P WMA 12/T30/00/5212/10

water & sanitation

Department:
Water and Sanitation
REPUBLIC OF SOUTH AFRICA

DIRECTORATE: OPTIONS ANALYSIS

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT

GEOTECHNICAL INVESTIGATIONS:

LALINI DAM AND HYDROPOWER SCHEME

OCTOBER 2014

FEASIBILITY STUDY FOR THE MZIMVUBU WATER PROJECT
APPROVAL

Report title:	***Geotechnical Investigations: Lalini Dam and Hydropower Scheme***
Authors:	*T Speirs and J Olivier*
Project name:	***Feasibility Study for the Mzimvubu Water Project***
DWS Report Number:	*P WMA 12/T30/00/5212/10*
PSP project reference number:	*002819*
Status of report:	*Final*
First Issue:	*October 2014*
Second Issue:	
Final issue:	*October 2014*

CONSULTANTS: JEFFARES & GREEN
Approved for Consultants:

T Speirs
Geotechnical Engineer

A Pepperell
Study Leader

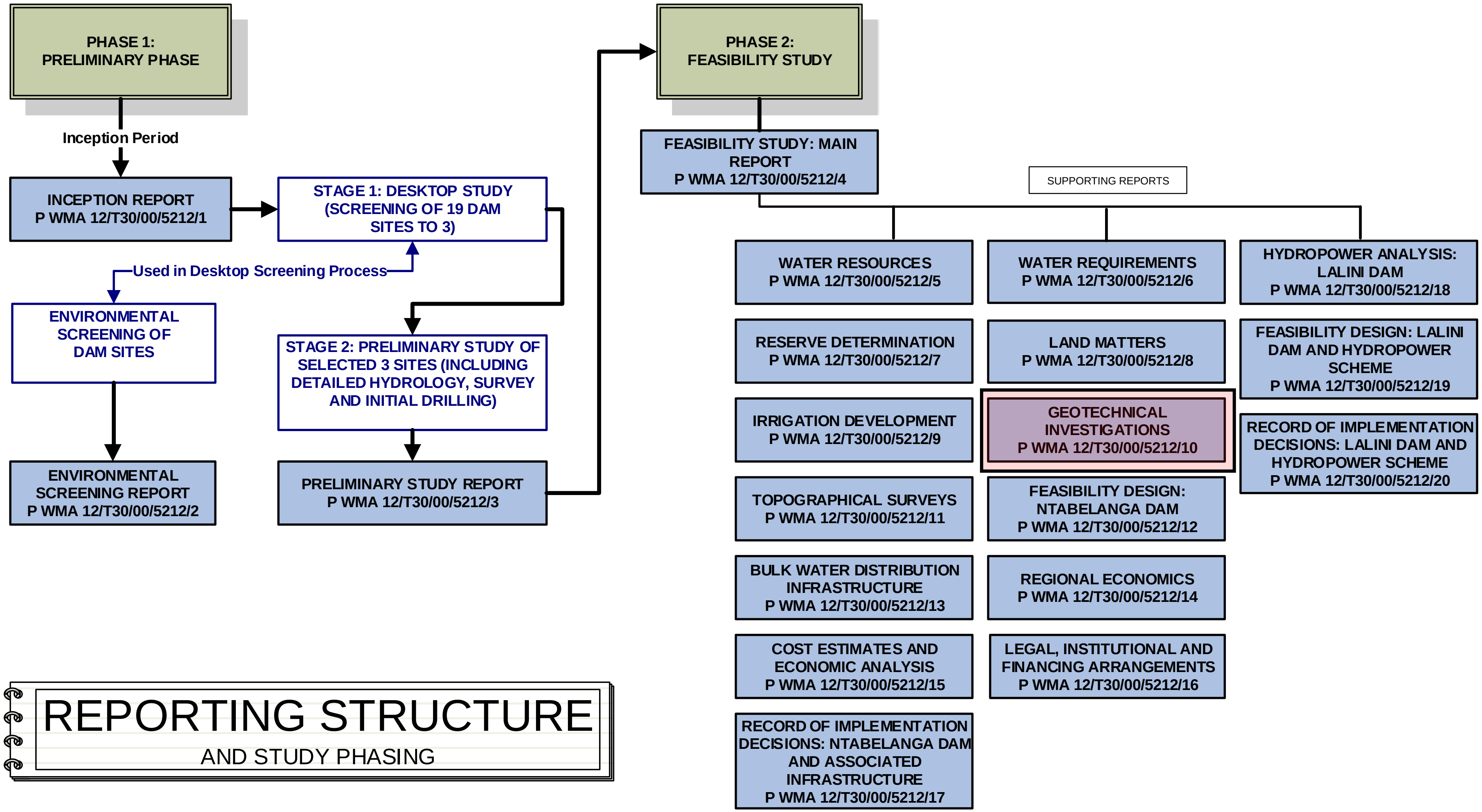
DEPARTMENT OF WATER AND SANITATION
Directorate: Options Analysis
Approved for DWS:

M Mugumo
Chief Engineer: Options Analysis (South)

L S Mabuda
Chief Director: Integrated Water Resource Planning

LIST OF REPORTS

REPORT TITLE	DWS REPORT NUMBER
Inception Report	P WMA 12/T30/00/5212/1
Environmental Screening	P WMA 12/T30/00/5212/2
Preliminary Study	P WMA 12/T30/00/5212/3
Feasibility Study: Main Report	P WMA 12/T30/00/5212/4
Volume 1: Report	
Volume 2: Book of Drawings	
FEASIBILITY STUDY: SUPPORTING REPORTS:	
Water Resources	P WMA 12/T30/00/5212/5
Water Requirements	P WMA 12/T30/00/5212/6
Reserve Determination	P WMA 12/T30/00/5212/7
Volume 1: River	
Volume 2: Estuary: Report	
Volume 3 :Estuary: Appendices	
Land Matters	P WMA 12/T30/00/5212/8
Irrigation Development	P WMA 12/T30/00/5212/9
Geotechnical Investigations	**P WMA 12/T30/00/5212/10**
Volume 1: Ntabelanga, Somabadi and Thabeng Dam Sites: Report	
Volume 2: Ntabelanga, Somabadi and Thabeng Dam Sites: Appendices	
Volume 3: Lalini Dam and Hydropower Scheme: Report	
Volume 4: Lalini Dam and Hydropower Scheme: Appendices	
Topographical Surveys	P WMA 12/T30/00/5212/11
Feasibility Design: Ntabelanga Dam	P WMA 12/T30/00/5212/12
Bulk Water Distribution Infrastructure	P WMA 12/T30/00/5212/13
Regional Economics	P WMA 12/T30/00/5212/14
Cost Estimates and Economic Analysis	P WMA 12/T30/00/5212/15
Legal, Institutional and Financing Arrangements	P WMA 12/T30/00/5212/16
Record of Implementation Decisions: Ntabelanga Dam and Associated Infrastructure	P WMA 12/T30/00/5212/17
Hydropower Analysis: Lalini Dam	P WMA 12/T30/00/5212/18
Feasibility Design: Lalini Dam and Hydropower Scheme	P WMA 12/T30/00/5212/19
Record of Implementation Decisions: Lalini Dam and Hydropower Scheme	P WMA 12/T30/00/5212/20


PHASE 1:
PRELIMINARY PHASE
Inception Period
INCEPTION REPORT
P WMA 12/T30/00/5212/1
STAGE 1: DESKTOP STUDY (SCREENING OF 19 DAM SITES TO 3)
Used in Desktop Screening Process
ENVIRONMENTAL SCREENING OF DAM SITES
STAGE 2: PRELIMINARY STUDY OF SELECTED 3 SITES (INCLUDING DETAILED HYDROLOGY, SURVEY AND INITIAL DRILLING)
ENVIRONMENTAL SCREENING REPORT
P WMA 12/T30/00/5212/2
PRELIMINARY STUDY REPORT
P WMA 12/T30/00/5212/3
PHASE 2:
FEASIBILITY STUDY
FEASIBILITY STUDY: MAIN REPORT
P WMA 12/T30/00/5212/4
SUPPORTING REPORTS
WATER RESOURCES
P WMA 12/T30/00/5212/5
WATER REQUIREMENTS
P WMA 12/T30/00/5212/6
HYDROPOWER ANALYSIS: LALINI DAM
P WMA 12/T30/00/5212/18
RESERVE DETERMINATION
P WMA 12/T30/00/5212/7
LAND MATTERS
P WMA 12/T30/00/5212/8
FEASIBILITY DESIGN: LALINI DAM AND HYDROPOWER SCHEME
P WMA 12/T30/00/5212/19
IRRIGATION DEVELOPMENT
P WMA 12/T30/00/5212/9
GEOTECHNICAL INVESTIGATIONS
P WMA 12/T30/00/5212/10
RECORD OF IMPLEMENTATION DECISIONS: LALINI DAM AND HYDROPOWER SCHEME
P WMA 12/T30/00/5212/20
TOPOGRAPHICAL SURVEYS
P WMA 12/T30/00/5212/11
FEASIBILITY DESIGN: NTABELANGA DAM
P WMA 12/T30/00/5212/12
BULK WATER DISTRIBUTION INFRASTRUCTURE
P WMA 12/T30/00/5212/13
REGIONAL ECONOMICS
P WMA 12/T30/00/5212/14
COST ESTIMATES AND ECONOMIC ANALYSIS
P WMA 12/T30/00/5212/15
LEGAL, INSTITUTIONAL AND FINANCING ARRANGEMENTS
P WMA 12/T30/00/5212/16
RECORD OF IMPLEMENTATION DECISIONS: NTABELANGA DAM AND ASSOCIATED INFRASTRUCTURE
P WMA 12/T30/00/5212/17
REPORTING STRUCTURE
AND STUDY PHASING

REFERENCE

This report is to be referred to in bibliographies as:

Department of Water and Sanitation (2014). ***Feasibility Study for the Mzimvubu Water Project: Geotechnical Investigations: Lalini Dam and Hydropower Scheme***

DWS Report No: P WMA 12/T30/00/5212/10

Prepared for: Directorate – Options Analysis

Prepared by: Jeffares & Green (Pty) Ltd, P O Box 794, Hilton, 3245
Tel: 033 343 6700, Fax: 033 343 6701
Contact: Mr A Pepperell
Email: pepperella@jgi.co.za

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

The Mzimvubu Water Project has been commissioned by the Department of Water and Sanitation to harness the resources of the Mzimvubu River catchment in developing the Eastern Cape region by means of multi-component schemes, including industrial and domestic water supply, irrigation and hydropower generation. The study described in this report pertains to a feasibility level geotechnical investigation undertaken for a proposed dam and hydro-power scheme on the lower Tsitsa River in the Lalini area, which falls within the Mhlonhlo Local Municipality.

The dam alignment investigated in this study was delineated during a site visit undertaken in mid-May 2014. At this preliminary stage in the design, the dam type and configuration had not been confirmed with certainty, proposed as either a roller compacted concrete (RCC) dam with a central spillway or an earth embankment dam with a side channel spillway cut into the left flank. It is understood that the RCC option is preferred. The final dam height was expected to be between about 50 m and 70 m, depending upon a number of interrelated factors.

The hydro-power component of the project is also in the conceptual stages of design and a number of alternative horizontal and vertical alignments have been proposed. It is understood that the preferred option is part pipeline and part tunnel, for which an alignment was initially proposed, on which the scope of this investigation has been based. A deeper vertical alignment has subsequently also been under consideration. Due to the fact that a number of alternative alignments are under consideration, the geotechnical investigation for the hydro-power component of the project was structured to provide an overall appraisal of geotechnical conditions over the general area under consideration for the tunnel route, but concentrating on that which was favoured at the time.

The feasibility level geotechnical investigation of the proposed Lalini Dam and Tunnel entailed the following:

1. *The drilling of four rotary core boreholes along the proposed alignment of the dam axis, two on the left flank and two on the right flank. Dolerite outcrop occurs across the river section.*
2. *The drilling of seven boreholes for the proposed hydro-power scheme, of which four were positioned along or adjacent to the preferred horizontal alignment, one just below the dam to cater for the pipeline section or an alternative tunnel alignment and one to the south west of the preferred tunnel alignment to cater for an alternative longer and deeper tunnel option. Five of the boreholes were inclined 5° off vertical to facilitate the undertaking of core orientation measurements.*
3. *The drilling of six boreholes in an identified potential rock quarry site.*
4. *A co-ordinated trial pitting investigation of identified potential borrow pits for earth embankment construction.*
5. *The excavation of trial pits along the proposed pipeline alignment.*
6. *Water pressure tests were conducted at representative intervals in all the dam boreholes and in one tunnel borehole.*
7. *Rock strength tests were conducted on representative borehole core samples, either by means of laboratory unconfined compressive strength (UCS) tests or point load strength index (PLSI) tests conducted on site.*
8. *Representative samples were retrieved of the unconsolidated materials proposed for earthfill dam construction to facilitate testing and analysis.*
9. *Water samples were retrieved from selected boreholes and from the Tsitsa River, the former for chemical aggressiveness testing and the latter to assess suitability for use in construction.*
10. *Associated rock exposure mapping and photography.*

The extent of the geotechnical investigations undertaken along the proposed dam axis have concluded that the site is suitable for the construction of either an earth-embankment dam or a RCC dam, albeit with relatively deep foundation excavation. Based upon the drilling undertaken the foundation invert will vary from between 6 m and 8 m on the upper flanks to between 3 m and 4 m on the lower flanks. Dolerite outcrops across the river section, implying negligible excavation in this area. The results of water pressure tests indicate that minor under-seepage is likely and that a cut-off grout curtain will be required. The need for consolidation grouting was not conclusively proven.

