthquake (MDE), and*
- *Maximum Credible Earthquake (MCE).*

Given the existence of 594 tectonic faults in the vicinity of the dam site (information provided by Jeffares & Green (Pty) Ltd), an investigation of the effect of potential seismic activity of the faults on the seismic hazard assessment was performed.

The results of the PSHA are given in terms of mean return periods and probabilities of being exceeded for horizontal component of the PGA.

Based on the logic tree formalism, the expected values of horizontal component of OBE, MDE and MCE for the site of Mzimvubu Dam, Eastern Cape are:

- *OBE (Return Period 144 years): 0.018 ± 0.003 g*
- *MDE (Return Period 475 years): 0.039 ± 0.012 g*
- *MCE (Return Period 10,000 years): 0.159 ± 0.043 g*

According to the applied guidelines, the site of the future dam is rated as low risk."

Even though the results of this special study indicate a low risk rating, a conservative approach has been taken and the embankment stability analyses have been undertaken for accelerations of both 0.10g and 0.15g. The analyses indicate that the different dam types will not fail as a result of a 0.15g earthquake loading. The results of these analyses, undertaken with the SLIDE software, are presented below.

c) Liquefaction

This is a loss of shear strength due to increased pore pressures caused by an earthquake. It can lead to catastrophic failure of embankments. Soils most susceptible to liquefaction are saturated sands, silty sands and gravelly sands.

Cyclic loading tends to cause densification of granular soils, just like compaction. However, the phenomenon of liquefaction occurs in certain saturated soils because they are not sufficiently permeable to allow drainage during cyclic loading. They do not allow a decrease in volume, and the tendency to decrease volume is counteracted by an increase in pore pressure with associated reduction in effective stress. The pore pressures gradually build up to equal the total stress and then a state of zero effective stress, or liquefaction, occurs.

Loose materials are more susceptible than dense materials. Materials with less than 5% fines are also thought to be more susceptible to liquefaction. An increase in fines reduces susceptibility.

Liquefaction of the embankment and foundation at Ntabelanga is unlikely given the density and physical properties of the construction materials in question, and the low seismicity of the region.

d) End of Construction

For embankment dams, the end of construction case can often be critical, as pore pressures in the lower half of an earth embankment rise with the additional loading of fill material as it is being placed.

Over time these pore pressures will dissipate but if the embankment is raised too quickly the build-up in pore pressure can result in a lowering of the effective strength of the materials and can lead to a failure.

e) First Filling

This analysis investigated the stability of the upstream shoulder during first filling of the reservoir, which would be undertaken shortly after the end of the construction phase. If done too quickly pore pressures in the embankment may not have had time to dissipate and could result in lower effective strengths, as for the end of construction phase. A major storm could potentially effect rapid filling in a matter of hours.

f) Full Supply Level

This was the first case to be checked, where the reservoir level is at its maximum operating level, and a steady state seepage condition exists within the embankment.

The recommended minimum factors of safety for each case analysis are presented in Table 5-2 below:

Table 5-2: Recommended Factor of Safety

Design Condition Analysed	Minimum Acceptable Factors of Safety*
End of construction:	
- downstream slope	1.3
- upstream slope	1.3
***Initial filling*:**	
- upstream slope	1.2
***Steady state seepage*:**	
- downstream slope	1.5
- upstream slope	1.5
***Rapid drawdown*:**	
- upstream slope	1.2
***Steady state seepage plus earthquake*:**	
- downstream slope	1.0
- upstream slope	1.0

** The results of the analysis are expressed as a factor of safety, which is defined as the ratio of available shear strength to that required for equilibrium.*

The slope stability programme *SLIDE version 06*, which is part of the *RocScience Suite* of geotechnical software programmes, was used for the analyses, uses both the Morgenstern-Price and Bishop Limit equilibrium methods.

As discussed earlier, the laboratory test results available for the various construction materials at the time of writing this report were used in this analysis, and more detailed site investigations during the detailed design stage will significantly improve the information available on the materials properties.

Following a precautionary approach, a degree of conservatism has been used in the selection of material properties used in the analyses. Table 5-3 summarises the values used. Critical State conditions were assumed for the Embankment Dam Stability Analyses, therefore the cohesion intercept of the remoulded material zones were reduced to zero or near zero values as indicated in Table 5-3.

Table 5-3: Summary of Material Properties

Material Type	Unit Weight (Kg/m^3)	Cohesion (kPa)	Internal angle of friction (degrees)	Permeability (m/s)
Core Material	1 800	2.0	26	1×10^{-9}
Shoulder Material (earth fill)	2 000	2.0	28	1×10^{-7}
Rock fill	2 200	0.0	42	1×10^{-3}
Alluvium	1 800	5.0	23	1×10^{-7}

It has been assumed that the shear strength of the foundation bedrock exceeds that of all other construction materials and that the embankment is rigidly bounded at this interface on the models. Discussion on the findings for the three dam types analysed follows.

5.2.2 Embankment Dams Stability Analyses Findings

a) Earthfill Embankment with Clay Core

The earth fill embankment with a clay core was analysed with an upstream shoulder at a slope of 1V: 3H, and the downstream shoulder at a slope of 1V: 2.5H. The crest width was6m, and the height of the dam above river bed level was 65 m. The following cases were analysed:

- The upstream and downstream shoulders for full supply level with steady state seepage conditions;
- The upstream and downstream shoulders with a horizontal seismic loading of 0.1g and 0.15g applied; and
- The rapid drawdown case.

The following plots illustrate the failure planes with the minimum Factors of Safety

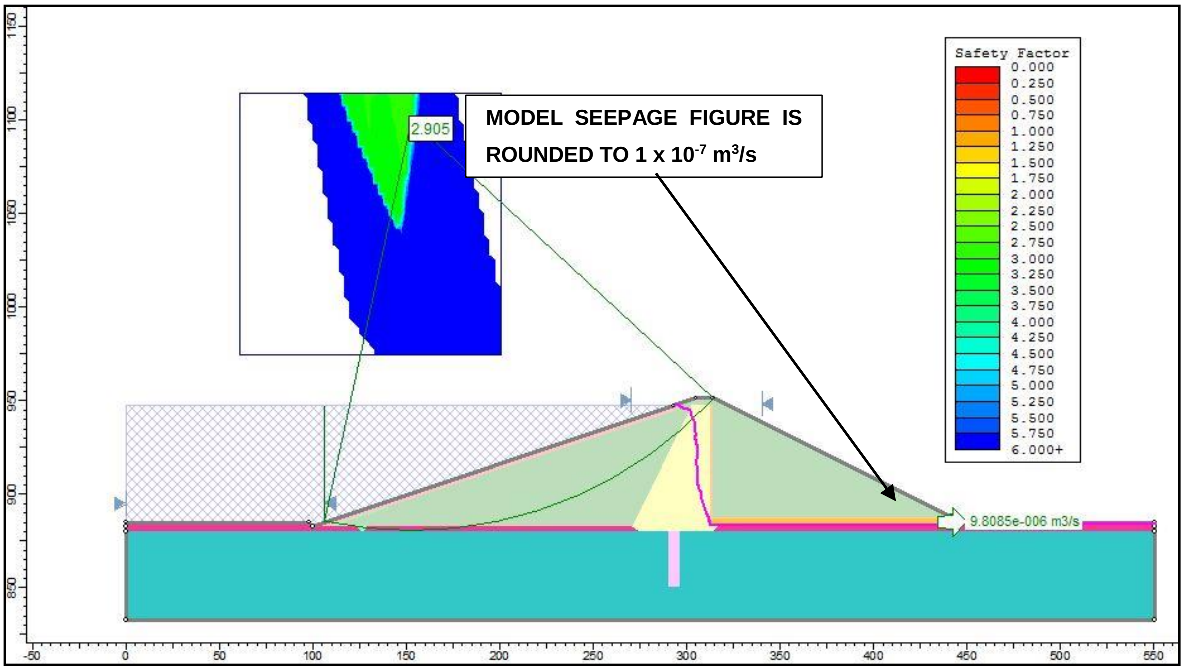


Figure 5-4: Earth fill Embankment: Upstream Shoulder
Steady State Seepage condition at Full Supply Level (FSL): Factor of Safety (FoS) = 2.905

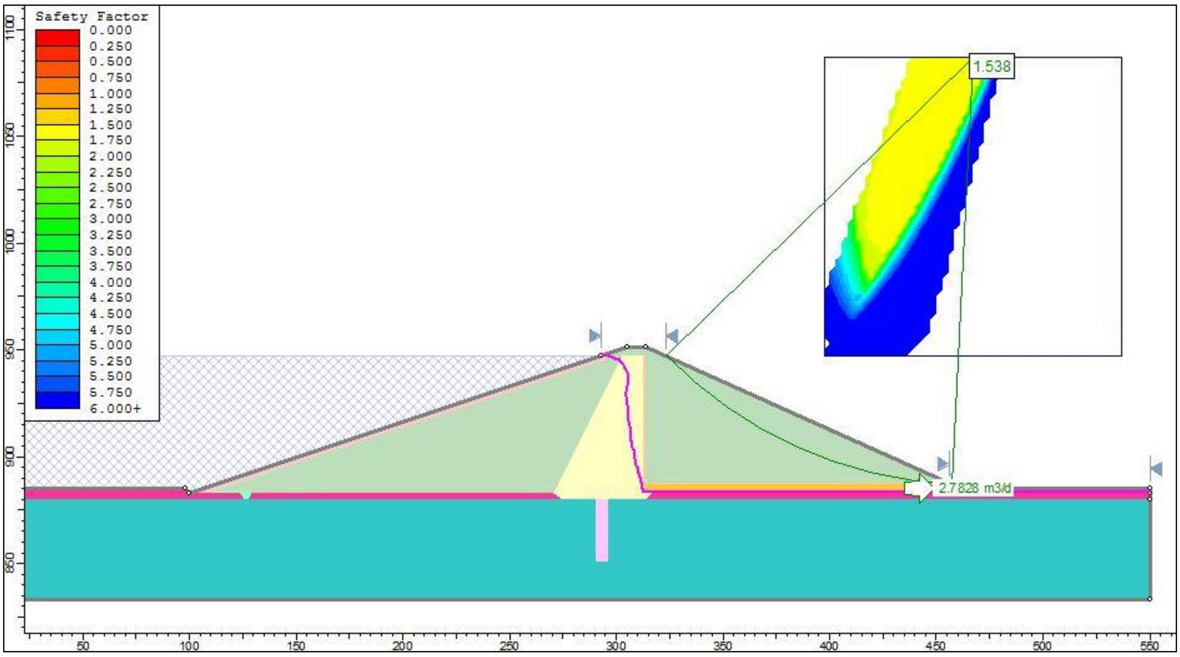


Figure 5-5: Earth fill Embankment: Downstream Shoulder
Steady State Seepage at Full Supply Level (FSL): FoS = 1.538

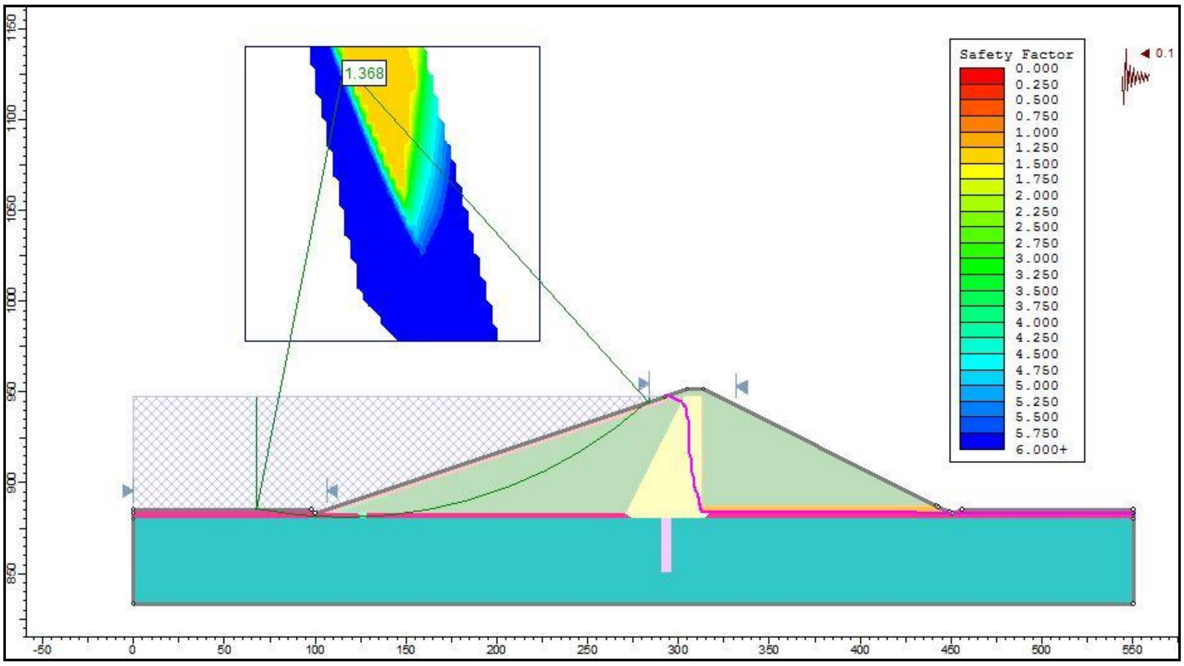


Figure 5-6: Earth fill Embankment: Upstream Shoulder
Full Supply Level with seismic loading 0.1g: FoS = 1.368