The reconnaissance for dam construction materials concentrated on areas falling within the future impoundment basin in order to avoid the negative environmental impacts and rehabilitation requirements associated with exploitation outside of the impoundment area. The area investigated as a potential rock quarry lies on the left hand or eastern side of the Tsitsa River, approximately 3.5 km upstream of the dam site. The investigation did prove good quality dolerite, but occurring beneath an excessively thick overburden mantle of unconsolidated, weathered and fractured materials. As a result of this, under normal circumstances the site would be regarded as being marginal for use as a rock quarry, but the use of the overburden materials in road construction, if found suitable, could mitigate the use of the area as a rock quarry. The investigation of road construction materials did not form part of the current geotechnical investigation, but it is a requirement of the overall project.

The naturally occurring sand in the channel of the Tsitsa River was found to be too finely graded for use as either concrete fine aggregate or filter medium. Its use would necessitate blending with an inert crushed rock product. Alternatively sand would have to be acquired from an approved off-site source.

Suitable core material was proved in adequate quantities, a short distance upstream of the dam within the impoundment basin. The area investigated as a shell borrow pit lies immediately upstream of the dam, with geology comprising mudrock and intercalated sandstone. The material tested is coarse grained, but with plastic fines, due to the preponderance of mudrock. The use of a tractor-loader-backhoe (TLB) in the investigation also limited the efficiency of excavation in this material and the volumes proved do not meet the volume requirements for shell. Based upon observations made on site the shell requirements, with further detailed assessment, can be optimised in terms of quality and quantity.

The favoured arrangement and alignment of the tunnel at the time of the investigation entailed a pipeline from the dam to the tunnel inlet portal. The length of the pipeline is approximately 3.5 km. The tunnel is approximately 4.4 km in length. A second pipeline would then convey water from the tunnel outlet portal to the hydropower plant generating substation. A subsequent proposal now under consideration is to eliminate the second pipeline by deepening the vertical alignment of the tunnel to exit at the hydropower plant. As a result of the drilling having concentrated on the first mentioned option, the majority of the boreholes were terminated above the tunnel zone of the second alignment option. The predominant geology encountered in the tunnel boreholes was sandstone with silty inter-beds and lesser dolerite. The boreholes drilled indicate that for the alignment favoured at the time, the tunnel would pass predominantly through laminated and inter-bedded sandstone. Through the tunnel zone of the upper alignment, the rock is competent with a Rock Mass Rating (RMR) value of about 70, based on a drill and blast 4 m high, horse-shoe shaped tunnel section. Finite element analyses indicate minor degrees of instability associated with the rock structure, requiring nominal support in the form of shotcrete and selected rock-bolting.

TABLE OF CONTENTS

APPENDICES

(See separate Volume 4 of
Report No. P WMA 12/T30/00/5212/10)

FIGURES

TABLES

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
RMR	Rock Mass Rating
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
Elevation	m a.s.l.
Height	m
Distance	m, km
Dimension	mm, m
Area	m^2 , ha or km^2
Volume (storage)	m^3
Yield, Mean Annual Runoff	m^3/a
Rotational speed	rpm
Head of Water	m
Pressure	Pa
Diameter	mm or m
Temperature	^oC

Description	Standard unit
Velocity, speed	m/s, km/hr
Discharge	m^3/s
Mass	kg, tonne
Force, weight	N
Gradient (V:H)	%
Slope (H:V) or (V:H)	1:5 (H:V) or 5:1 (V:H)
Volt	V
Power	W
Energy used	kWh
Acceleration	m/s^2
Density	kg/m^3
Frequency	Hz

1. INTRODUCTION

The Mzimvubu River catchment in the Eastern Cape Province 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:

- Forestry;
- 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 was 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.

It was confirmed and agreed that the sizing and modus operandi of the Lalini Dam and its associated works would take into account its main role, namely:

- to generate hydropower locally at the dam wall and some 7 km downstream of the dam; and
- to provide sufficient flow of water downstream of the Lalini Dam to meet environmental water requirements for an ecological Class B/C.

Hydropower generation potential investigated in this study stemmed from the above referenced reports. The development of a second dam at Lalini together with its hydropower infrastructure, which dam would be operated conjunctively with the Ntabelanga Dam, is a result of energy output optimisation. The purpose of this second dam and hydropower scheme would be to sell energy into the ESKOM grid, thus generating a net positive income stream which would be used to subsidise the energy and operating costs of the main Ntabelanga water supply and irrigation scheme, thus providing self-sustainability.

1.1 Lalini Dam Location

The preferred site is at a narrowing neck of the Tsitsa River approximately 3.5 km along the river centreline upstream of the Tsitsa Falls, at co-ordinates: 31°15' 44.76"S, 28°55' 15.87"E.

It was concluded that there were no better upstream dam wall locations available with regard to river valley shape (which affects dam wall length), geology/founding conditions, close proximity to construction materials, and the depth verses volume characteristics of the impoundment.

This location also offered several different options for hydropower configurations which are described in Report Nos. P WMA 12/T30/00/5212/18 and 19.

1.2 Purpose of this Report

This report presents the results of a feasibility level geotechnical investigation undertaken for the proposed Lalini Dam and hydropower scheme on the Tsitsa River.

The feasibility level geotechnical investigation entailed limited drilling along the dam centre-line and proposed tunnel route, as well as reconnaissance and investigations to identify sources of dam construction materials.

The dam centre-line investigation entailed the drilling of two vertical boreholes on each flank to depths of about 25 m. Water pressure tests were conducted at intervals down the boreholes and representative core samples were subjected to rock strength testing and mineralogical analyses.

A potential rock quarry site was investigated by the drilling of six boreholes, vertically to depths of between 10 m and 15 m. Identified core and shell sources were investigated by the excavation of trial pits, using a tractor-loader-backhoe (TLB). Samples of the materials were retrieved from the trial pits and submitted for laboratory analysis. Sand was sampled from the channel of the Tsitsa River.

The tunnel geotechnical investigation entailed the drilling of seven boreholes, both vertical and inclined, and varying in length between 20 m and 150 m.

2. DESCRIPTION OF THE PROJECT AREA

2.1 General

The locality of the Ntabelanga Dam and Lalini Dam and the proposed arrangement of the conjunctive hydropower scheme is indicated on Figures 2-1 and 2-2.

The project area is located in the Eastern Cape Province and falls within the Mhlontlo Local Municipality, which forms part of the greater OR Tambo District Municipality. It lies approximately 8 km to the east of National Route 2 where it crosses the Tsitsa River Bridge, approximately 16 km south-east of Qumbu and approximately 20 km north-east of Tsolo. The dam site is located between the villages of Lalini and Lotana. A short section near to the start of the proposed tunnel runs beneath the eastern outskirts of Lotana.

There are two main accesses into the site from the N2. The first is via a surfaced road to the village of Lalini that intersects the N2 immediately north of the Tsitsa River Bridge. The second is along a gravel road to the village of Lotana that intersects the N2 diagonally opposite the N2 / Main Road R396 intersection.

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

2.2 Climate

Climatic data for the nearby village of Shawbury is summarised in Table 2-1. The area has a warm and temperate climate with rainfall all year, but distributed predominantly over the summer months from October to March. The difference between the driest and the wettest month is 105 mm. The mean annual rainfall is 838 mm.

Table 2-1: Climatic Data for Shawbury

Climatic Data for Shawbury Village			
Month	**Average Rainfall (mm)**	**Average Minimum Temperature (°C)**	**Average Maximum Temperature (°C)**
January	122	14.1	25.1
February	122	14.4	25.2
March	116	13.2	24.3
April	47	10.1	22.3
May	30	6.8	20.5
June	17	3.9	18.3
July	19	3.8	18.2
August	24	5.4	19.9
September	48	7.9	21.4
October	73	9.8	22.1
November	103	11.7	23.1
December	117	13.0	24.3
Annual	**838**		

Information extracted from Climate-Data.org

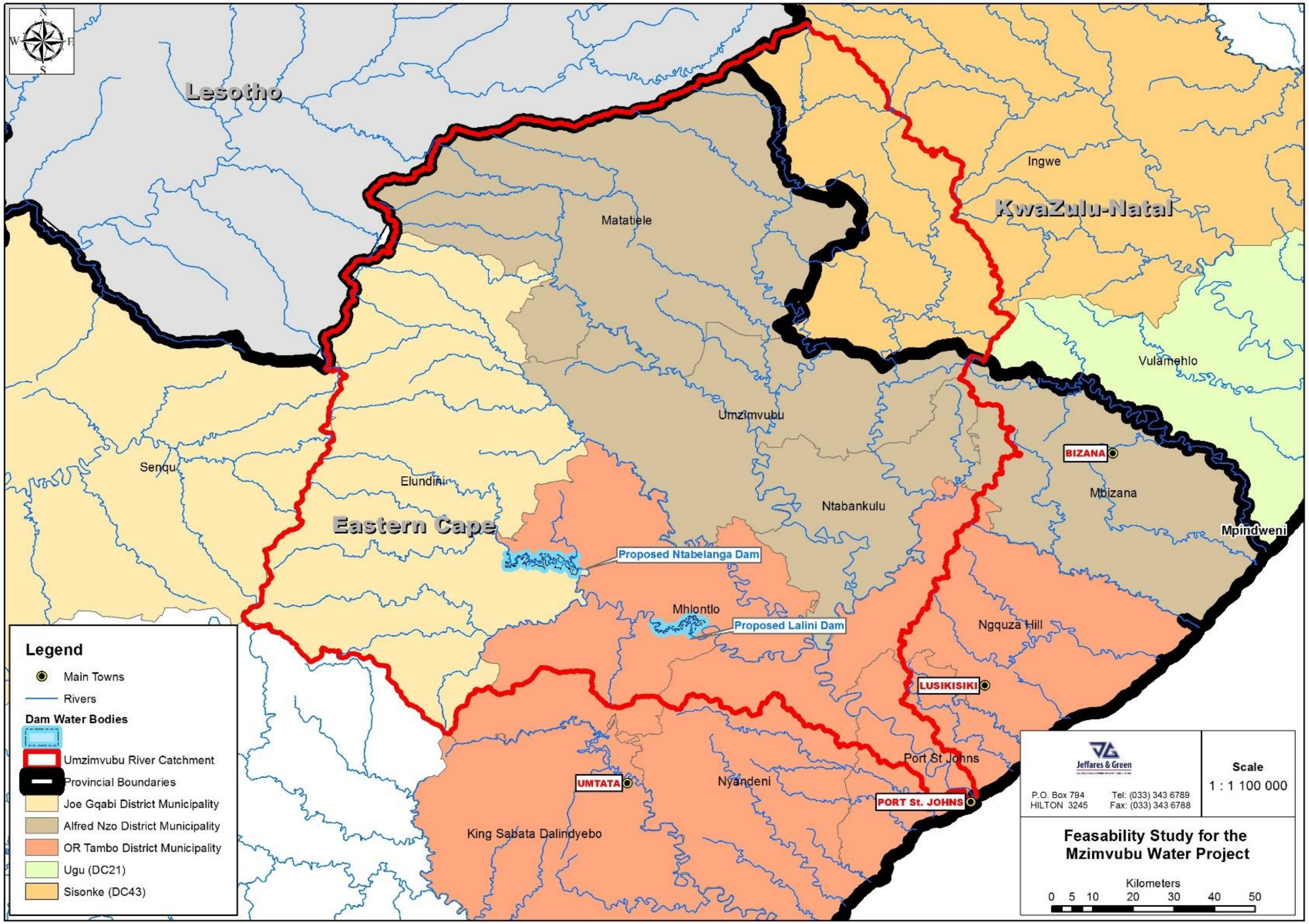


Figure 2-1: Locality of the Lalini Dam

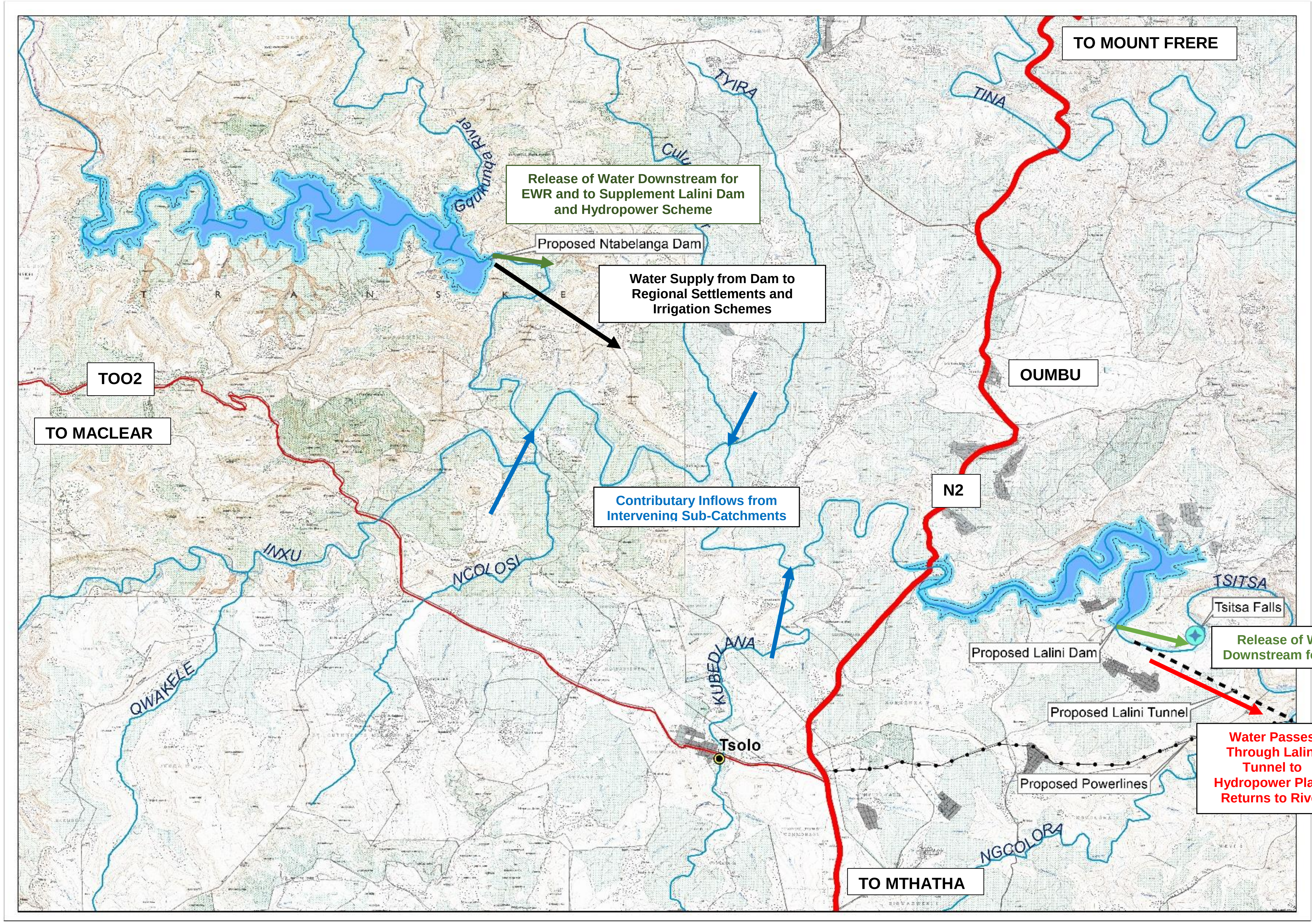


Figure 2-2: Ntabelanga and Lalini Dam and Hydropower Scheme Locality Plan

2.3 Topography

Upstream of the Tsitsa Falls, the Tsitsa River flows in a relatively shallow, but narrow valley. The dam axis is across a valley constriction formed by a transecting dolerite sill that outcrops in the bed of the river and forms a relatively flat grade control in the river profile. The river bed elevation on the dam axis is 720 m a.m.s.l. (above mean sea level). The basin topography upstream of the dam remains relatively steep, particularly on the northern side of the river, so that the dam will be confined to a relatively deep and narrow impoundment basin.

Below the Tsitsa Falls the river flows in a deep, steep sided ravine. From the dam the pipeline and tunnel alignment runs in an east-south-east direction. The elevation at the inlet portal is 707.5 m a.m.s.l.. The upper tunnel alignment runs at a constant declination of 0.3% from the inlet portal to an elevation of 697.6 m a.m.s.l. at the outlet portal. The outlet portal for the lower alignment is at an elevation of 441.3 m a.m.s.l., with a tunnel grade of 6.2%. The outfall from the hydropower plant lies downstream of a wide horseshoe meander of the Tsitsa River and it re-enters the river at an elevation of 406 m a.m.s.l..

2.4 Geology

The 1:250 000 Geological Series map 3128 Umtata (1979), indicates the project area to be underlain by sedimentary rocks of the Adelaide Subgroup of the Beaufort Group of the Karoo Supergroup and post-Karoo dolerite intrusions. The surrounding hills to the north and west of the dam site are capped by sedimentary rocks of the Tarkastad Subgroup, also of the Beaufort Group, Karoo Supergroup. An extract of the geological map is presented as Figure 2 in Appendix A.

According to Johnson et al (2006), the Adelaide Subgroup in the area comprises alternating mudrock and sandstone, deposited in a fluvial environment. Sandstone generally constitutes 20% to 30% of the total thickness, in the range of 10% to 60% in places. The sandstone units, according to Johnson et al (2006), generally vary in thickness from a few metres up to about 60 m. In this investigation, some sandstone units were thicker than this in certain boreholes drilled along the tunnel alignment. The sandstone intersected in the investigation boreholes comprised grey, medium grained, laminated sandstone inter-banded with brownish grey, fine grained sandstone and siltstone.

Dolerite is an intrusive, hyperbyssal igneous rock of post-Karoo age that has intruded the sedimentary host rocks, mainly in the form of concordant sills and to a lesser extent as discordant dykes. It is a dark grey, crystalline, rock composed mainly of plagioclase feldspar and pyroxene, with accessory amounts of olivine, biotite, amphibole, apatite and iron ore minerals.

Whilst generally of medium grained texture, the dolerite adjacent to the sedimentary contacts is often of a finer texture due to rapid cooling of the magma. The intrusions have also frequently resulted in the formation of an alteration or “baked” zone in the sedimentary rocks adjacent to the contacts. The joints in the dolerite are in most cases filled or coated by secondary calcite and chlorite, deposited by the subsequent circulation of magmatic fluids. It is the resistant nature of the dolerite sill at the Tsitsa Falls that has created the upper base level and pronounced grade control in the river, with a contrast in the morphology of the river valley upstream and downstream of the falls. Downstream of the falls the river valley shows marked incision and flows in a deep ravine-like gorge with almost precipitous valley sides.

The climatic N-value of the project area is less than 5, with a value of 2.3 for Umtata[1] (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 and under conducive topographic conditions will produce 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 mantle of residual and colluvial soils, the thickness of which is dependent upon topographic position and associated influences on deposition and erosion. The properties of the transported colluvial soils are dependent upon their mode of origin. Those of doleritic derivation comprise red or black clayey soils. The colluvial deposits originating from mudrock and sandstone are brown with secondary colouration of grey, yellow and red. They range in composition from gravely sand and clay to sandy and clayey gravel.

Alluvial sand occurs in the course of the Tsitsa River and certain tributaries. The sand displays a textural variation from medium grained sand in the river channel to fine grained sand on the banks, representing over-bank deposits.

The study of the geological maps, aerial photo stereo-pairs and field observations did not detect any tectonically induced linear structural features, such as faults, at either the dam site nor along the tunnel alignment.

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 100 years, for a maximum credible intensity (MCI) of xii (Fernandez and Guzman, 1979).

A detailed seismic hazard analysis of the region was conducted by Professor Kijko of the Natural Hazard Assessment Centre for the project region in this reach of the Tsitsa River. The conclusion in his report entitled “Probabilistic Seismic Hazard Analysis for the Mzimvubu Dam Site, The Eastern Cape”, was that according to the applied guidelines this region is rated as low risk.

This report is contained in Appendix D of the Feasibility Design: Ntabelanga Dam Report No. P WMA 12/T30/00/5212/12.

[1] *Now known as Mthatha*

3. DAM GEOTECHNICAL INVESTIGATION

The geotechnical investigation along the proposed dam axis entailed the drilling of four rotary core boreholes, two on each flank, as indicated on Figure 3-1. The boreholes were drilled vertically to terminate at depths of about 25.00 m. Water pressure tests were carried out at selected depths within the boreholes.

Core sticks were also subjected to point load strength index (PLSI) tests on site and samples were retrieved for the purpose of undertaking unconfined compressive strength (UCS) tests and petrographic analyses. Joint orientation measurements were undertaken on the dolerite outcropping in the river section.

Various views of the dam site are depicted on Figures 3-2 to 3-4.

3.1 Left Flank Boreholes

Boreholes D1 and D2 were drilled on the mid-to-upper and the mid-to-lower left flank respectively. The borehole logs and photographs are presented in Appendix B and the drilling results are summarised in Tables 3-1 and 3-2.

From the Borehole D1 results, it is recommended that the foundation level should be at a depth of 8.08 m on medium to slightly weathered, medium hard rock, sandstone with a rest water level at 16.00 m.

UCS and PLSI Tests focussed on core samples retrieved between depths 8.12 m and 9.50 m, as this horizon will be below the recommended foundation level.

From the Borehole D2 results, it is recommended that the foundation level should be at a depth of 4.46 m on completely to highly weathered, very soft to soft rock, dolerite with a rest water level at 0.30 m.

Table 3-1: Left Flank Boreholes – Borehole D1

LITHOLOGY		WPT		PLSI		UCS	
Depth (m)	**Description**	**Depth (m)**	**Lugeon**	**Depth (m)**	**$I_{s\,(50)}$ (MPa)**	**Depth (m)**	**UCS (MPa)**
0.15	Dark brown to black, sandy clay, colluvium	-	-	-	-	-	-
2.59	Residual reddish brown, clayey sand & gravel	-	-	-	-	-	-
8.08	Olive grey-brown medium to slightly weathered, medium hard rock Sandstone. 200 mm alteration zone on bottom contact	10.00 - 16.30	7	-	-	-	-
16.3	Olive grey-brown, slightly weathered, medium hard to hard rock, Sandstone	-	-	8.12 9.46-9.5 9.5	1.3 1.5 1.8	-	-
18.75	Slightly to unweathered, hard to very hard rock, Dolerite	16.30 - 21.46 Water loss at highest pressure	8	-	-	-	-
25.39	Unweathered, very hard rock, Dolerite	21.46 - 25.39	4	-	-	-	-
Notes	Water rest level 16.00 m - Recommended foundation level 8.08 m						