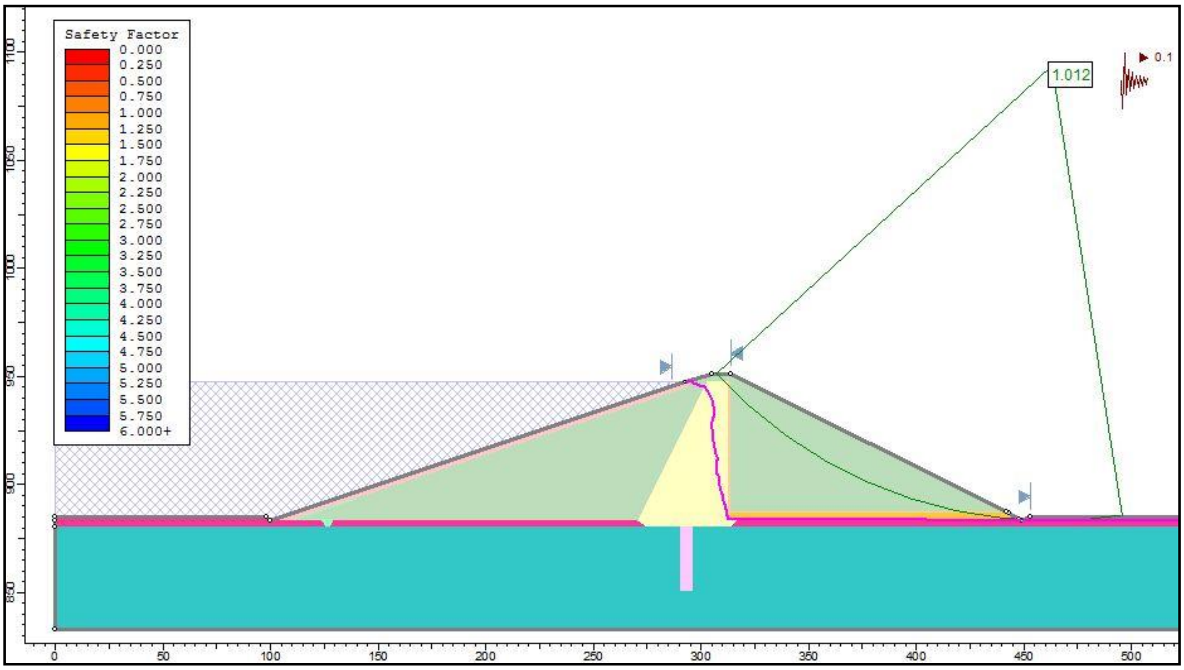


Figure 5-7: Earth fill Embankment: Downstream Shoulder
(Full Supply Level with seismic loading 0.1g: FoS = 1.012)

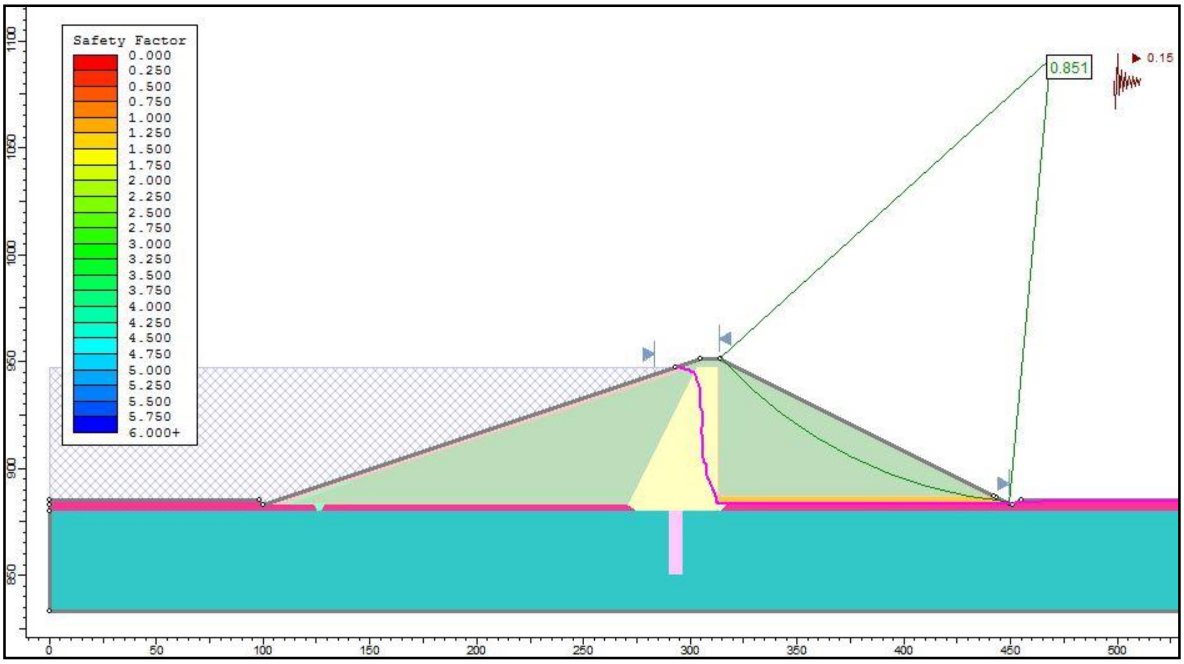


Figure 5-8: Earth fill Embankment: Downstream Shoulder
(Full Supply Level with seismic loading 0.15g: FoS = 0.851)

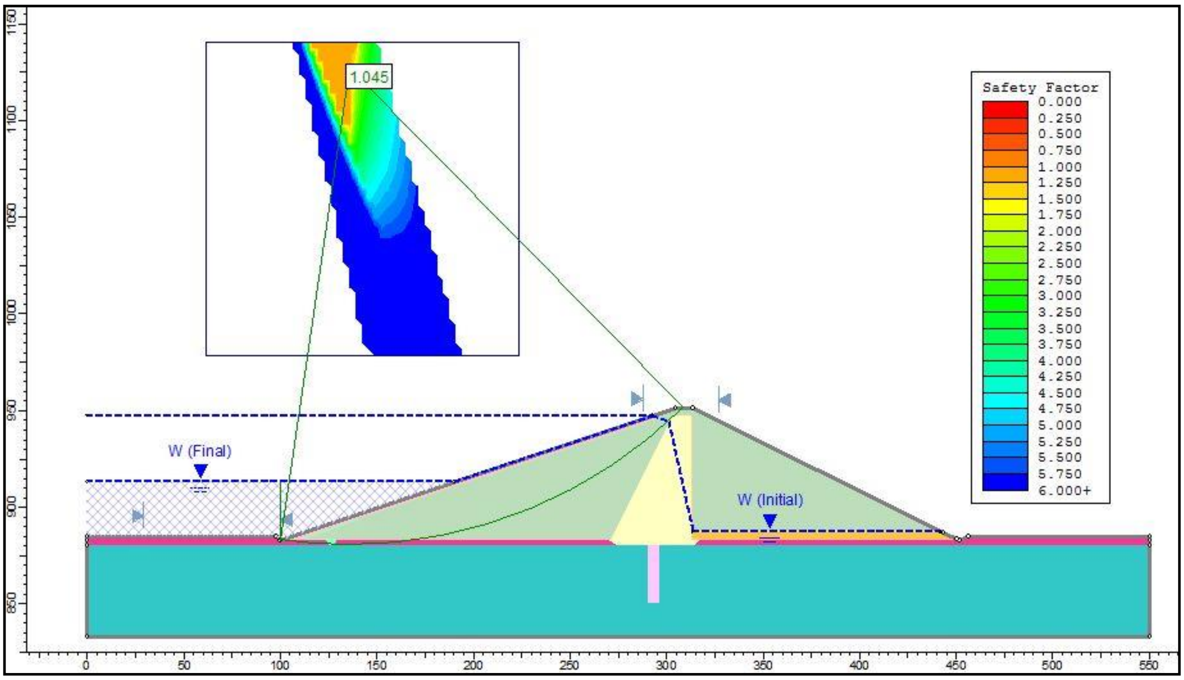


Figure 5-9: Earth fill Embankment: Upstream Shoulder
(Rapid Drawdown (RDD): FoS = 1.045)

b) Earth Core Rockfill Dam

A cross-section was analysed with the following geometry, upstream shoulder 1V:1.5H, and downstream shoulder with a slope of 1V: 1.5H. Filters were incorporated on either side of the core. The same loading conditions were applied to the Earth Core Rock fill Dam as for the Earth fill Dam with a clay core.

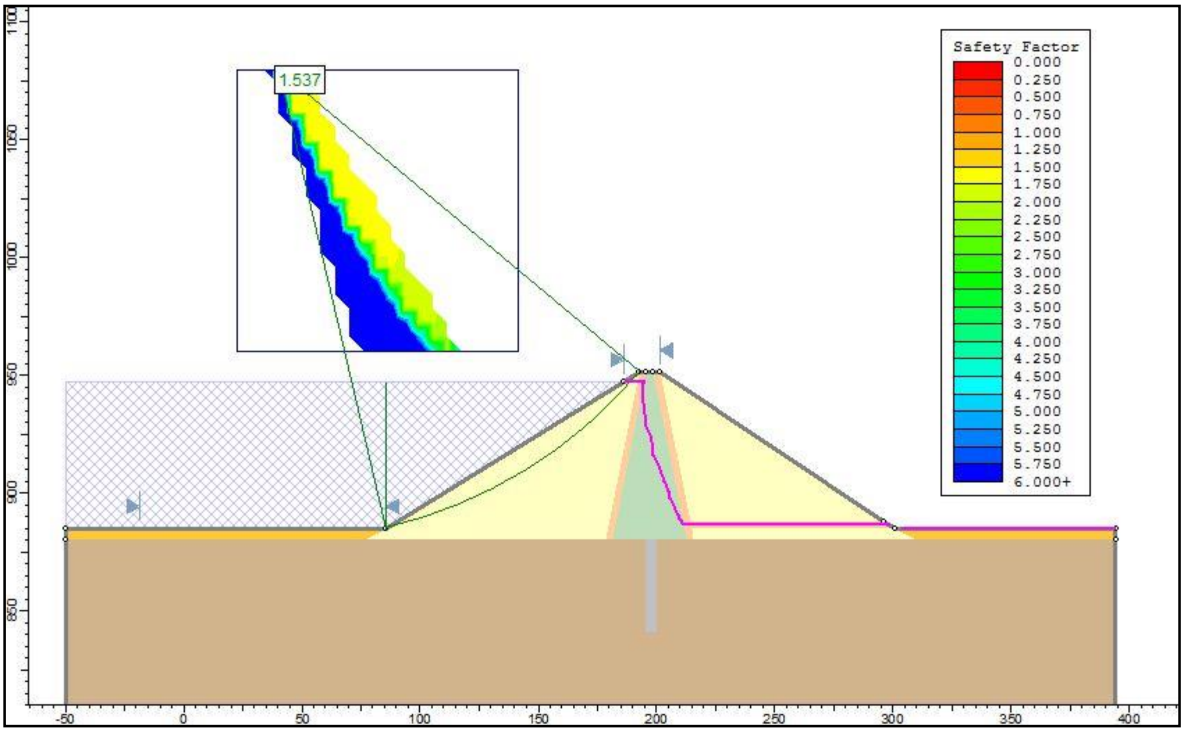


Figure 5-10: Earth Core Rock fill Dam: Upstream Face
(Full Supply Level (FSL): FoS = 1.537)