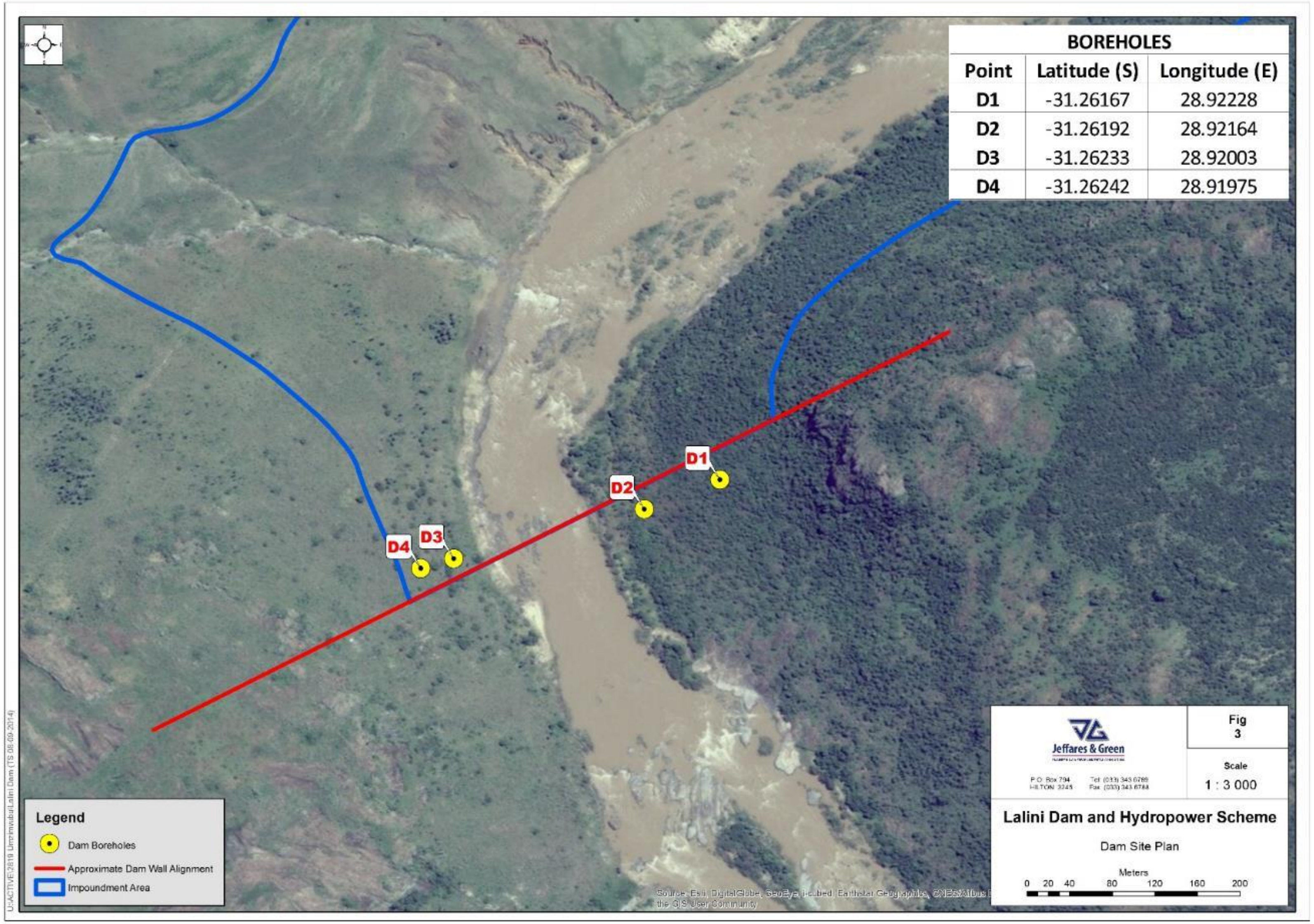


Figure 3-1: Locations of Boreholes Drilled on Dam Wall Centreline

Figure 3-2: The Dam Site Viewed from Upstream

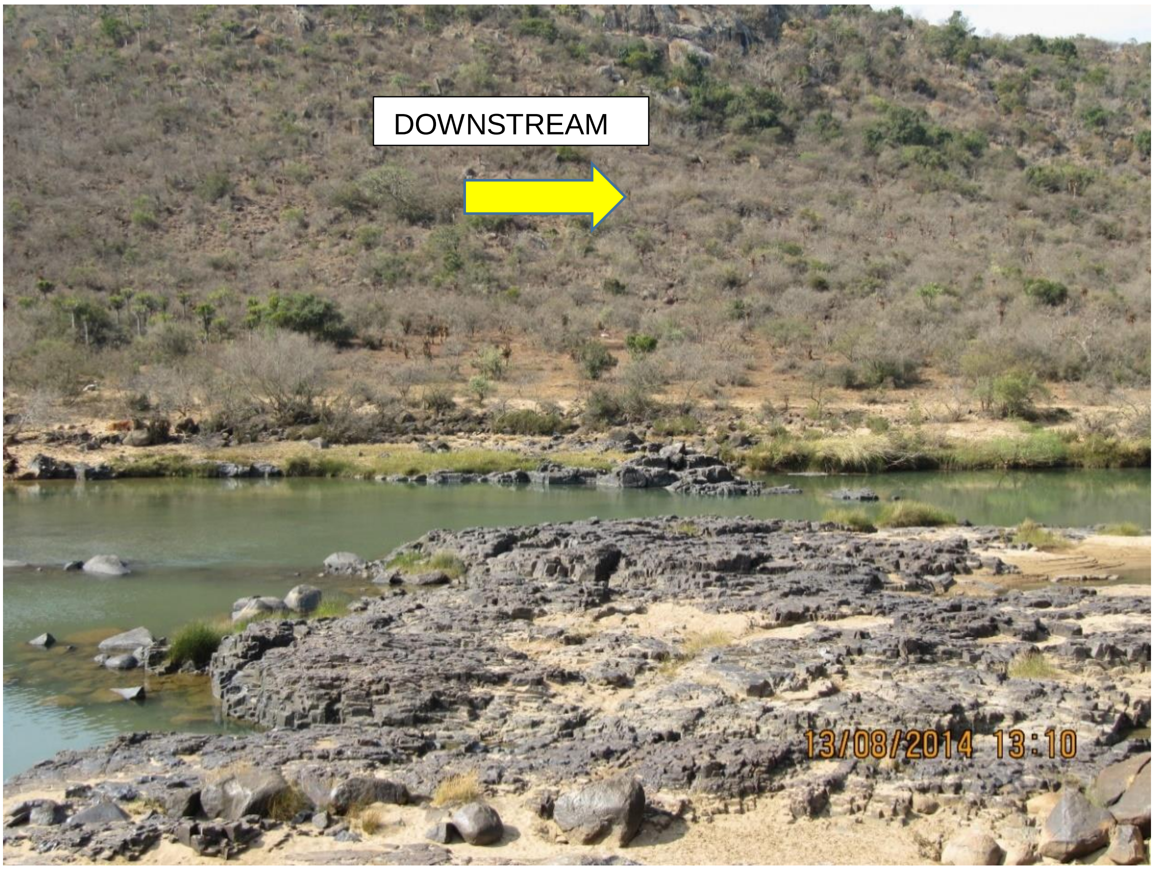


Figure 3-3: View of the River Section and Left Flank from the Right Bank.

Note: Dolerite outcrop across the river

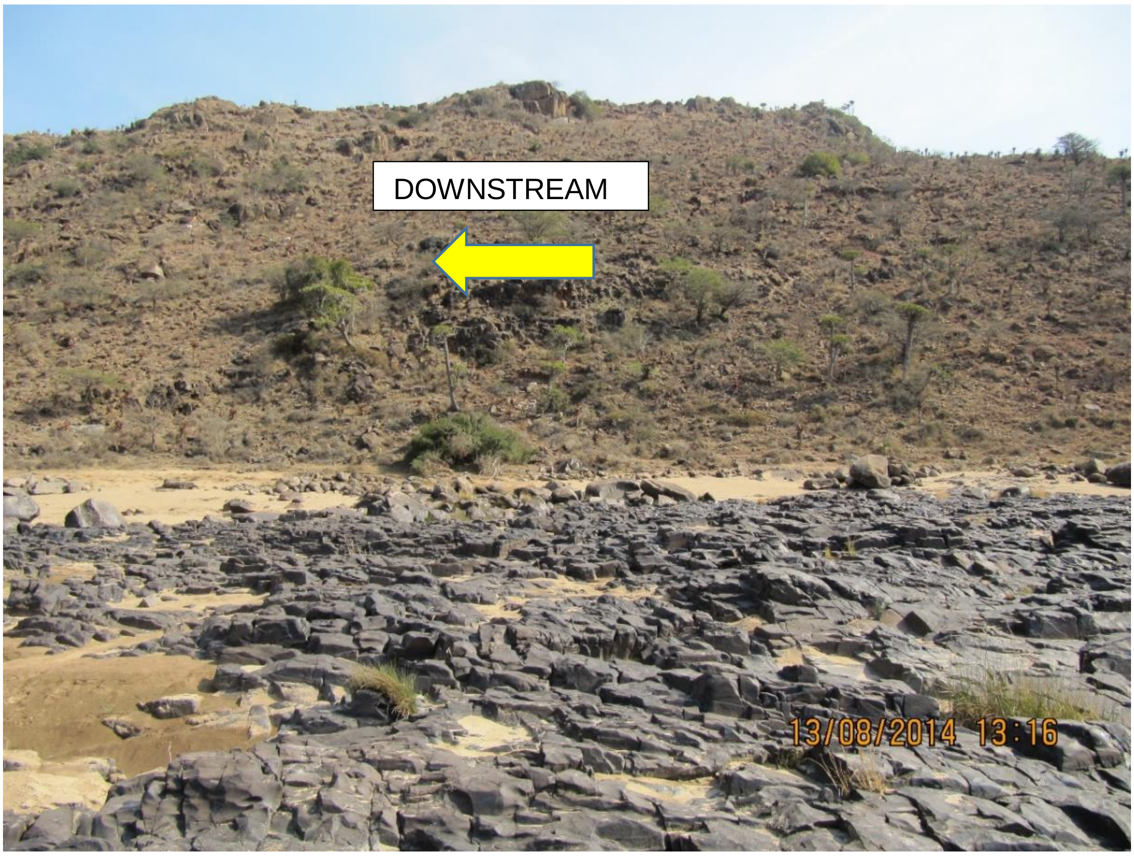


Figure 3-4: View of the Right Flank from Approximately the Middle of the River

UCS and PLSI Tests focussed on core samples retrieved between depths 4.46 m and 9.60 m, as this horizon will be below the foundation level.

The Lugeon test carried out at a depth between 18.50 m and 25.82 m were inconclusive due to water loss and gauge pressure loss. The likely result is packer leakage, as no significant fracture zones are apparent.

Table 3-2: Left Flank Boreholes – Borehole D2

LITHOLOGY		WPT		PLSI		UCS	
Depth (m)	**Description**	**Depth (m)**	**Lugeon**	**Depth (m)**	**$I_{S(50)}$ (MPa)**	**Depth (m)**	**UCS (MPa)**
0.88	Colluvial sandy clay	-	-	-	-	-	-
2.11	Residual sandy clay and gravel	-	-	-	-	-	-
4.46	Completely to highly weathered, very soft to soft rock, Dolerite	-	-	-	-	-	-
19.6	Slightly weathered, hard to very hard rock, Dolerite	7-12 12-18.35 18.5-25.82	23 2 Inconcl-usive	4.46-4.51 4.51 5.78 5.88	8.7 8.6 7.5 9.6	4.51-4.76	192.0
25.80	Slightly weathered to unweathered, hard to very hard rock, Dolerite	-	-	-	-	-	-
Notes	Water rest level 0.30 m Recommended foundation level 4.46 m Drilling water loss at 21.4 m WPT 18.5 - 25.82 m – inconclusive result due to water loss and gauge pressure loss. Likely the result of packer leakage, as no significant fracture zones.						

3.2 Right Flank Boreholes

Boreholes D3 and D4 were drilled on the mid-to-lower and the mid-to-upper right flank respectively. The borehole logs and photographs are presented in Appendix B and the drilling results are summarised in Tables 3-3 and 3-4.

From the Borehole D3 results, it is recommended that the foundation level should be at a depth of 3.19 m on highly weathered, medium to hard rock, sandstone with a rest water level at 14.3 m.

Tests focussed on core samples retrieved between depths 3.75 m and 13.5 m, as this horizon would be just below the foundation level.

Table 3-3: Right Flank Boreholes – Borehole D3

LITHOLOGY		WPT		PLSI		UCS	
Depth (m)	**Description**	**Depth (m)**	**Lugeon**	**Depth (m)**	**$I_{s\,(50)}$ (MPa)**	**Depth (m)**	**UCS (MPa)**
1.43	Gravely silty clay	-	-	-	-	-	-
2.06	Medium to slightly weathered, hard rock Dolerite	-	-	-	-	-	-
3.19	Highly weathered, medium hard to hard rock, Sandstone	-	-	-	-	-	-
6.35	Medium weathered, hard rock, Sandstone	3.5-7.22	17	3.75-3.8	5.4	3.8-3.98	110.9
13.52	Slightly weathered, hard to very hard rock, Dolerite	7.5-13.5	1	10.29 10.047	13.9 12.5	-	-
14.49	Slightly to unweathered, hard rock, Sandstone	13.5-24.2	1	-	-	-	-
16.93	Unweathered, hard to very hard rock, Dolerite		-	-	-	-	-
20.51	Unweathered, hard rock, Sandstone		-	-	-	-	-
20.97	Unweathered, hard to very hard rock, Dolerite		-	-	-	-	
21.49	Unweathered, hard rock, Sandstone		-	-	-	-	-
22.09	Unweathered, hard to very hard rock, Dolerite		-	-	-	-	-
24.20	Slightly weathered, hard rock, Sandstone		-	-	-	-	-
Notes	Water rest level 14.30 m Recommended foundation level 3.19 m						

From the Borehole D4 results, it is recommended that the foundation level should be at a depth of 6.40 m on medium weathered, hard rock, dolerite with a rest water level at 20.00 m. UCS and PLSI Tests focussed on core samples retrieved between depths 6.40 m and 7.55 m, as this horizon will be below the recommended foundation level.

3.3 Dam Basin

The Tsitsa River in the area of future impoundment flows in a generally narrow, incised valley, so that the basin morphology is generally steep, particularly on the northern side. Aerial photo interpretation did not detect any tectonically induced lineaments in the basin that could lead to potential large scale slope failure. The steepness of the topography has the potential for localised downslope slumps and slides.

Table 3-4: Right Flank Boreholes – Borehole D4

LITHOLOGY		WPT		PLSI		UCS	
Depth (m)	**Description**	**Depth (m)**	**Lugeon**	**Depth (m)**	**$I_{s(50)}$ (MPa)**	**Depth (m)**	**UCS (MPa)**
2.12	Residual soil and boulders	-	-	-	-	-	-
3.13	Highly to medium weathered, medium hard rock, Sandstone	-	-	-	-	-	-
5.02	Highly to medium weathered, medium hard to hard rock, Dolerite	-	-	-	-	-	-
6.40	Medium weathered, hard rock, Dolerite	-	-	-	-	-	-
7.55	Slightly weathered, hard rock, Dolerite	-	-	6.47 7.23	5.3 7.1	6.67-6.8	145.2
24.81	Slightly weathered to unweathered, hard to very hard rock, Dolerite	15-20.5 20.5-24.81	13 Inconclusive	15.77	8.9	-	-
Notes	Water rest level 20.00 m Recommended foundation level 6.40 m Drilling water loss at 8.68 and 12.60. Grouted hole from 8.00 m to 14.63 m. WPT 20.5 - 24.81 m – inconclusive result due to water loss and gauge pressure loss. Either packer leakage or localised fracture zone (21.63 – 21.74 m).						