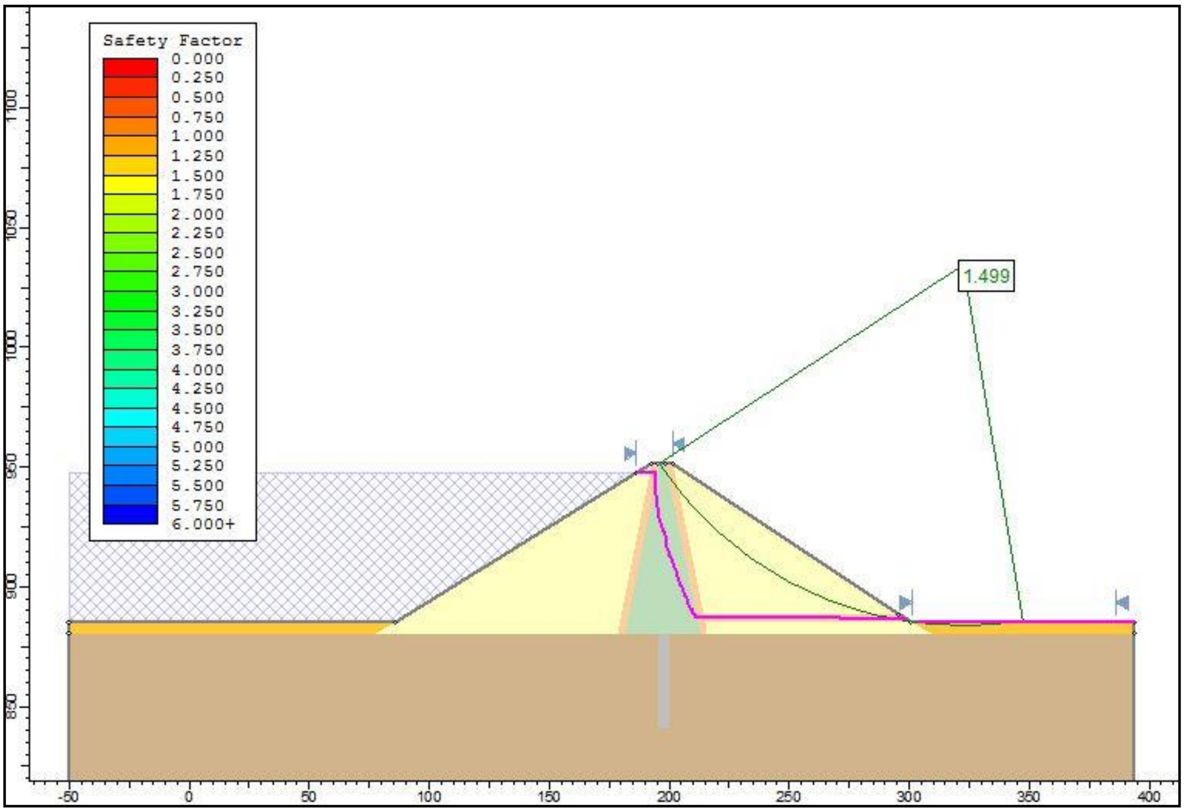


Figure 5-11: Earth Core Rock fill Dam: Downstream Shoulder
(Full Supply Level (FSL): FoS = 1.499)

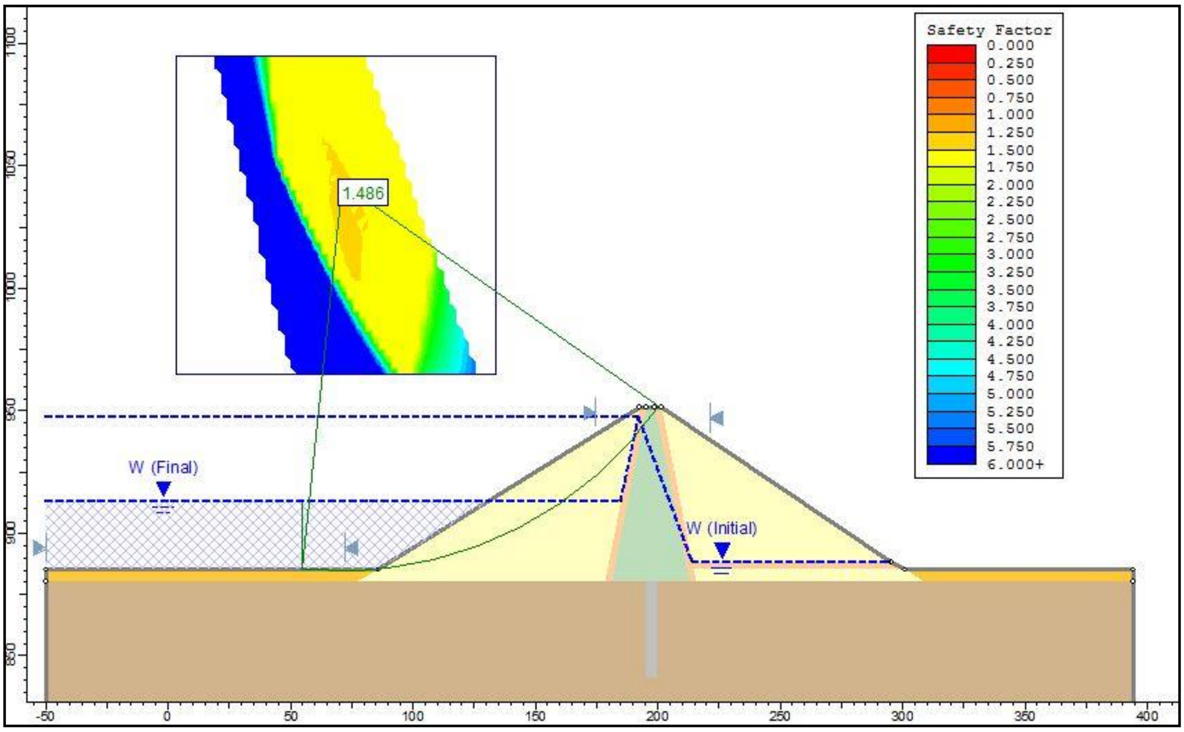


Figure 5-12: Earth Core Rock fill Dam: Upstream Shoulder
(Rapid Drawdown: FoS = 1.486)

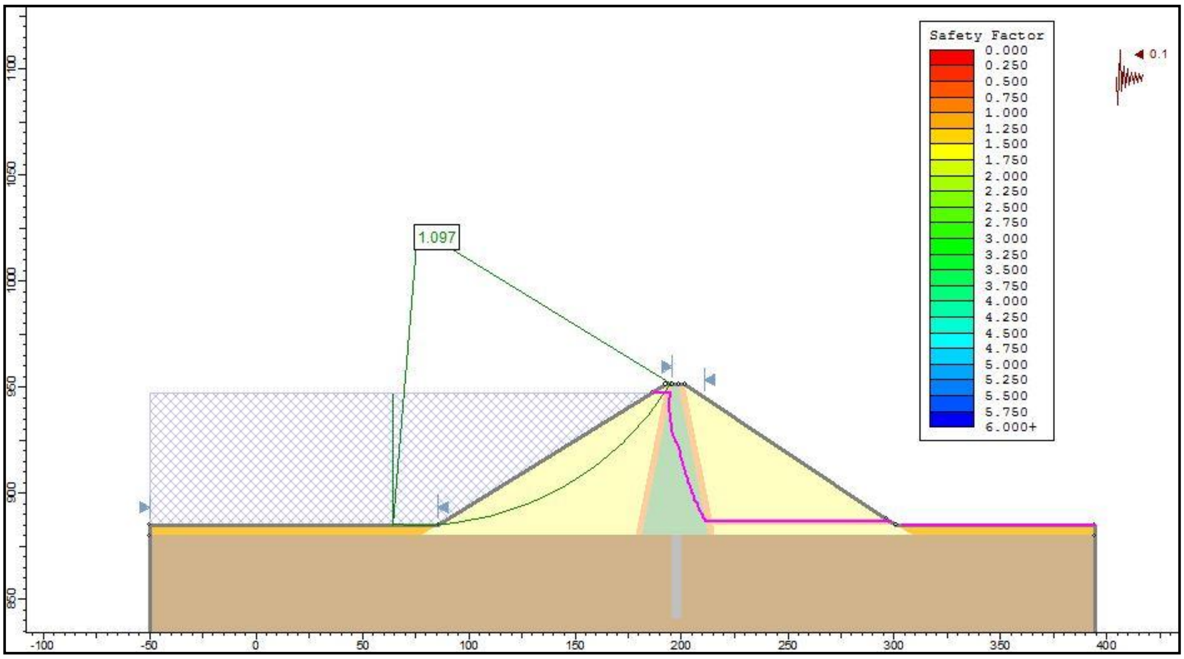


Figure 5-13: Earth Core Rock fill Dam: Upstream Shoulder
(Seismic Loading of 0.1g: FoS = 1.097)

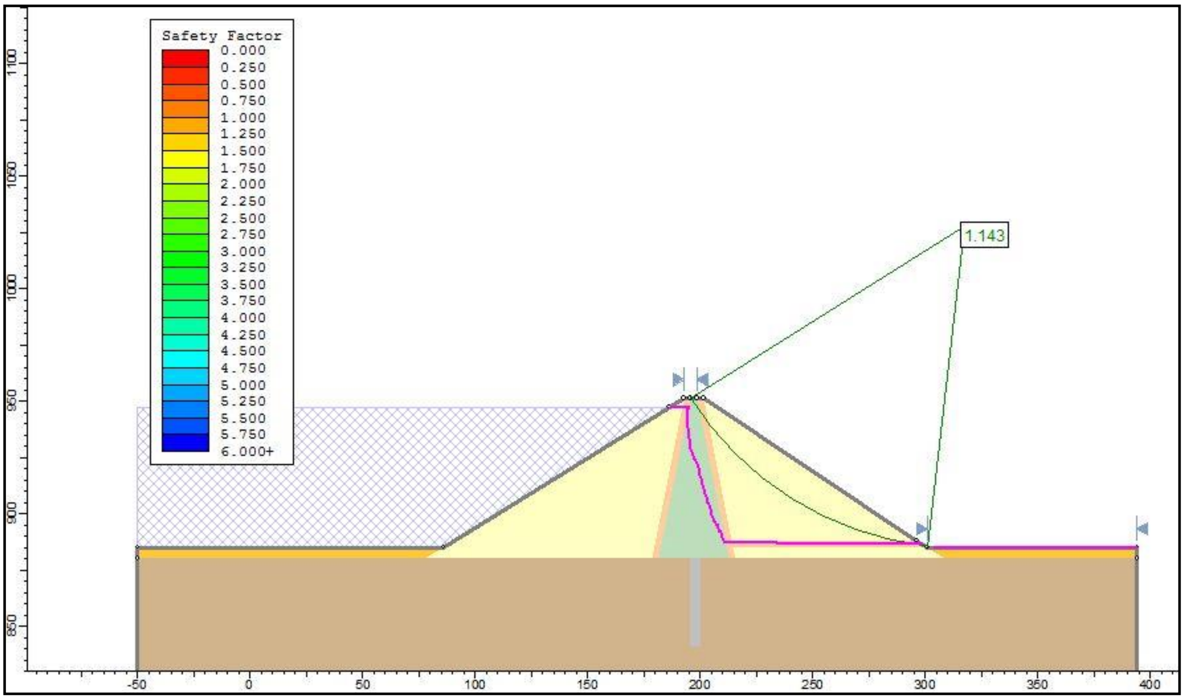


Figure 5-14: Seismic Loading of 0.1g: FoS = 1.143
(Earth Core Rock fill Dam: Downstream Shoulder)

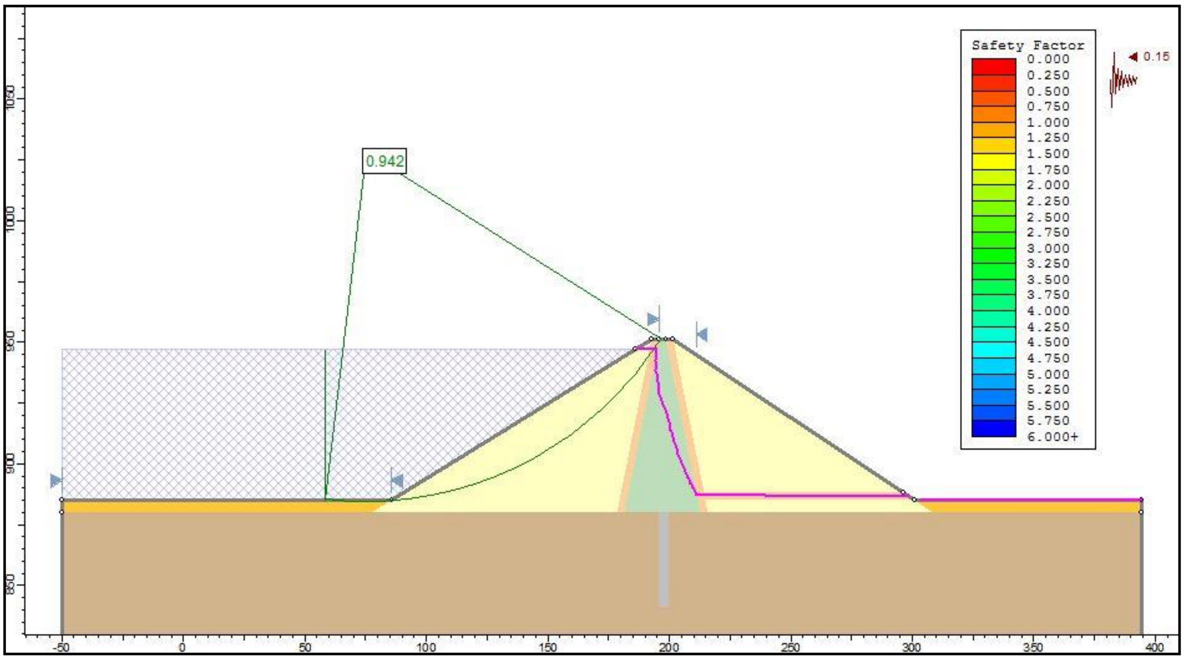


Figure 5-15: Seismic Loading of 0.15g: FoS = 0.942
(Earth Core Rock fill Dam: Upstream Shoulder)

c) Concrete Faced Rockfill

The third dam type analysed was a concrete faced rock fill dam with an upstream concrete face, with a cut off at the toe of the upstream shoulder. This cross-section was not modelled for the Rapid Drawdown loading condition as the concrete face, and filter zone on the outer face will prevent the build-up of pore pressures within the body of the dam.