3.4 Joint Orientation Data for the Dolerite Bedrock

Mapping of the dolerite outcrop across the river section revealed three main sub-vertical joints and a sub-horizontal joint, as indicated on the stereographic plot in Figure 3-5.

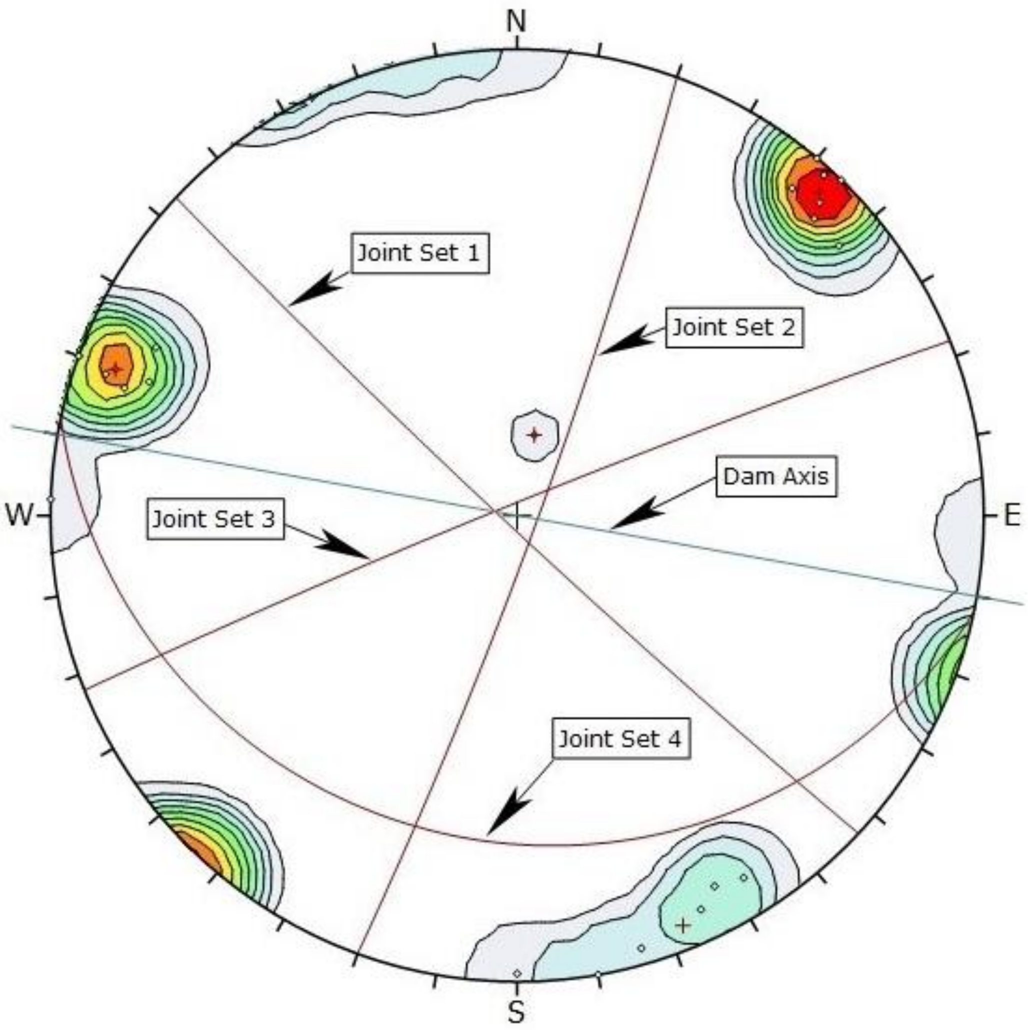
The mean dip and dip directions of the joint sets, depicted as great circles on Figure 3-5, are as follows:

The joint survey indicates that the 3 major joint sets are vertical to near vertical in character and they are orthogonal relative each other.

Table 3-5: Mean Joint Orientation Data

Joint Set No.	Dip	Dip Direction (relative to magnetic North)	Dip Direction (relative to true North)
1	87°	223°	249°
2	85°	110°	136°
3	87°	338°	364°
4	20°	192°	218°
Dam axis	-	100° / 280°	126° / 306°

The dam foundation excavation line and borehole log summary are depicted on Figure 3-6.



Symbol	Feature
◇	Pole Vectors

Color	Density Concentrations
	0.00 - 3.20
	3.20 - 6.40
	6.40 - 9.60
	9.60 - 12.80
	12.80 - 16.00
	16.00 - 19.20
	19.20 - 22.40
	22.40 - 25.60
	25.60 - 28.80
	28.80 - 32.00
Maximum Density	31.96%
Contour Data	Pole Vectors
Contour Distribution	Fisher
Counting Circle Size	1.0%

Plot Mode	Pole Vectors
Vector Count	23 (23 Entries)
Hemisphere	Lower
Projection	Equal Angle

Figure 3-5: Dolerite Joint Orientation Data on Dam Centre-line

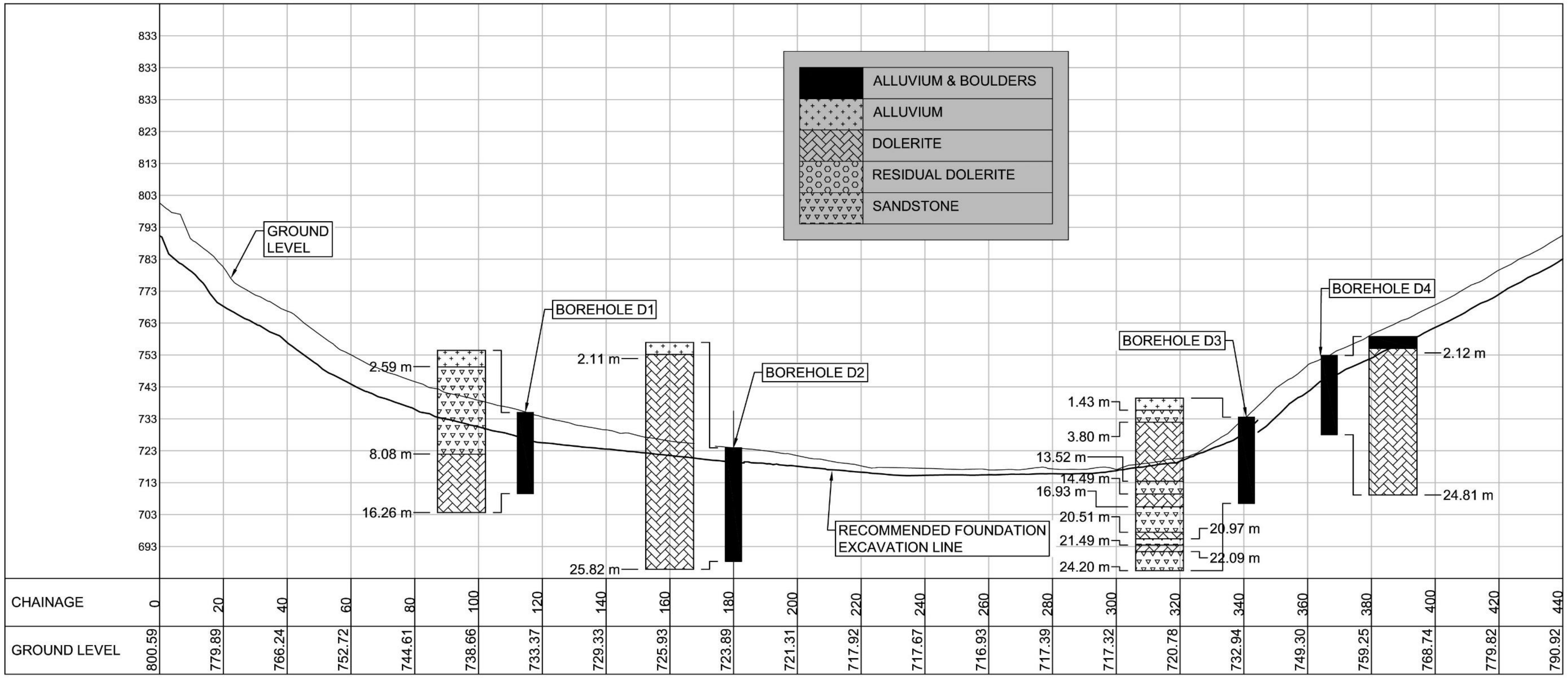


Figure 3-6: Borehole Log Summary along Dam Profile

4. CONSTRUCTION MATERIALS INVESTIGATION

The construction materials requirements for the two dam alternatives are summarised in Table 4-1.

Table 4-1: Construction Material Requirements

Material Zone	Required Quantities (m^3)		Proved at this Stage	Estimated Available
	Earth Fill Dam	RCC Dam	(m^3)	(m^3)
Concrete: coarse aggregate (rock)	650	200 000	400 000	400 000
Rip-rap	20 000	-		
Coarse filter	60 000	-		
Concrete: fine aggregate (sand)	350	100 000	960 000	> 960 000
Fine filter	60 000	-		
Core	280 000	-	1 000 000	1 000 000
Shell	1 400 000	-	740 000	> 1 400 000

In addition, the following materials will be required for the construction of roads in the project area:

G9 quality material: 25 965 m^3
G7 quality material: 59 310 m^3
G5 quality material: 37 700 m^3
G2 quality material: 21 500 m^3
Gravel wearing course material: 65 250 m^3

Road material quality specifications according to TRH 14 (1985) and TRH 20 (1990).

The locations of the potential materials sources investigated are indicated on Figures 4-1 to 4-5.

4.1 Rock Aggregate Quarry

The reconnaissance for a rock aggregate quarry aimed at locating a dolerite source within the future impoundment basin, which was also within close proximity and easily accessible from the dam site. This, mainly in order to minimise haulage costs, environmental impacts and subsequent rehabilitation costs.

The area investigated is located approximately 3.5 km upstream of the dam on the eastern side of the Tsitsa River, as indicated on Figure 4-2.

The investigation of the site entailed the drilling of six boreholes to depths of between 10 m and 15 m.

The borehole location co-ordinates, logs and photographs are presented in Appendix C of the volume accompanying this report.

Representative samples of the core were retrieved for UCS testing, petrographic analyses and ethylene glycol durability tests. The results of the drilling and core testing are summarised in the following sub-sections.