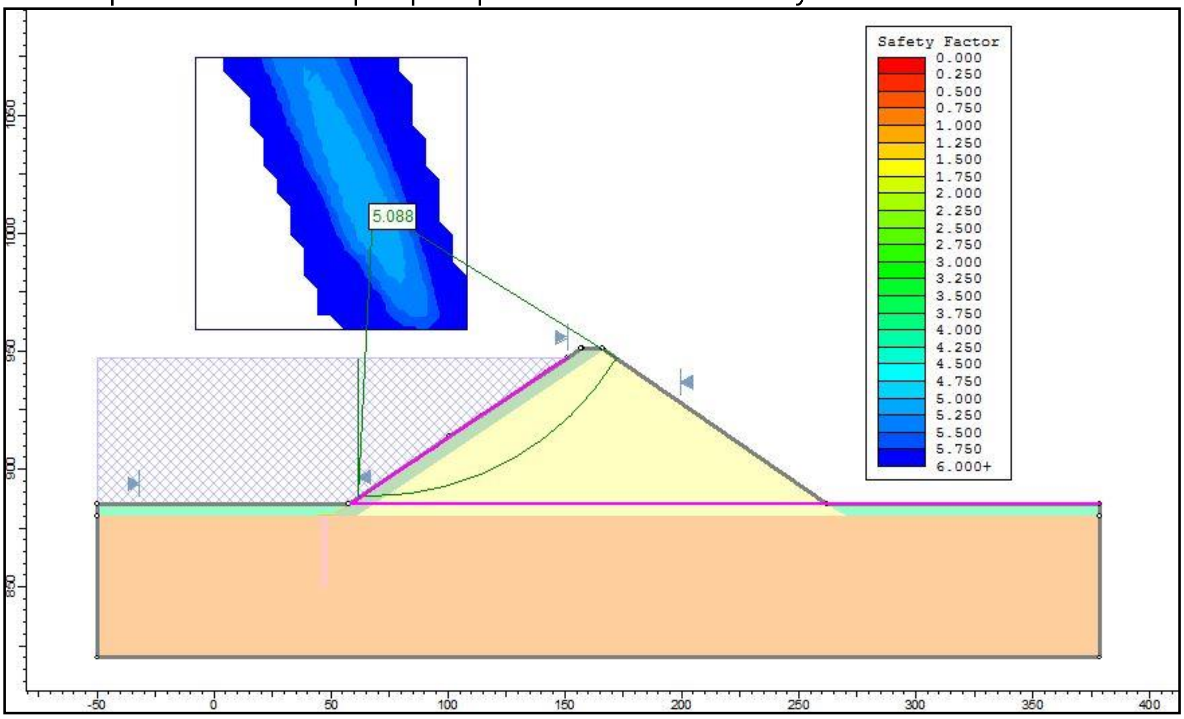


Figure 5-16: Full Supply Level: FoS = 5.088
(Concrete Face Rock fill Dam: Upstream Shoulder)

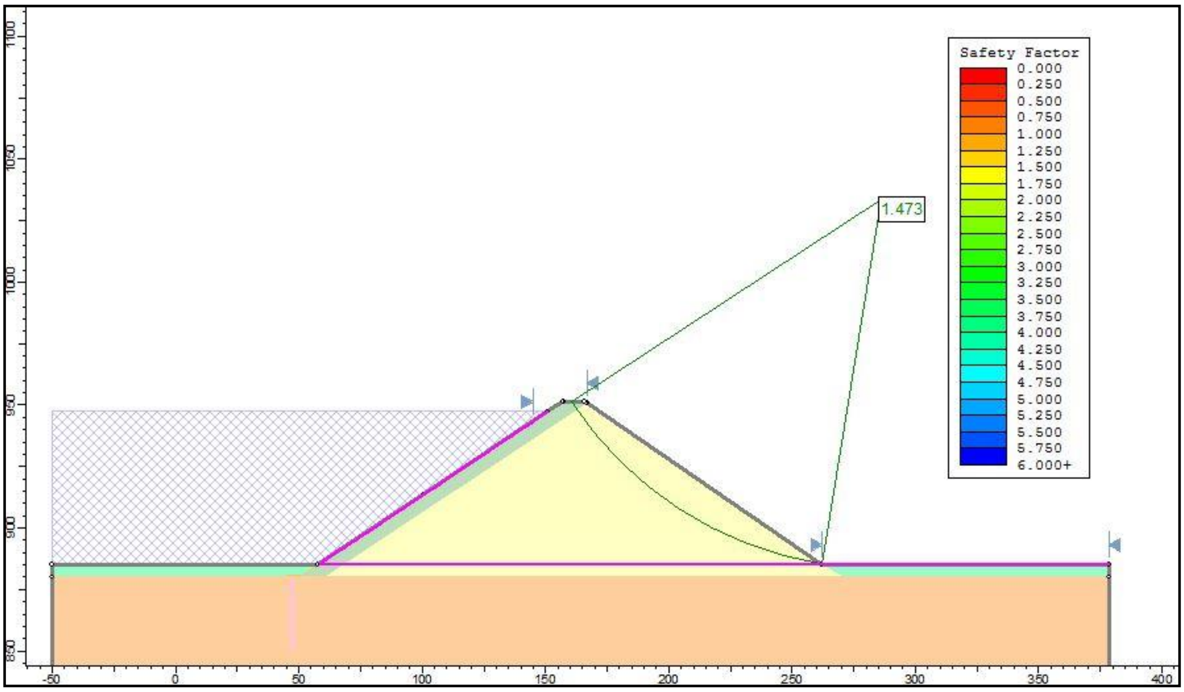


Figure 5-17: Full Supply Level (FSL): FoS=1.473
(Concrete Face Rock fill Dam: Downstream Shoulder)

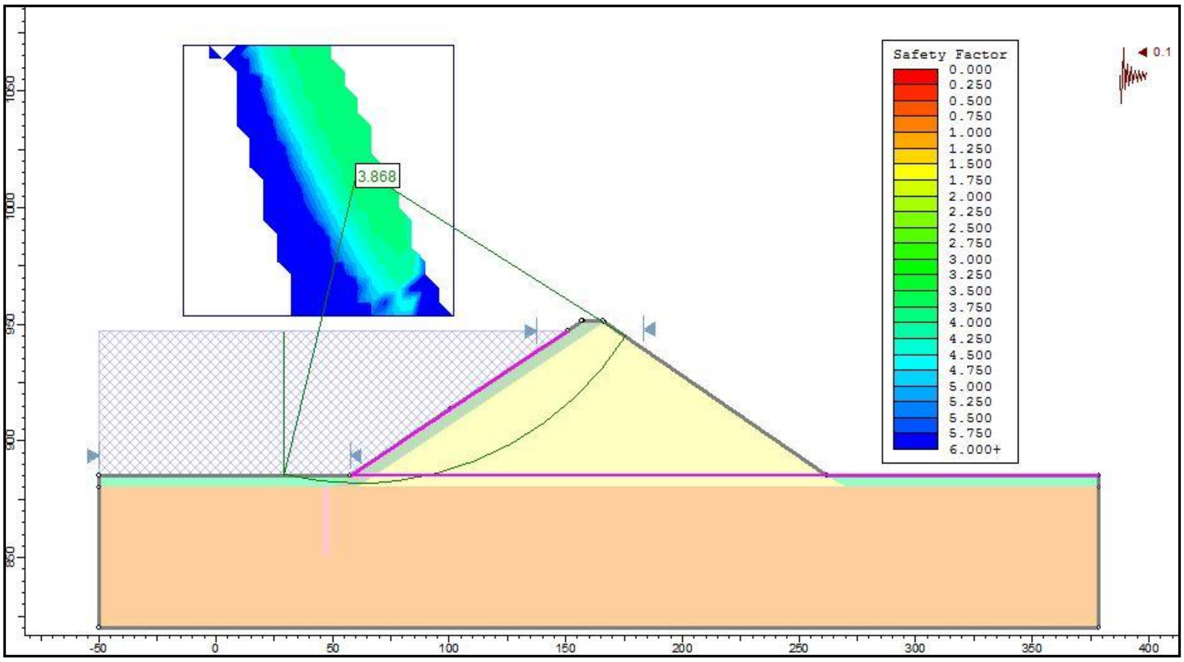


Figure 5-18: Concrete Face Rock fill Dam: Upstream Face
(Seismic Loading of 0.1g: FoS = 3.868)

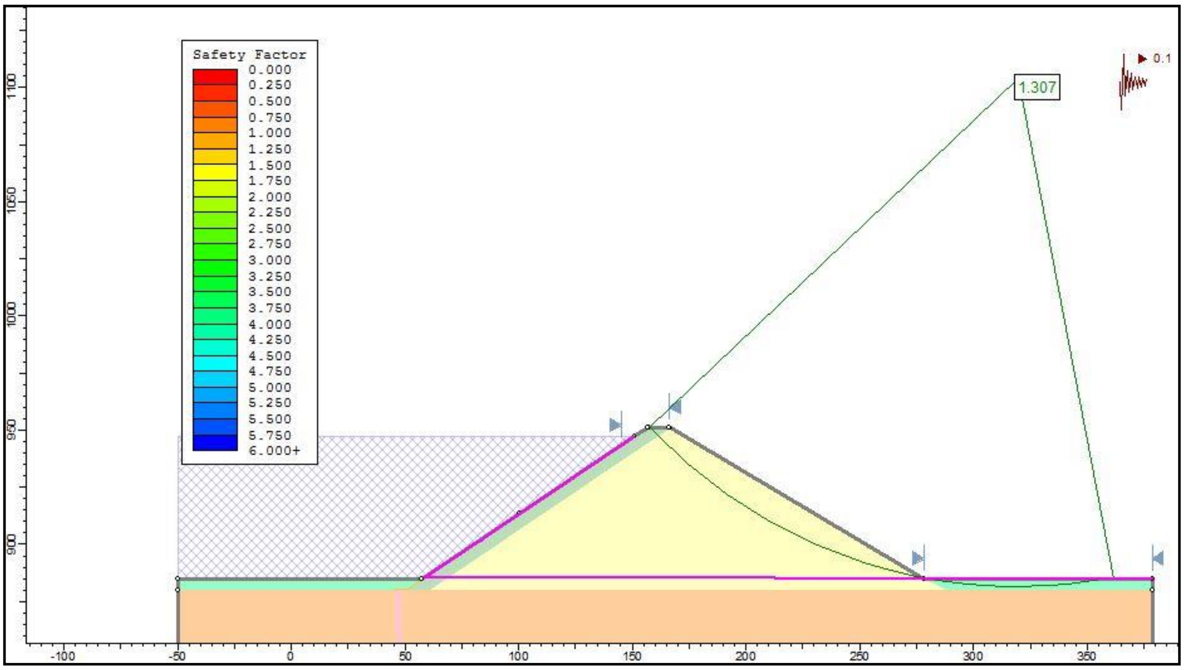


Figure 5-19: Concrete Face Rock fill Dam: Downstream Face
(Seismic Loading of 0.1g: FoS = 1.307)

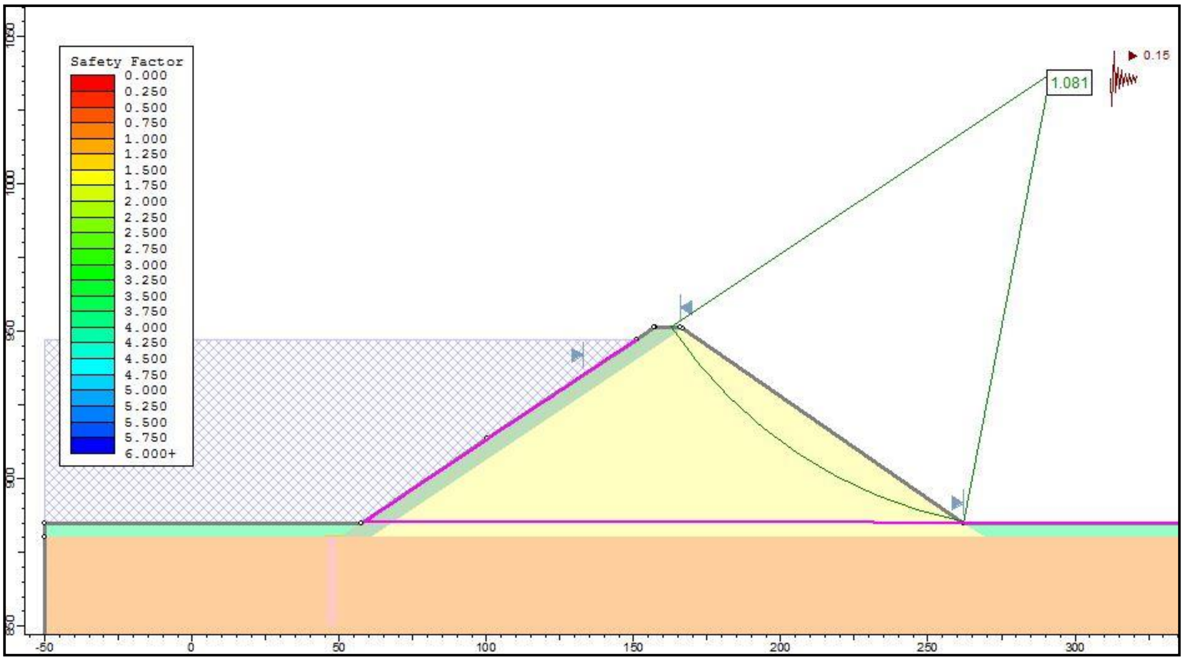


Figure 5-20: Concrete Face Rock fill Dam: Downstream Face
(Seismic Loading of 0.15g: FoS = 1.081)

5.3 Summary of Stability Analyses Results

Refer to Table 5-4 for the summary of the calculated factors of safety.