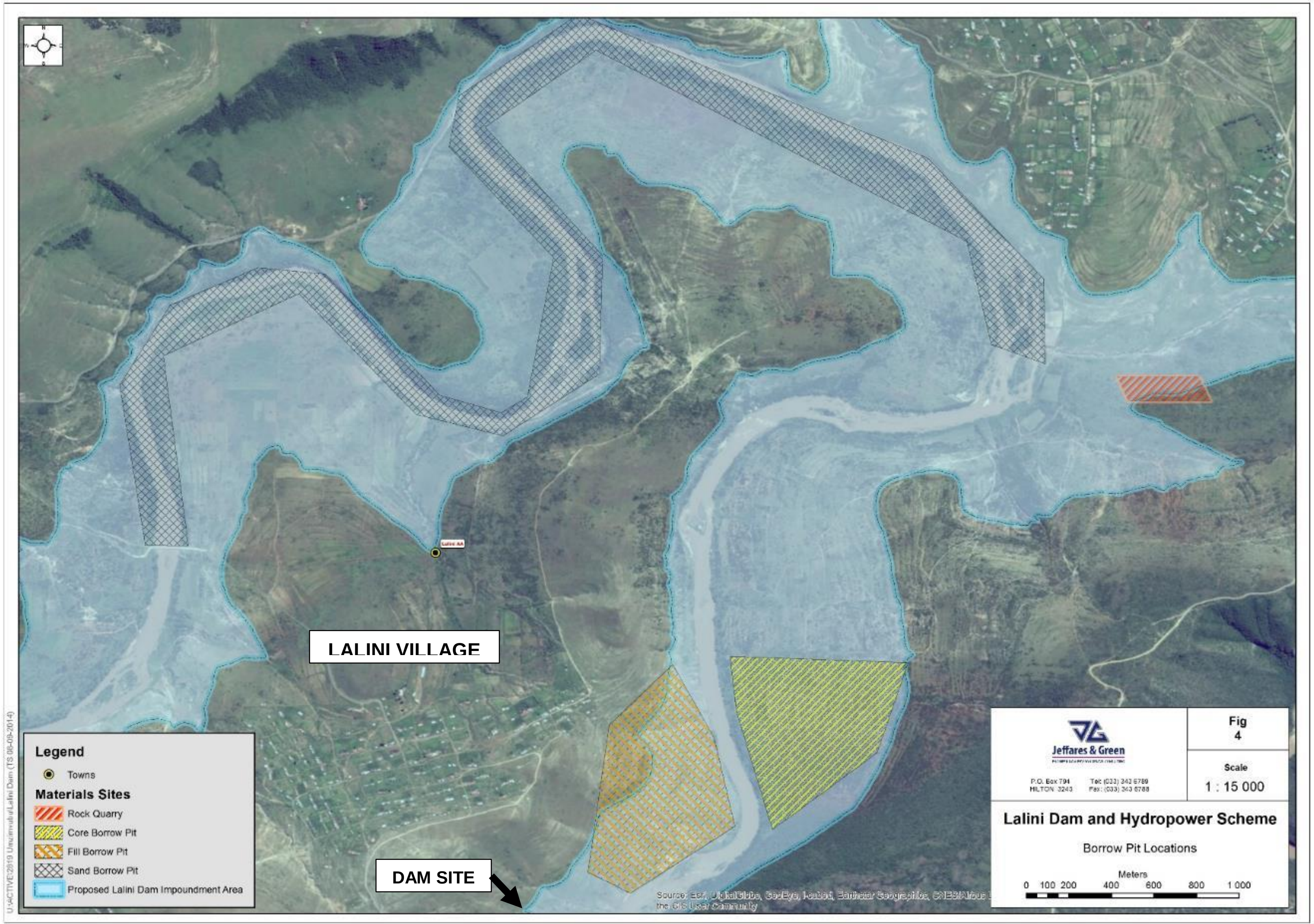


Figure 4-1: Borrow Pit Locations

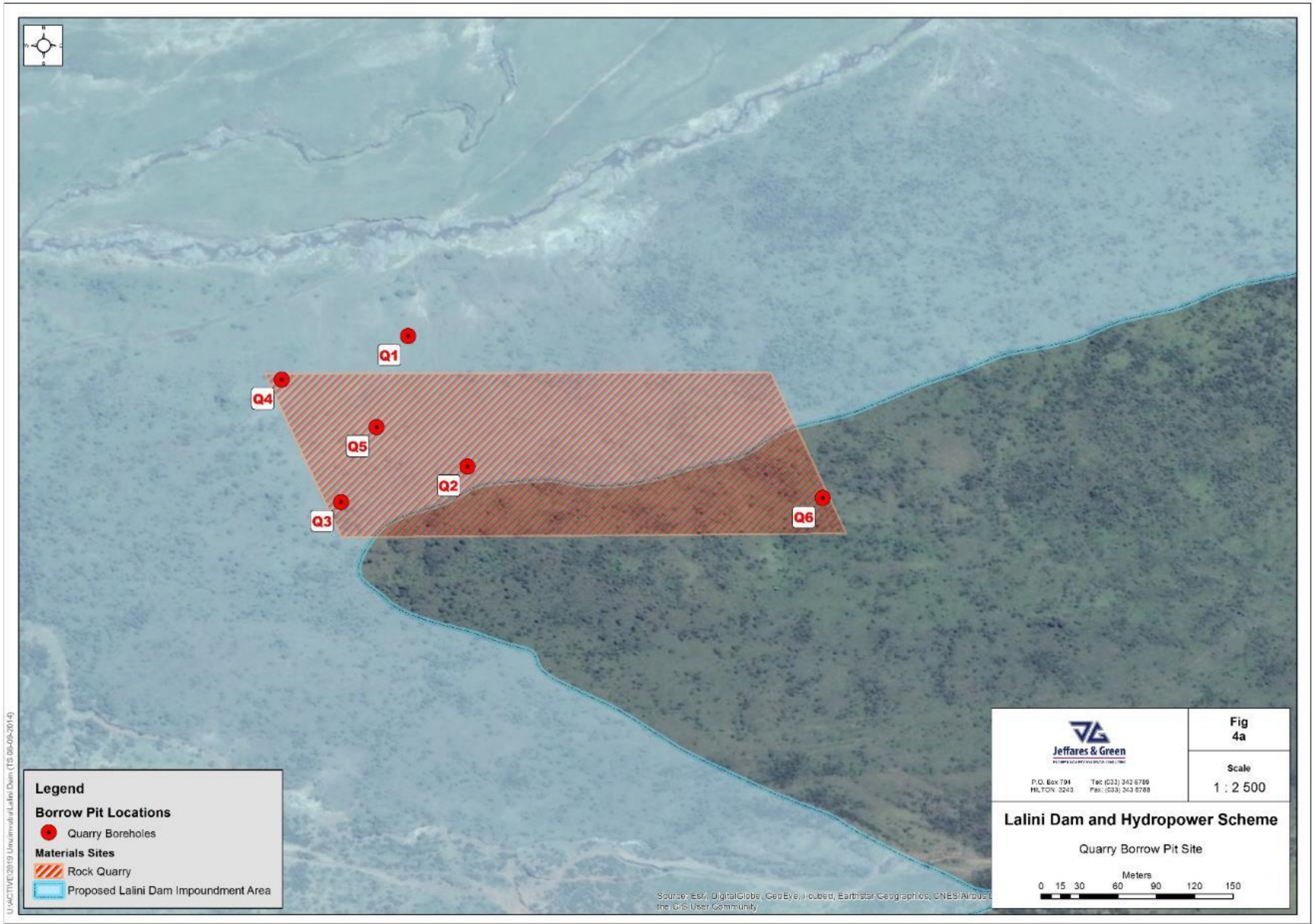


Figure 4-2: Rock Quarry Borehole Sites

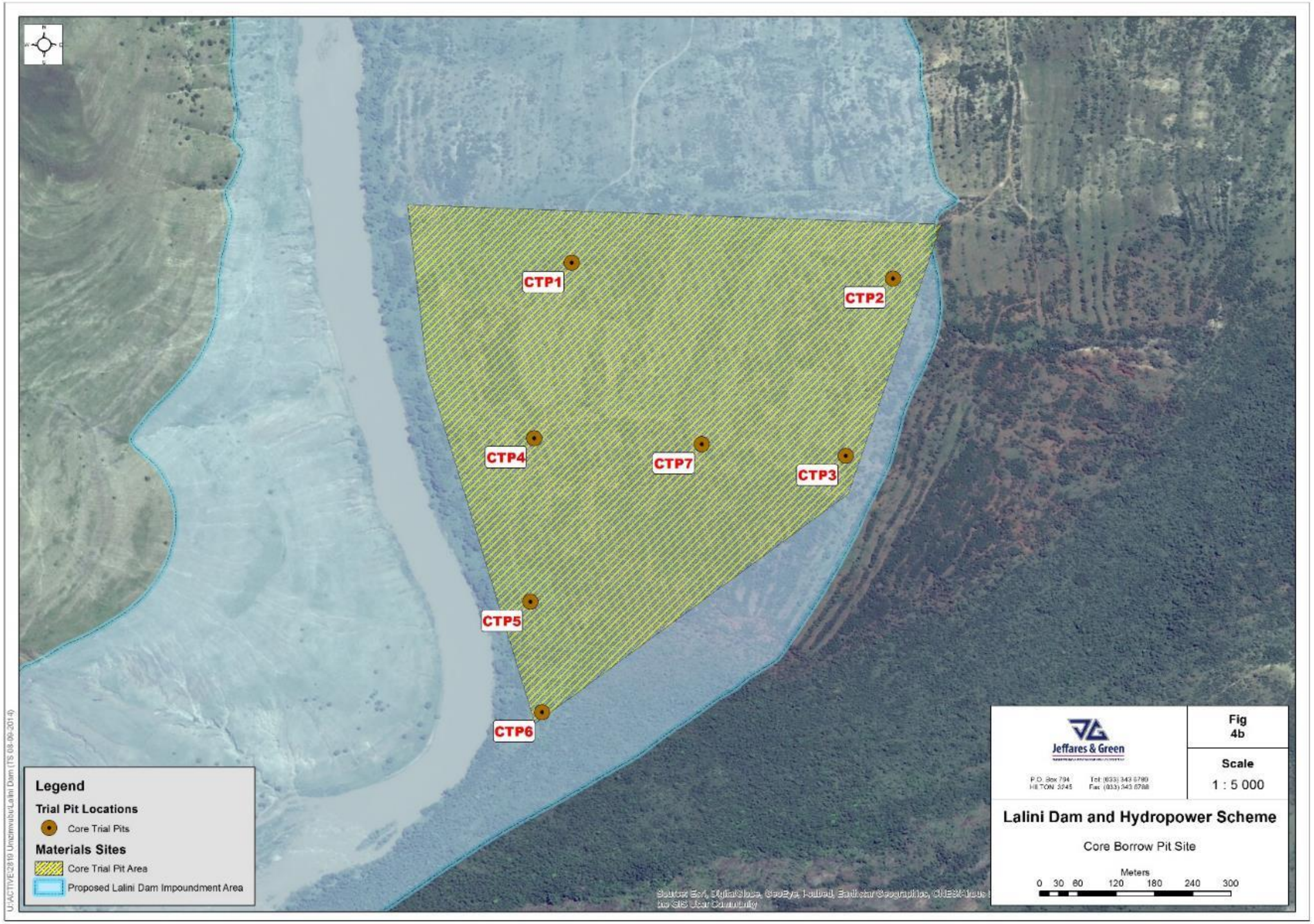


Figure 4-3: Core Borrow Trial Pit Locations

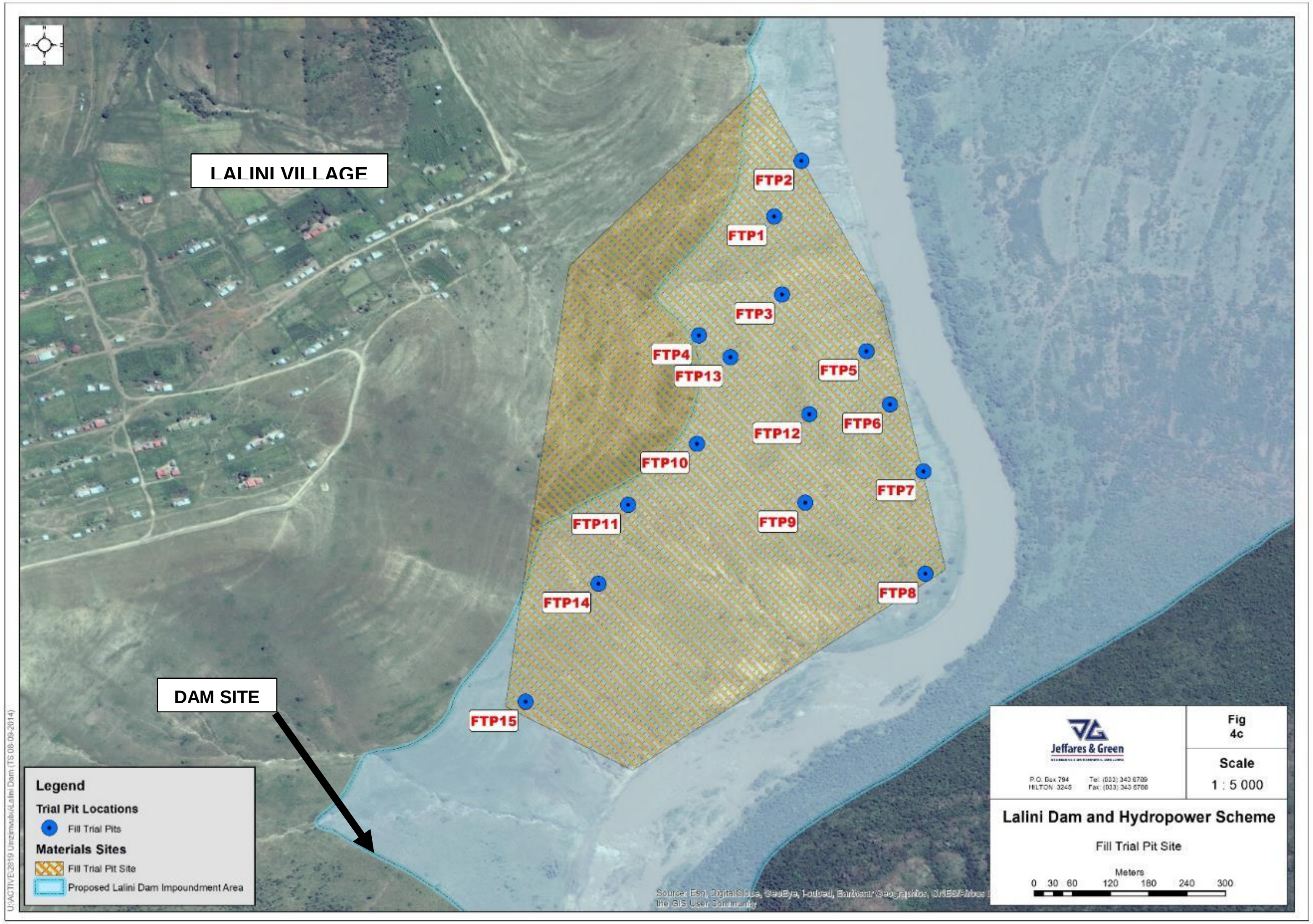


Figure 4-4: Embankment Fill Trial Pit Locations

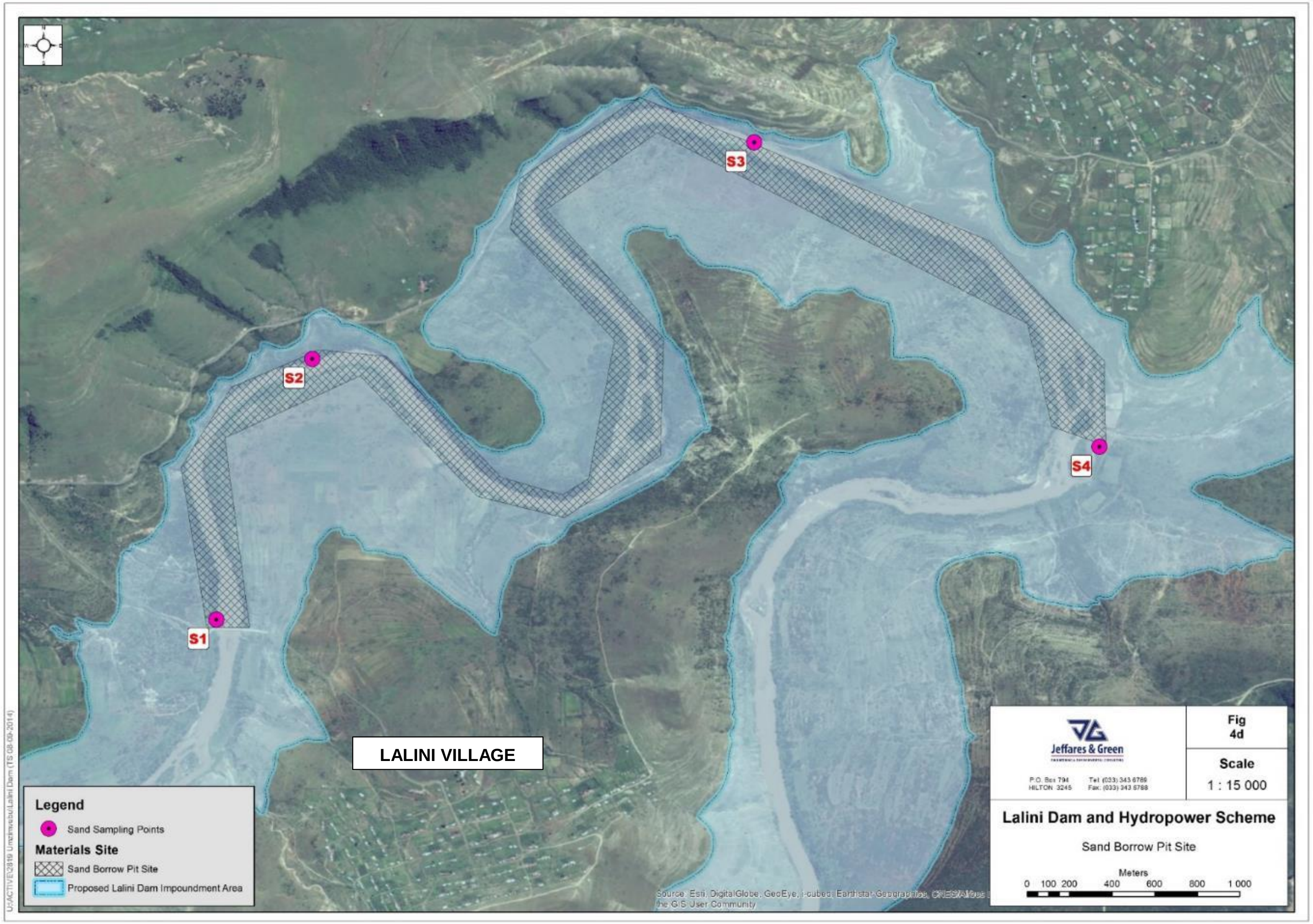


Figure 4-5: Sand Source Sampling Locations

4.1.1 Description of Rock Profile

The borehole profiles are summarised in Figure 4-6.

The boreholes drilled generally show a deep weathering profile over the area investigated with a thick overburden mantle, which under normal circumstances would render the site marginal to unsuitable for exploitation as a rock quarry, due to the excessive thickness of unusable overburden material that would require removal and spoiling. In this case, the residual and weathered dolerite overburden has potential usage as road construction material, which if confirmed as being suitable could make the site feasibly exploitable. This would require verification by means of a more detailed investigation and testing programme.

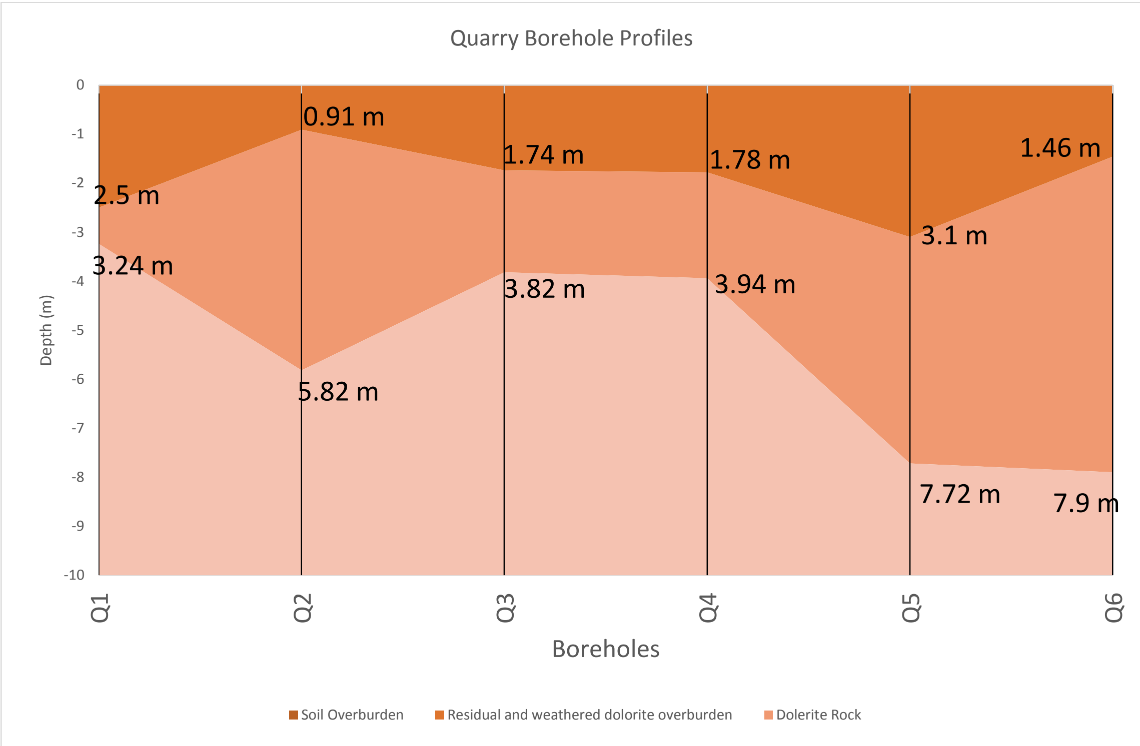


Figure 4-6: Schematic Rock Quarry Borehole Profiles

Once encountered the un-weathered dolerite is of good quality, as confirmed by the strength, mineralogical and durability tests undertaken. The estimated volume of good quality dolerite rock available for the manufacture of crushed rock aggregates, excluding poor quality overburden, is in excess of 400 000 m³ which meets the requirements for RCC and Earthfill dam types.

4.1.2 Unconfined Compressive Strength

The unconfined compressive strength (UCS) results on dolerite core samples from Boreholes Q2 and Q3 are presented in Table 4-2.

Table 4-2: Results of UCS Tests on Quarry Dolerite Samples

Borehole No.	Depth (m)	UCS (MPa)	Rock Strength
Q2	5.89 – 6.09	189.0	Very Hard Rock
Q3	4.49 – 4.63	170.6	Very Hard Rock

4.1.3 Petrographic and X-Ray Diffraction (XRD) Analyses

The analyses undertaken on a dolerite sample from Borehole Q2 between 6.24 and 6.36 m indicate a rock composed predominantly of feldspar and pyroxene with minor amounts of quartz, mica and hornblende. It displays a low degree of alteration with no evidence of smectite[2] group minerals being present.

Results of laboratory tests and petrographic and XRD analyses are given in Appendix E which is contained in Volume 4 of this Report No. P WMA 12/T30/00/5212/10.

Figures 4-7 to 4-9 show views of the proposed quarry site and photographs of examples of the core recovered from the boreholes drilled.

Figure 4-7: View of the Proposed Quarry Site from the North

[2] *Smectite is a deleterious clay mineral that is sometimes present, which swells and can cause low durability in concrete made with such aggregate.*

Figure 4-8: Dolerite Outcrop below Borehole Q6.

Note: Borehole Q6 drilled on top of the hill

Figure 4-9: Section of Borehole Q3

Note: *Generally good quality dolerite. Fracturing and drilling breaks between 8.65 and 9.09m (end of last row).*

4.1.4 Ethylene Glycol Durability Testing