The only case which showed a Factor of Safety below the recommended minimum is the earth fill embankment dam for the Rapid Drawdown case, where the Factor of Safety was below 1.2.

The available material for use in the construction of the embankment shoulders is a mudstone, which although available in sufficient quantity, when broken down on site and compacted, results in an increase in the percentage of fines and results in a material with a permeability in the order of 10^{-9} m/s.

A permeability of 10^{-9} m/s is generally deemed to be unsuitable for use in the shoulder zones of an earth fill clay core embankment, as it would take a significant amount of time for pore pressures to dissipate with fluctuating reservoir levels which would result in lower shear strengths during any drawdown scenario. As will be shown below, this particular option is shown to not be a preferred solution.

The analyses for the ECRD embankment with clay core, indicated sufficient factors of safety for all cases. The ECRD option also makes good use of the available materials on site from the proposed spillway excavation, and would obviate the need to find an alternative use for the rock or a suitable spoil site. A suitable source of impermeable core material has been identified within the dam basin, which would be economically utilised in the embankment.

Likewise, the option of a concrete faced rock fill embankment (CFRD) passes all the stability criteria, and makes good use of the good quality dolerite available from the spillway excavation. These types of dams are inherently unlikely to fail due to the high permeability of the rock fill body of the dam, i.e. should the concrete face leak, the rock fill, if correctly graded and placed, will accommodate large leakages without jeopardising the stability of the dam.

Pore pressures inside the dam would remain relatively constant irrespective of reservoir levels as the impermeable zone is on the upstream face. The filter zone behind the concrete face should be graded so that the fine particles are on the outer edge of the filter, and the coarser material on the inner side. This will also provide a low permeability material under the concrete face and will assist in limiting leakage should cracks develop in the concrete.

The conclusion was that the above embankment profiles were viable feasibility designs and were suitable for usage in dam type comparative analysis.

Table 5-4: Summary of Calculated Factors of Safety

Dam Type	Analysis Description	Factor of Safety
Earth fill Embankment with Clay Core	Full supply level with steady state seepage conditions, for the upstream shoulder (US) and downstream shoulder (DS)	US: 2.90 DS: 1.53
	Full supply level with steady state seepage conditions and an applied seismic loading, for the most critical failure plane	0.1g: 1.00 0.15g: 0.85
	Rapid Drawdown	1.06
Earth Core Rock fill Dam	Full supply level with steady state seepage conditions, for the upstream shoulder and downstream shoulder	US: 1.53 DS: 1.50
	Full supply level with steady state seepage conditions and an applied seismic loading, for the most critical failure plane	0.1g: 1.05 0.15g: 0.94
	Rapid Drawdown	1.48
Concrete Faced Rock fill Dam	Full supply level with steady state seepage conditions, for the upstream shoulder and downstream shoulder	US: 5.00 DS: 1.50
	Full supply level with steady state seepage conditions and an applied seismic loading, for the most critical failure plane	0.1g: 1.30 0.15g: 1.08

5.4 Embankment Dams Seepage Analysis Findings

Using the models developed for the SLIDE slope stability package, a preliminary seepage assessment was undertaken for each of the three dam types.

The seepage analysis was undertaken prior to undertaking the stability analyses, and was used to determine the phreatic level through the embankment, using the hydraulic properties of the various layers. This phreatic level was then used in the stability analyses described above.

Permeabilities for the various materials were generally estimated from the material grading, apart from the core and shoulder material for the earth dam option for which the properties were determined from the laboratory test results.

These models should be re–run once more information is available at the detailed design stage.

Plots of the anticipated seepage pore pressures are presented below.

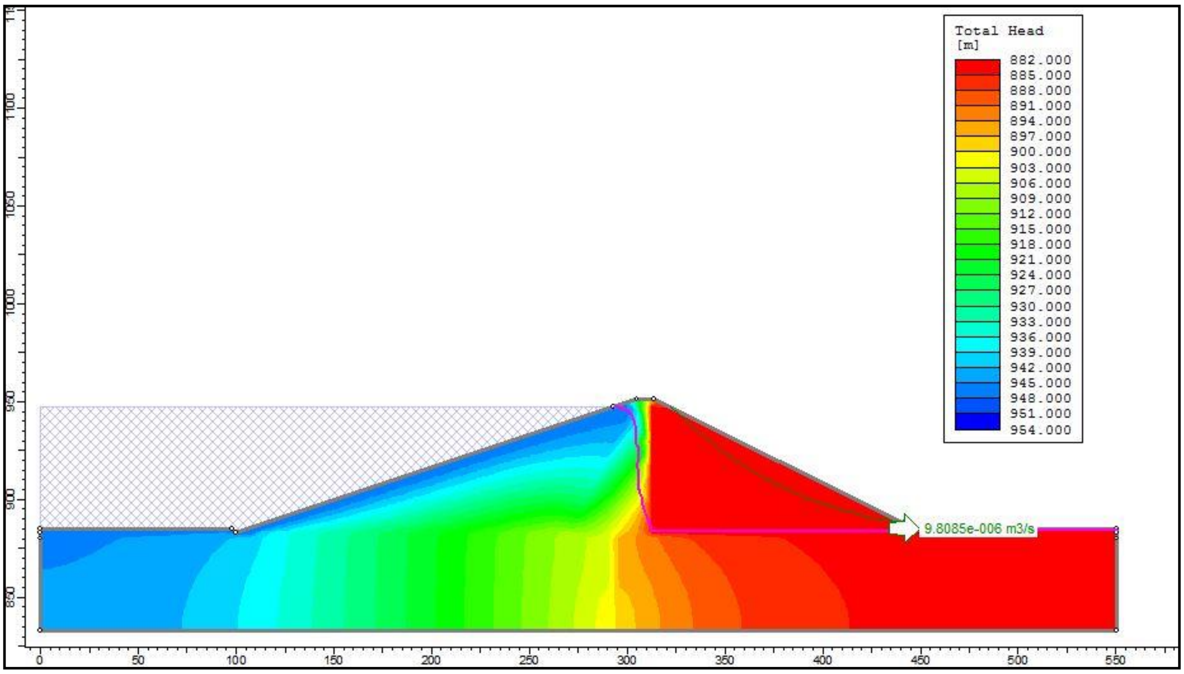


Figure 5-21: Earth fill Embankment: Seepage Analysis

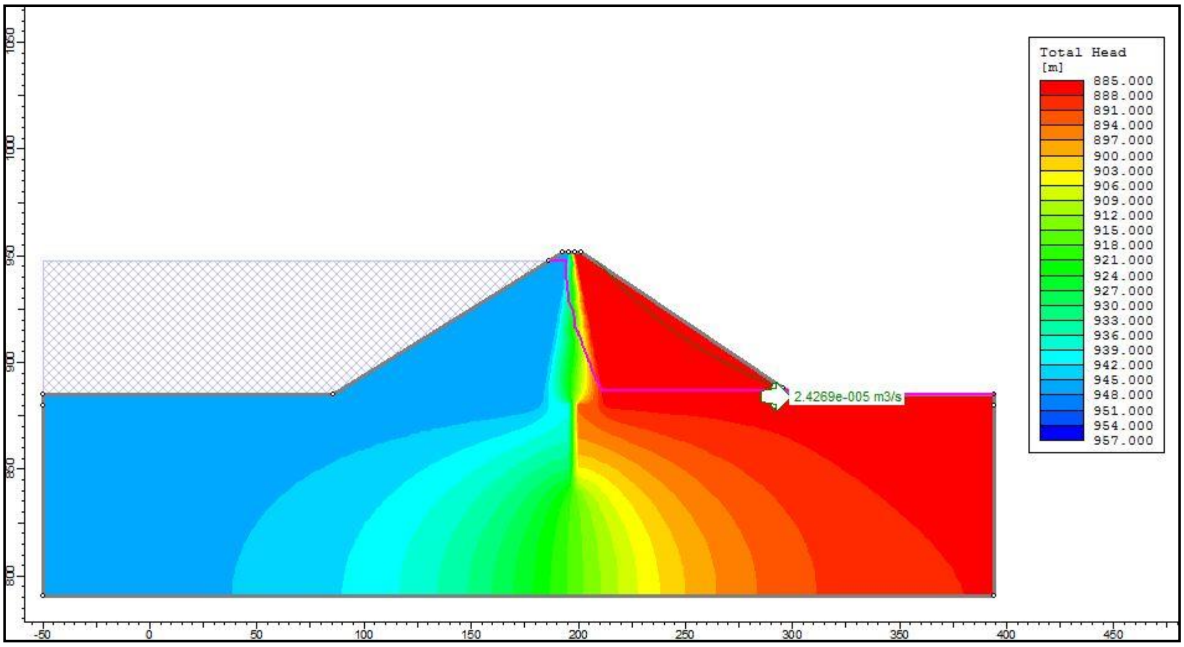


Figure 5-22: Earth Core Rock Fill: Seepage Analysis

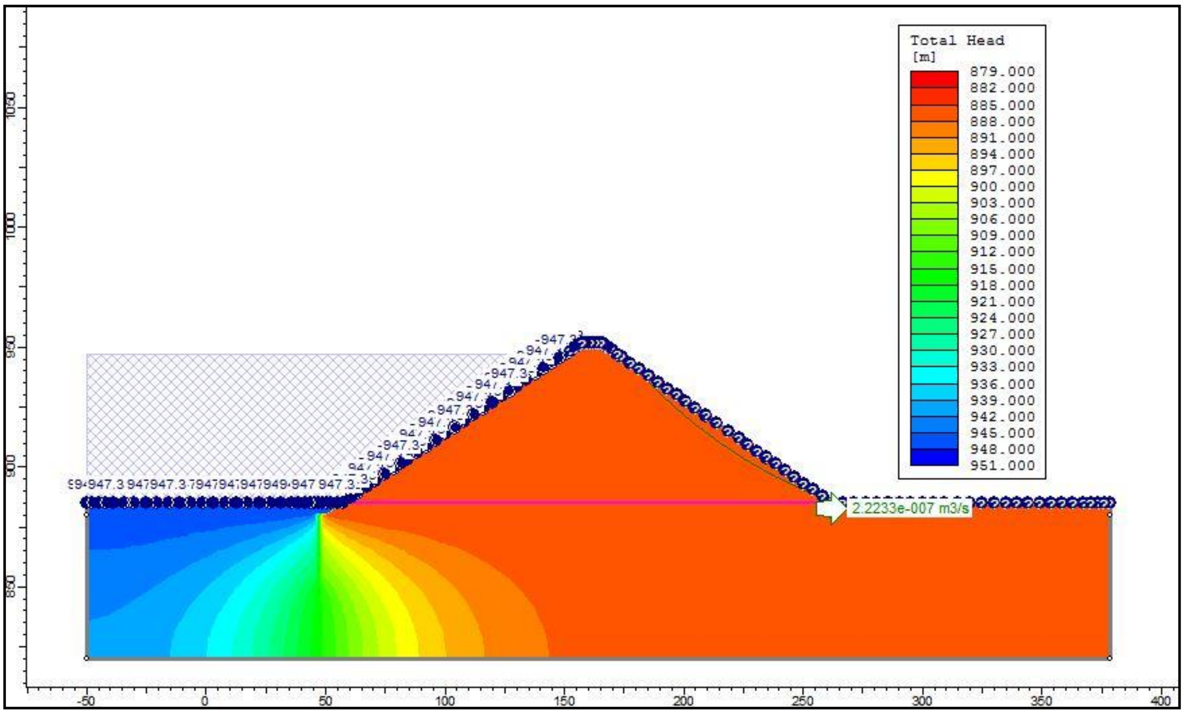


Figure 5-23: Concrete Faced Rock fill Dam: Seepage Analysis

Table 5-5: Summary of seepage per dam type

Dam Type	**Seepage (per metre length of wall)**
Earth fill Embankment	9.8×10^{-6} m^3/s
Earth Core Rock fill	2.4×10^{-5} m^3/s
Concrete Faced Rock fill	2.2×10^{-7} m^3/s

5.5 Settlement

Settlement within earthfill dams is complex and often does not follow the theory for classical Terzaghi consolidation theory as conventional oedometer consolidation tests use saturated soils whereas those in a fill or core of the dam are partially saturated.

Much of the consolidation consists of gas in the voids being compressed and forced into solution, rather than flow of water from the system. As a result, settlement is general relatively quick and predominantly happens during construction.

As recommended by Fell, R et al 2005, settlement of embankment earthfill during and after construction, can be estimated using the results of monitoring from other dams. Hunter & Fell, 2003 derived the following equation for construction settlement;

Therefore: $S_{FILL} = H\,(0.183H + 7.461)$
Where: S_{FILL} = settlement in mm
H =Embankment height in m
With H = 65 m
S_{FILL}=1258 mm

Long term settlement is generally limited to <1% of construction height.

Rockfills, if well compacted, generally settle <1% of construction height during construction, with post construction <0.5%.

If the earthfill embankment with the clay core option is selected it is however necessary to determine the settlement of the in-situ colluvium as a result of the applied load of the embankment.

The oedometer test performed on the undisturbed sample, obtained from the right flank colluvial material, indicated that the coefficient of volume compressibility m_v is 0.07381 1/MPa. One dimensional consolidation theory may be applied to estimate the settlement as a result of the substantial width of the embankment.

Therefore; $S_{oed} = mv \times H \times \Delta\sigma'$

Where; S_{oed} = settlement in mm
m_v = coefficient of volume compressibility
H = thickness of the in-situ material layer
$\Delta\sigma'$ = change in effective stress

S_{oed} = 430 mm

Total settlement, in the case of the earthfill embankment with clay core, is estimated to be:

$S_{TOTAL} = S_{FILL} + S_{oed}$
S_{TOTAL} = 1688 mm

This settlement allowance would need to have be taken into consideration when undertaking the detailed design, should an earth embankment solution have been adopted. However, the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/521/12 confirms that a roller compacted concrete (RCC) dam is the preferred solution.

5.6 Grouting

The water pressure tests undertaken generally indicate low water losses.

In Borehole N3 on the upper left flank, a high lugeon value of 145 was recorded for the test section between a depth of 6 m and 9 m. Below this zero water loss was recorded.

Borehole N1 on the lower left flank recorded high lugeon values in the upper stages. No result was obtained for the second stage test between a depth of 8.98 m and 11.84 m due to packer leakage. Below this lugeon values are generally below 5, except between a depth of 32.81 m and 35.23 m, where a value of 7 was recorded.

Borehole N2 on the lower right flank recorded zero water loss to a depth of 29 m. The last four stages below this recorded relatively high values ranging between 2 and 14 lugeons. This does not corroborate the rock description, of a competent rock with generally low fracture frequencies and is possibly the result of leakage past the packers.

Boreholes NL2/6 and NL2/9 on the mid and upper right flank indicated zero water loss throughout the lengths of the boreholes.

The water pressure test results indicate that foundation grouting will be required along the entire left flank, river section and at least to about midway along the right flank.

Curtain grouting should comprise primary grout holes at 6 m centres and to depths of 30 m through the river section, reducing sequentially to a depth of 12 m at the top of the left flank. Although it is recommended that provision is made for secondary grouting, the need for this must be assessed by ongoing water pressure testing during the grouting operation. The same for tertiary grouting. At this stage the drilling results do not indicate a need for consolidation grouting.

5.7 Spillway

In the case of an embankment dam, three alternative spillway arrangements have been proposed, two on the right flank and one on the left flank, as indicated on Figures 5-24 to 5-26.

The left flank option would have the least impact on associated infrastructure, as both right flank options would probably require pipeline and road crossings over the discharge chute.

Conversely the left flank option is unlikely to generate sufficient good quality dolerite from excavation to serve the project requirements for crushed rock aggregates, rock-fill and rip-rap, whereas a spillway excavation on the right flank could duplicate as a rock quarry capable of supplying the project requirements.

The first 330 m of Spillway Option 1 (Figure 5-24) runs on near surface and outcropping dolerite. Thereafter proceeding downslope towards the return-to-river the weathering profile becomes deeper and the channel will require lining to prevent excess erosion and scour.

The rock from excavation would be suitable for use as concrete aggregates, rock-fill and rip-rap. Any rock quantities required over and above the excavation volumes can be procured by extending or deepening the approach channel in an upstream direction.

Spillway Option 2 proposes a cutting through the hill as indicated on Figure 5-25. It would generate vast quantities of hard rock dolerite, likely to be in excess of the project requirements for concrete aggregates, rock-fill and rip-rap. The discharge chute invert is underlain by variable geology, from relatively weak mudrock to more competent sandstone in the lower reaches of the chute and stilling basin.

Spillway Option 3 (Figure 5-26) would predominantly comprise excavation in sandstone, with only the lower discharge chute in dolerite. Although with verification the sandstone generated from excavation is likely to be suitable as rock-fill, it will not be suitable for use as crushed aggregates or rip-rap. This option would most probably require the commissioning of a separate rock quarry to provide the project requirements.

A RCC dam option envisages a central in-channel spillway arrangement.

5.8 RCC Dam Option Analysis

CADAM software was used for the stability analysis. Figure 5-27 shows a general layout of the proposed RCC Dam and its juxtaposition with the associated infrastructure. This also includes recommendations for construction site area allocations.

The model was set up based on simple beam theory. This is a methodology mainly used for gravity dam design. Figure 5-28 shows the proposed cross section of the central uncontrolled ogee spillway. That is considered to be the deepest and for which the structural analysis was performed.