The ethylene glycol durability test is an index test used to identify rapid weathering in basic igneous rocks, such as dolerite. Such rocks undergo rapid weathering and breakdown under atmospheric conditions due to the presence of swelling clays of the smectite group infilling joints and micro-fractures. Immersion in ethylene glycol serves as a rapid indicator to distinguish between rapid weathering dolerite and stable dolerite, as it causes swelling of smectite minerals, which if present results in breakdown of the rock.

The results of the ethylene glycol durability tests corroborate the petrographic analyses in respect of an absence of smectite clay minerals in the dolerite. Whilst initially there appeared to be minor reaction in the samples from Borehole Q3 between 8.65 m and 9.09 m, subsequent monitoring failed to detect any further reaction and it is interpreted as a prevailing condition rather than one induced by swelling smectite mineral constituents.

4.2 Sand

A stretch of the Tsitsa River, as indicated on Figure 4-1, which lies within the impoundment basin was initially proposed as a potential sand source. Sand samples were retrieved from within the river channel at various locations (S1 – S4) along this section of the river, as indicated on Figure 4-5. The estimated volume of exploitable sand from this section of the river is approximately 960 000 m^3.

The laboratory test results carried out on the sand are presented in Appendix E3. The tests indicate that, chemically, the sand complies with the minimum requirements specified by SANS 1083 (2006) for fine concrete aggregate. The grading of the sand is finer than and falls outside of the envelope specified by SANS 1083 (2006) for concrete aggregate and in order to conform to the specification would require blending with coarser material, such as graded crushed rock aggregate.

A filter design was done for all the samples obtained from the core borrow pit. The base soil samples were obtained from test pits Core Trial Pit (CTP) 1 to (CTP) 7.

The sand particle size distribution for all the sand samples is shown in Figure 4-10 as indicated by the legend displayed below the graph. Based upon the core base material, the sand filter is required to fall within the lower and upper limits of particle size distribution shown by the black lines.

Figure 4-10 indicates that the sand grading falls outside of the upper limit set by the Federal Emergency Management Agency (FEMA) (2011) 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 envelope defined by the upper and lower limits.

In conclusion, the Tsitsa River sand occurring in the project area is too finely graded to comply with the specifications for both concrete fine aggregate and filter medium.

Solutions would include to importation of suitable sand from more distant sources, perhaps in the river further upstream, and/or the use of crusher sand to blend with the finer sand found in this river section.

The further geotechnical and materials investigations to be undertaken during the detailed design stage should include the identification of the best source of sand in this respect.