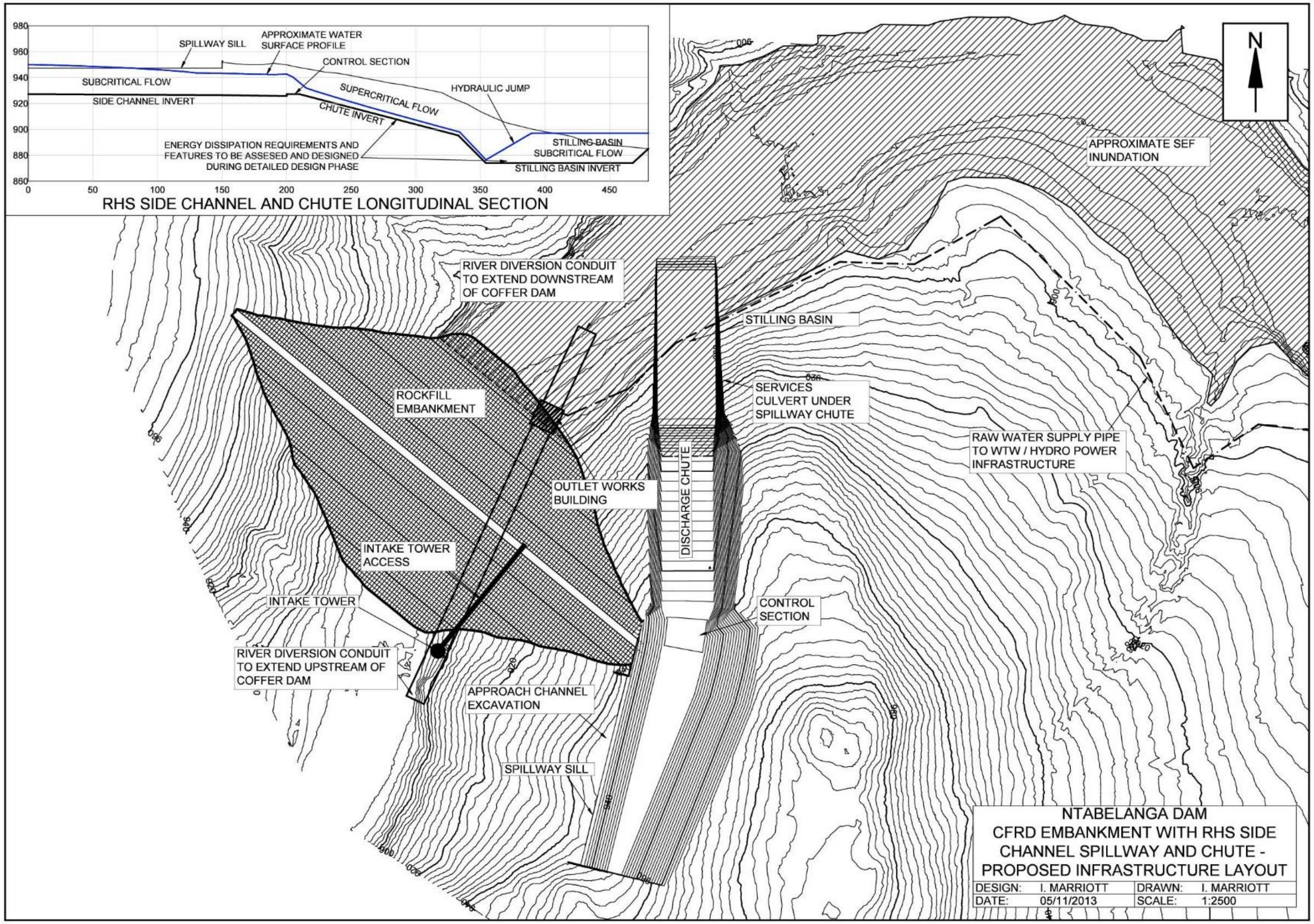


Figure 5-24: Spillway Option 1

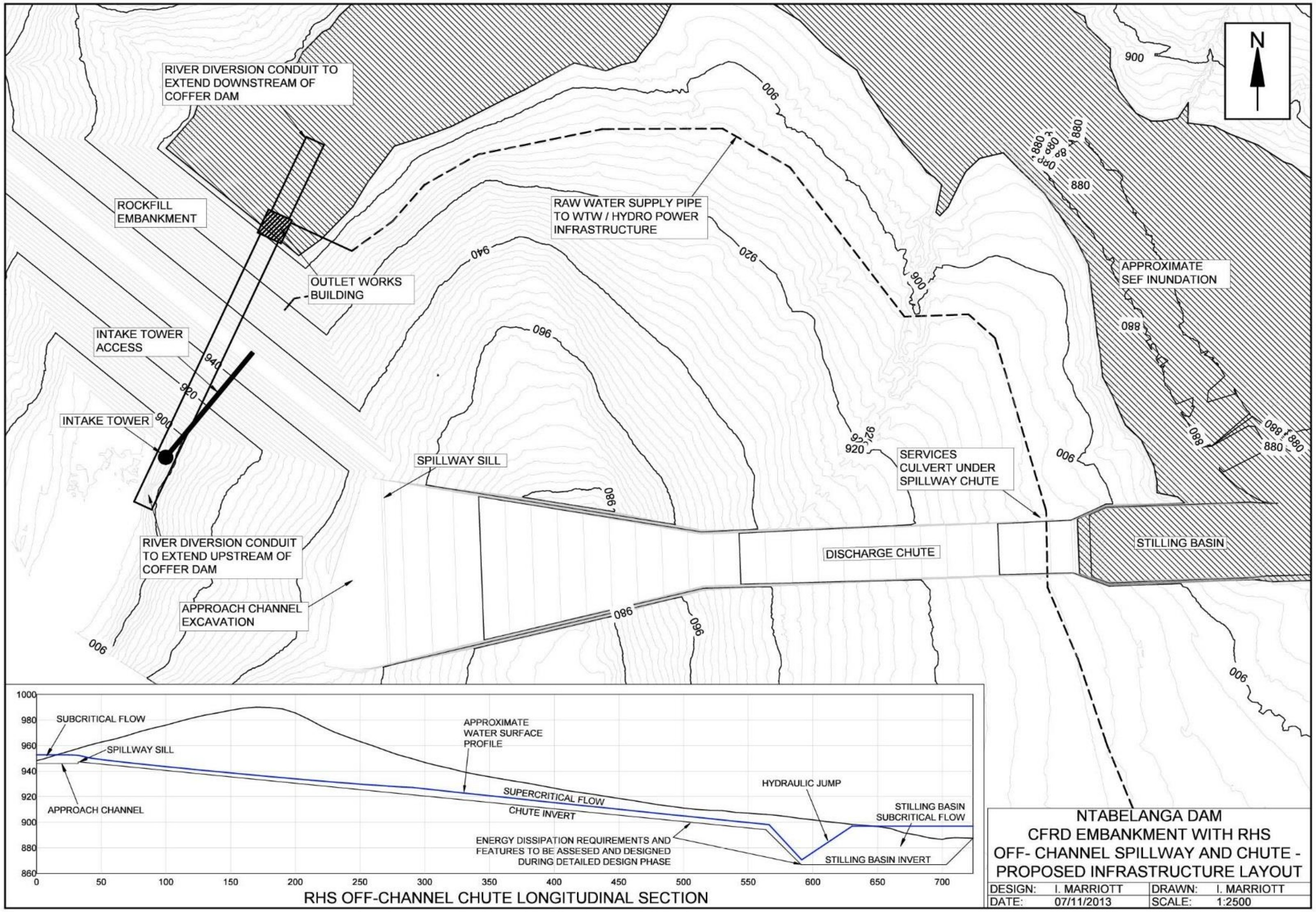


Figure 5-25: Spillway Option 2

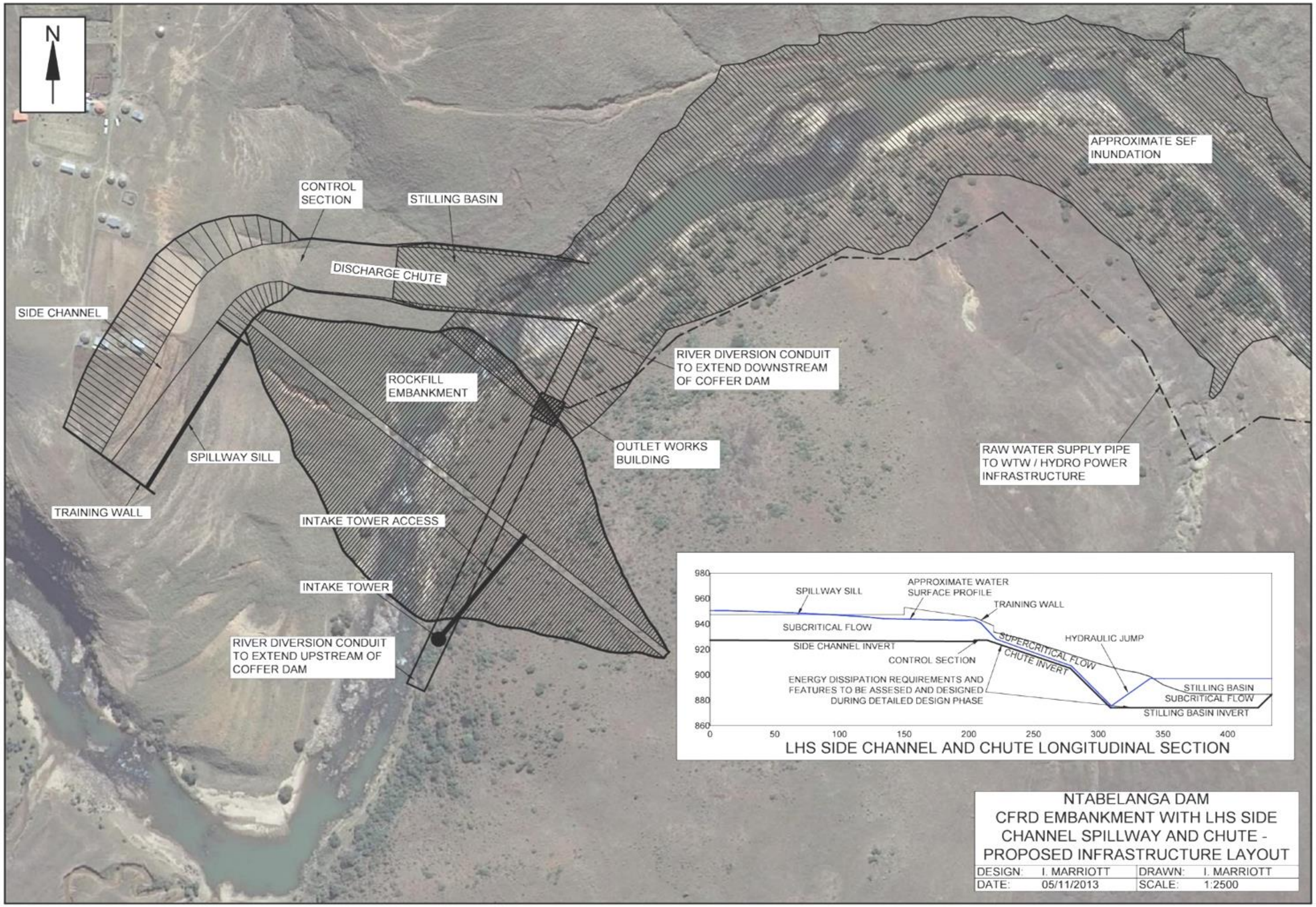


Figure 5-26: Spillway Option 3

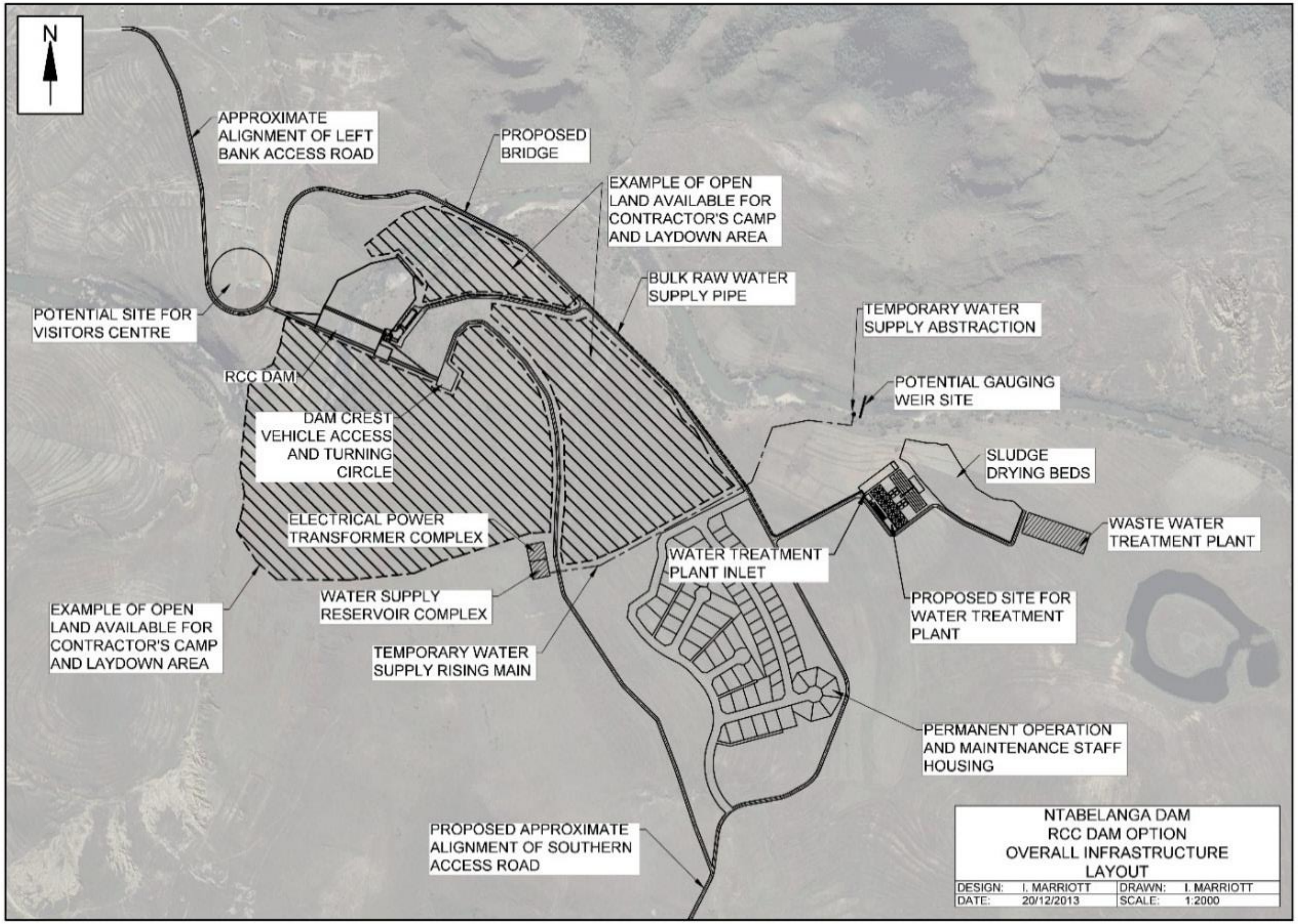


Figure 5-27: General Layout of the Proposed Dam and Associated Works

Figure 5-28 shows a configuration with a 1:0.70 (V:H) downstream slope for the wall and spillway, but model runs were also undertaken for a downstream slope of 1:0.75.

The following information and assumptions were used in undertaking the analysis:

- Ntabelanga Dam would have a maximum height of 67 m from the river bed level and a total crest length of 440 m;
- Floods would be discharged by means of un-controlled ogee stepped spillway;
- Concrete density of 2 400 kg/m^3;
- Concrete grade C15/53 would be used mainly for the RCC;
- [1]Solid dolerite founding condition with minimum cohesion of 0.3 MPa and minimum angle of friction of 35°;
- Horizontal component of peak ground acceleration = 0.15 g; and
- Vertical component of peak ground acceleration = 0.08 g.

The loading conditions to be investigated were discussed and agreed with the Department of Water and Sanitation and are shown in Table 5-6.

Table 5-6: Loading Conditions

Type	Case	FSL	RDF	SEF	Silt (S)	Tail water(TW)	Drained (D)	Undrained (UD)	Seismic (SM)
Normal	1	√			√		√		
	2		√		√	√	√		
Abnormal	3		√		√	√		√	
	4			√	√	√	√		
	5	√			√	√	√		√
Extreme	6		√		√	√	√		√
	7			√	√	√		√	

Tables 5-7 and 5-8 present the results obtained from the various load cases in Table 5-6. The analysis results are compared with the allowable factors of safety and maximum stresses according to various international guidelines. Analysis was run for downstream wall slopes of both 1:0.70 and 1:0.75.

Table 5-7: Analysis Results and Comparison (1:0.70 d/s Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturning Factor of safety (FOS)	
		R	A	R	A	R	A	R	A
Normal	1	+0.19	0.0	-1.2	-3.0	1.5	1.5	1.5	1.5
	2	+0.4	0.0	-1.4	-3.0	1.3	1.4	1.3	1.4
Abnormal	3	+0.61	0.2	-1.4	-4.5	1.1	1.1	1.1	1.2
	4	+0.56	0.2	-1.5	-4.5	1.1	1.1	1.2	1.2
	5	-0.27	0.2	-0.88	-4.5	2.2	1.1	1.7	1.2
Extreme	6	-0.07	0.35	-1.04	-4.5	1.9	1.0	1.5	1.1
	7	+0.77	0.35	-1.5	-4.5	1.0	1.0	1.0	1.1

*Legend - **A** = Allowable - = Compression **R** = Result + = Tension*

[1] *Literature on rock mass properties state cohesion can be in the range of 0.3 to 30 MPa (but this is not a sensitive parameter in this analysis) and an angle of friction up to 55°.*

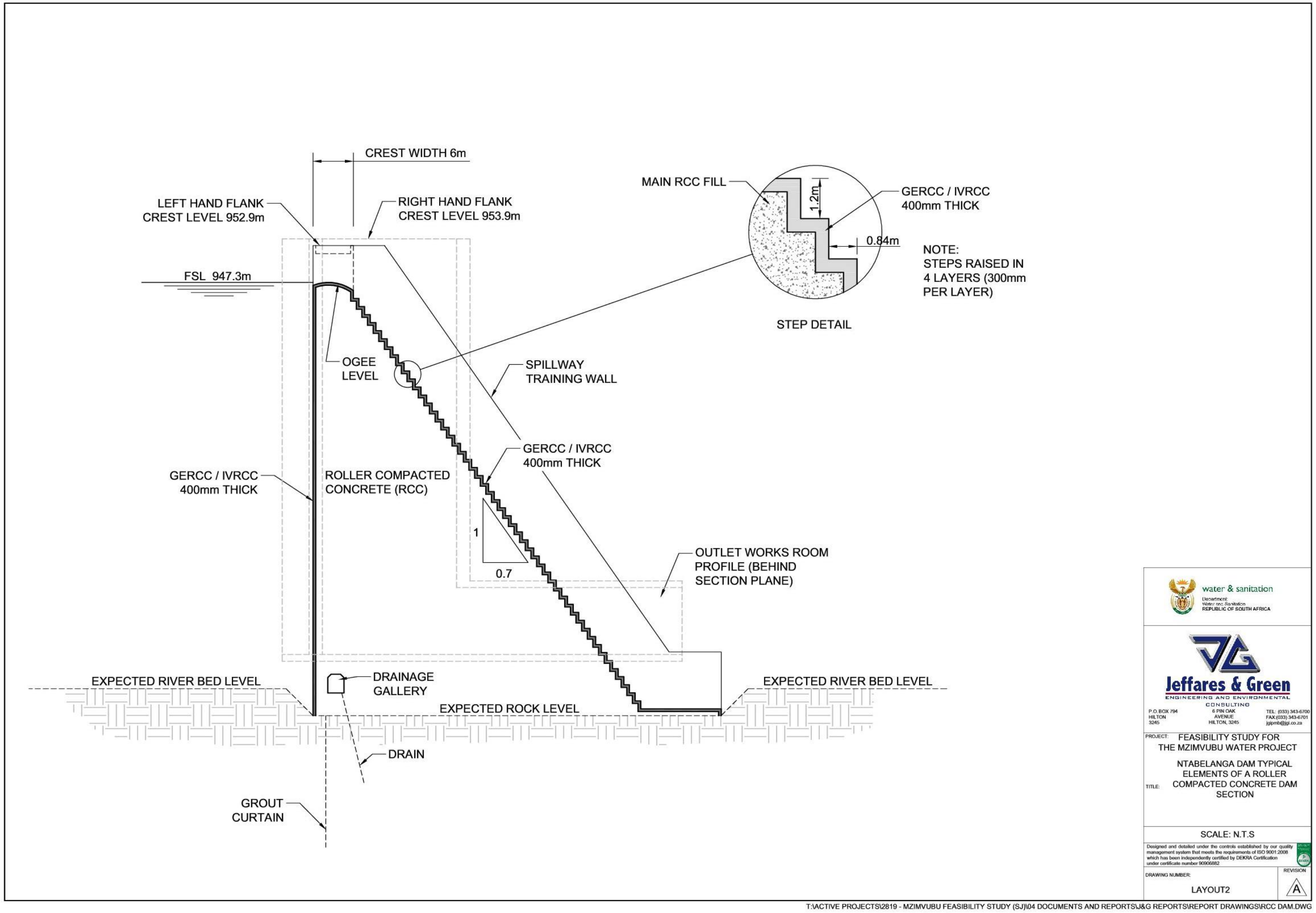


Figure 5-28: General Layout of the Proposed Dam and Associated Works

Table 5-8: Analysis Results and Comparison (1:0.75 d/s Slope)

Type	Case	Tensile Stress (MPa)		Compressive Stress (MPa)		Sliding (residual) Factor of safety (FOS)		Downstream overturning Factor of safety (FOS)	
		R	A	R	A	R	A	R	A
Normal	1	+0.03	0.0	-1.1	-3.0	1.62	1.5	1.54	1.5
	2	+0.22	0.0	-1.9	-3.0	1.4	1.4	1.4	1.4
Abnormal	3	+0.43	0.2	-1.9	-4.5	1.1	1.1	1.2	1.2
	4	+0.36	0.2	-2.0	-4.5	1.2	1.1	1.3	1.2
	5	-0.4	0.2	-1.2	-4.5	2.4	1.1	1.83	1.2
Extreme	6	-0.22	0.35	-1.4	-4.5	2.1	1.0	1.65	1.1
	7	+0.57	0.35	-2.0	-4.5	1.0	1.0	1.1	1.1

*Legend - **A** = Allowable - = Compression **R** = Result + = Tension*

These feasibility level results show that factors of safety for sliding and overturning are very close to those allowable for the 1:0.70 downstream slope option, and are conservative for the 1:0.75 downstream slope option. In both options, some of the tensile stress results are higher than allowable.

The eventual geometry of the dam wall would be determined following an extensive detailed design process including finite element and numerical elastic analyses, and this is normally a balance between minimising cost and meeting all of the allowable safety criteria.

This would include consideration of various cross section profiles, mix designs, and tensile crack control/induction methodologies. This will also include considering whether a sloped (rather than vertical) upstream face, or horizontally arched upstream face option is a beneficial and economic solution.

Typically RCC dams are built with downstream slopes of between 1:0.70 and 1:0.80, but this can be steeper on the upper part of the embankment if a non-symmetrical slope approach (base slope shallower than higher up the wall) is adopted.

For the feasibility design and costing of the Ntabelanga Dam, a simple symmetrical profile as given in Figure 5-28 was adopted, with a slope of 1:0.70.

5.9 Recommendations for Further Detailed Geotechnical Investigations

The feasibility level geotechnical investigations of the Ntabelanga Dam site entailed an overall assessment of the two alternative dam alignments, namely Line 1 and Line 2 as shown in Figure 3-2. Line 1 is the currently favoured alignment.

The investigations were also not specific to any particular dam type, the options being roller compacted concrete (RCC), concrete faced rock-fill dam (CFRD), earth core rock-fill dam (ECRD) or an earth embankment earth core dam. Three alternative spillway options are also under consideration.

Based upon the results of the feasibility level investigations, founding conditions along the Line 1 alignment are suitable for any of the alternative dam types. A constraint to the construction of an earth embankment dam is a shortfall of good quality pervious and semi-pervious shell material occurring within the future impoundment basin.

At this stage, additional, detailed investigations considered necessary to bring the level of detail up to that required to undertake the detailed design and tender documentation for the proposed construction of the dam and appurtenant works is listed below.

5.9.1 Rotary Core Drilling

As no geophysical anomalies were detected, the detailed rotary core drilling investigation would concentrate on infill drilling of both dam flanks, the selected spillway alignment, appurtenant structures and to prove sufficient reserves of rock aggregate for the construction.

a) Infill Drilling on the Left Flank Line 1 Alignment and Dam Footprint

It is recommended that an inclined borehole is drilled through the dolerite / sandstone contact on the mid left flank and that another inclined borehole is drilled beneath the river section from the left river bank. Provision must also be made for additional drilling on both the upstream and downstream dam embankment footprints.

b) Infill Drilling on the Right Flank Line 1 Alignment and Dam Footprint

It is recommended that an inclined borehole is drilled beneath the river section from the right bank. In addition a borehole will be required for the assessment of the slope transition from the lower to mid right flank. Whilst fairly comprehensively covered in the feasibility level drilling, provision must be made for additional drilling on both the upstream and downstream footprints.

c) Infill Drilling on Spillway, Stilling Basin and Chute Alignment

Following confirmation of the spillway alignment, infill drilling will be required to augment the feasibility level drilling results. This will serve the dual purpose of proving sufficient reserves of hard rock dolerite for use in construction.

d) *Appurtenant* Works

As part of the overall drilling programme for the detailed investigations, provision must be made for drilling to assess founding conditions for the intake tower, outlet works, downstream bridge crossing, water treatment works and other related infrastructure.

5.9.2 Trial Pitting

a) Pipelines

Trial pitting, either by means of a TLB or an excavator will be required along the route of the pipeline from the dam to the treatment works, as well as the distribution pipelines from the water treatment works. It is recommended that trial pits be excavated at intervals of about 100 m, or as deemed appropriate according to the prevailing conditions and pipe details.

b) Assessment of Founding Conditions for Structures and Appurtenant Works

It is recommended that trial pits are over the proposed development area on a grid spacing of approximately 100m. Where necessary trial pits may be augmented by the undertaking of dynamic cone penetrometer (DCP) tests. As part of this assessment, sampling and testing of the materials must be undertaken.

c) Borrow Pits for Dam Construction

Based upon the dam type and material requirements, trial pitting will be required to prove sufficient reserves of core, sand and other unconsolidated materials for the construction of the dam.

d) Access Road Centre-line and Borrow Pit Investigation

It is understood that the conceptual design envisages the upgrading of the existing access roads into the site, namely the road from Ntywenka and that from Somerville. In addition a new road is planned from the existing gravel road into the site and continuing across the river to provide access to the areas on the northern side of the Tsitsa River. The dam full supply line also inundates section of the existing roads and these will require re-aligning.

Trial pitting at intervals of about 250 m to 300 m will be required along the effected roads for the assessment of subgrade conditions. In addition the investigation of borrow pits will be required for road construction. The trial pitting must be combined with sampling and testing.

The cuttings down the hill from Ntywenka into the valley shows evidence of slope instability and provision must be made to undertake slope stability analyses.

6. CONCLUSIONS

The foregoing report presents the sequence of geotechnical investigations undertaken for the feasibility study into the development of a multi-purpose dam in the Mzimvubu catchment area in the Eastern Cape.

The Phase 1 component involved geotechnical investigations of three shortlisted dam sites, namely the Thabeng and Somabadi sites on the Kinira River between Matatiele and Mount Fletcher and the Ntabelanga site on the Tsitsa River between Maclear and Tsolo.

Based upon various interrelated evaluations including the results of the preliminary geotechnical investigations, the Ntabelanga site was selected as the preferred site for the subsequent Phase 2 feasibility level investigations.

The Phase 2 geotechnical investigations entailed additional drilling, trial pitting, geophysics, materials investigations, sampling and testing. The investigations were not specific to any dam type or centre-line alignment, as these had not been defined at the time. The alternative dam types considered included, an earth-fill dam, earth core rock-fill dam, concrete faced rock-fill dam and a roller compacted concrete dam.

Two alternative alignments were initially considered, namely the Line 1 upstream alignment and the Line 2 downstream alignment. In addition, during the course of the investigations no decision had been made on the locations of the inlet works, outlet works, spillway, pump-station, treatment works and other infrastructure components.

The geotechnical investigations were therefore planned to provide an overall assessment of founding conditions and the availability of materials for the proposed dam construction. The investigation undertaken has addressed these issues and determined the Ntabelanga site to be suitable for a number of alternative dam types.

The drilling and geophysics undertaken indicated the Line 1 alignment to offer better founding conditions. The upper to mid left flank of the Line 1 alignment is underlain by sandstone, with depth of excavation for the cut-off trench of the order of 7 m to 8 m. The lower left flank is underlain by dolerite with cut-off depths varying from about 6 m to 3 m on the edge of the river.

The lower right flank is underlain by unconsolidated transported and residual deposits over dolerite bedrock, with depth of cut-off varying between about 2 m and 6 m. The mid to upper right flank is underlain by near surface and outcropping dolerite and excavation for the cut-off along this section will be required merely to key in the foundation.

Additional drilling during the detailed investigation phase will be required for infill purposes and to verify conditions at transitions, such as at the sandstone / dolerite contact on the middle left flank on the transition from the unconsolidated soil mantle to near surface dolerite on the lower to mid right flank. Infill drilling will be required on the embankment footprint, particularly on the upstream side. Drilling on the positions of component structures will also be required.

The investigation of potential construction material sources was undertaken before the required material types or quantities had been defined and was therefore again an overall evaluation of materials availability and quality.

Based upon initial reconnaissance, trial pitting and testing, materials suitable for use as impervious core are expected to occur within the future impoundment basin in sufficient quantities for construction purposes.

Weathered sedimentary rocks comprising mainly mudrock with subordinate, intercalated sandstone were investigated as potential shell material. Testing on the materials indicated a tendency to break down under compaction, to the extent that it is unsuitable as pervious fill and marginally suitable as semi-pervious fill, with permeability values of about 10^{-9} m/s, which is comparable to the specification for impervious core.

This is viewed as constraint to the construction of an earth embankment dam. Whilst alternative material types are available in the project area which visually appear suitable for use as shell, these occur predominantly outside of the future impoundment basin, with consequential cost and environmental implications associated with their use.

Other dam construction materials, including sand and rock occur within the basin in sufficient volumes for the construction of any of the dam alternatives. The Tsitsa River sand appears suitable for use as filter and fine drainage medium, as well as for use as fine concrete aggregate. It is difficult to predict whether this sand quality will be consistently and quantity sufficient once borrow areas along the river are opened up, and provision should be made for importation of suitably graded sand from other sources or crusher sand used to blend with the sand obtained from the borrow pits upstream of the dam site. This need could be verified by further materials investigations during the detailed design stage.

Hard rock dolerite is plentiful in the dam basin and specifically occurs as near surface to outcropping rock on an extensive spur that forms the right flank of the dam. Good quality rock is available in abundance from excavation in the middle to upper right flank and extending upstream of the dam. Strength and mineralogical tests undertaken on the rock indicate that it is suitable for use as crushed rock aggregates, rock-fill and rip-rap.

At the time of these geotechnical investigations, three optional spillway alignments were identified, two on the right flank and one on the left flank. The advantage of a spillway on the right flank would be that the excavation would generate good quality dolerite rock for use in the dam construction.

Conversely the two right flank alternatives would require longer discharge chutes that would require lining along a major proportion of their length and would require crossing points to be provided for roads and pipelines. A spillway excavation on the left flank would be predominantly in sandstone, which although possibly suitable as rock-fill is not considered suitable as a crushed rock aggregate. This would necessitate the opening of a separate rock quarry. A spillway on the left flank would require a shorter discharge chute the invert of which is likely to be in rock of the major proportion of its length.

In respect of an earth embankment dam, the availability of approximately 2.1 million m³ of good quality pervious and semi-pervious shell material in the dam basin within economical haulage distance could be a problem.

Regarding the four dam types under consideration, based on the geotechnical investigations undertaken, founding conditions at the Ntabelanga site are considered suitable for all of the alternatives.

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