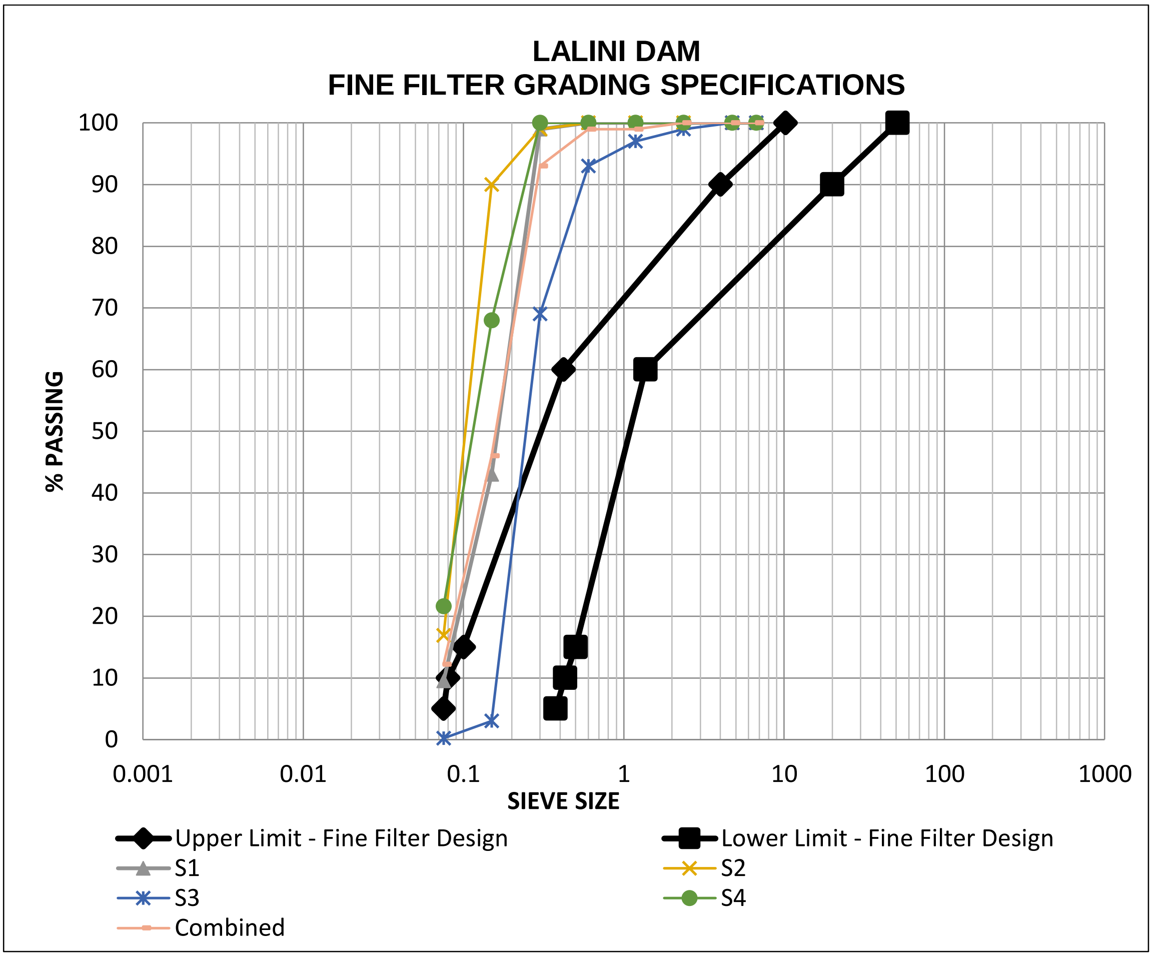


Figure 4-10: Filter Gradation Curves

4.3 Core Material

The proposed core borrow pit is located on the left flank, less than a kilometre upstream of the dam, as indicated on Figures 4-1 and 4-3.

The area of potential exploitation was investigated and delineated by means of trial pits, sampling and testing. Seven trial pits were excavated across the proposed borrow pit by means of a tractor-loader-backhoe (TLB).

The material comprises red-brown, sandy silty clay, colluvium of doleritic origin. The laboratory test results are presented in Appendix E4, and summarised in Table 4-3.

Grading and Atterberg Limits was done all samples and specialized testing was done on a combined sample comprising equal proportions on all samples.

Table 4-3: Core Material Borrow Test Results

Property	Trial Pit Number								
	CTP1	**CTP2**	**CTP3**	**CTP4**	**CTP5**	**CTP6**	**CTP7**	**Combined**	**Specification**
Passing 4.75 mm %	98	99 99	100	99 100	100	100	99	99	90-100
Passing 0.425 mm %	96	92 94	100	91 99	100	100	94	93	60-100
Passing 0.075 mm %	ok	ok ok	ok	ok ok	ok	ok	ok	ok	≥25% of percentage passes the 4.75 mm sieve
Passing 0.002 mm %	55	34 48	51	41 55	43	41	42	35	10-30* 10-40
Liquid Limit	50	36 47	52	43 48	41	44	47	47	25-60* 25-70
Plasticity Index	26	19 28	30	22 25	21	24	24	25	10-30
Linear Shrinkage	12	9.5 12.5	12.5	9.5 10.5	8	11.5	10.5	10.5	6-14
Plasticity Product	2 106	1 406 2 100	2 700	1 672 2 100	1 848	1 992	1 944	1 950	500-1 000 PI x% <0.075mm
Proctor OMC %	-	-	-	-	-	-	-	22.6	12-25
Proctor MDD Kg/m³	-	-	-	-	-	-	-	1 591	1 350-1 700
Potential. Expansive-ness	L-M	L M	M-H	L-M L	L-M	M-H	M	M-H	Ideally low
Permeability m/s	-	-	-	-	-	-	-	2.7×10^{-8}	1×10^{-9}
Dispersion	-	-	-	-	-	-	-	18 Non-dispersive	Ideally ND
Friction angle degrees	-	-	-	-	-	-	-	-	20-30
Cohesion kPa	-	-	-	-	-	-	-	-	15-24
Organic content %	Not tested – unlikely to be a problem as the material is not alluvium.								<2%

Specification according to Elges et al (1994), Melvill (2002) and Mouton (2010)

Plasticity Product = PI x percentage passing 0.075 mm sieve
MDD: Maximum Dry Density
** denotes different specifications by different authors*

OMC: Optimum Moisture Content
ND: Non-dispersive

The materials are fine grained with high clay contents. According to Table 4-3, the materials are suitable in most respects for use as core.

In addition to the specification limits in Table 4-3, according to the USBR (1974), the relationship between the liquid limit and plasticity index may also be used in evaluating the suitability of a material for use as core.

Figure 4-11 shows the samples from the proposed core borrow pit plotted on the Casagrande plasticity chart. The elliptical area represents the zone of suitability for use as core. All of the samples plotted within the elliptical area, implying that they are suitable for use as core.

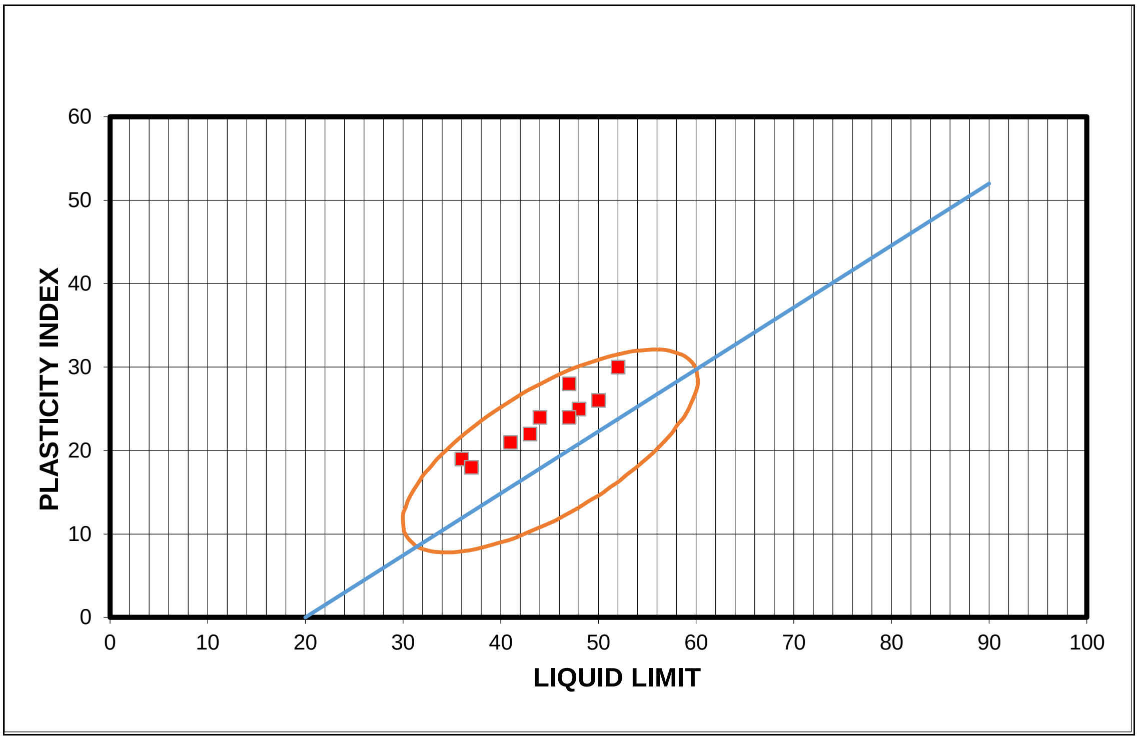


Figure 4-11: Core Borrow Pit Plasticity Chart

The estimated volume of exploitable materials from within the area investigated is of the order of 1 000 000 m³, which is far in excess of the estimated project requirements of 280 000 m³.

4.4 Fill Material

The proposed fill material (sometimes called shell material) borrow pit is located on the right flank, less than a kilometre upstream of the dam, as indicated on Figure 4-1 and 4-4. The area investigated was confined to below the full supply level within the lower valley section of the impoundment basin.

The area of potential exploitation was investigated and delineated by means of trial pits, sampling and testing.

Thirteen trial pits were excavated across the proposed borrow pit by means of a tractor-loader-backhoe (TLB). The material comprises weathered sedimentary rock of the Adelaide Formation, comprising mainly mudrock with subordinate, interlayered sandstone.

The laboratory test results are presented in Appendix E4, and are summarised in Table 4-4, together with the specifications for pervious and semi-pervious fill.

Table 4-4: Fill Borrow Pit Test Results

Property / Trial Pit	% Passing 4.75 mm	% Passing 0.425 mm	% Passing 0.002 mm	Liquid Limit %	Plasticity Index	Linear Shrinkage %	Proctor OMC %	Proctor MDD Kg/m³	Friction angle Degrees	Cohesion kPa	Permeability m/s
FTP1	54	38	11	37	18	9.5	-	-	-	-	-
FTP2*	-	-	-	-	-	-	-	-	-	-	-
FTP3	68	55	14	26	12	6	-	-	-	-	-
FTP4	38	30	6	29	11	6	-	-	-	-	-
FTP5	55	39	9	28	12	6.5	-	-	-	-	-
FT6	32	24	11	45	26	12	-	-	-	-	-
FTP7	49	41	15	37	22	10	-	-	-	-	-
FTP8*	-	-	-	-	-	-	-	-	-	-	-
FTP9	47	34	12	36	18	9.5	-	-	-	-	-
FTP10	53	46	12	32	15	7.5	-	-	-	-	-
FTP11				29	11	5.5	-	-	-	-	-
FTP12	39	22	8	37	19	10	-	-	-	-	-
FTP13	53	42	15	35	19	9.5	-	-	-	-	-
Combined	40	29	9	33	16	8.5	10.5	2030	-	-	-
Specification: Pervious	-	≥20	<20	<20	<5	<2	8-12	1700 – 2000	>35	<10	$1x10^{-5}$
Specification: Semi-pervious	60 - 100	30 – 100	<25	<25	<10	<5	10 – 15	1600 – 1850	30 – 35	10 – 15	$1x10^{-7}$

Specification according to Elges et al (1994) and Mouton (2010)
**Trial pits were dug and logged but not sampled for testing as other pits gave similar representative samples.*

The grading of the materials indicated in Table 4-4 is relatively coarse and in compliance with the grading specification for pervious shell, but the Atterberg limits (liquid limit, plasticity index and linear shrinkage) indicate a material whose fines are too plastic for both pervious and semi-pervious fill.

This will negatively affect the free draining characteristics required of a pervious and semi-pervious embankment. As previously mentioned, the investigation concentrated on the area of future impoundment, which is dominated by mudrock. In addition, due to the generally shallow excavation depths achieved using a TLB, the volume proved is 740 000 m³, which is below the project requirements for fill.

Similar material was observed in abundance further upstream and there are also other options available to achieve the required volumes of fill. Incorporating a higher proportion of sandstone, which is more prolific at higher elevations in the valley sides has the potential to produce a material with a reduced plasticity and with increased free drainage. Weathered dolerite could also be considered, which is abundant in the area. This aspect of the investigation would require further assessment should an earthfill dam option be pursued. However, the indications were that this material would not be required as an RCC dam solution is preferred.

4.5 Water

Test results on water sampled from the Tsitsa River are presented in Appendix E6 in the separate Appendices volume, and are summarised in Table 4-5, together with the specification for suitability of water for use in the manufacture of concrete (Portland Cement Institute, 1994).

Table 4-5: Laboratory Test Results on Tsitsa River Water

Property		Tsitsa River Water	Specification
Total Dissolved Solids		105 mg/l	≤ 2 000 mg/l
Chlorides (Cl^-)		16 mg/l	≤ 500 mg/l
Calcium hardness ($CaCO_3$)		71 mg/l	≤ 400 mg/l
pH		8.51	6 – 8
Sulphate (SO_3)		0 mg/l	≤ 1 000 mg/l
Strength	24 Hr	90%	≥ 90%
	3 Day	94%	≥ 90%
	7 Day	94%	≥ 90%
	28 Day	No test done	≥ 90%

According to the test results, the Tsitsa River water is suitable for use in the manufacture of concrete. If the pH of the water falls below the required specifications it will reduce the capability for the concrete to hold the aggregate together. According to the Portland Cement Institute (1994) limits for concrete mixing water up to a pH of 8.5 is acceptable. Tsitsa River water is at the upper limit of the requirements for concrete and is implying that it is alkaline and it has the potential to marginally retard concrete setting times.

4.6 Summary of Materials Availability

A summary of materials availability for RCC and earth embankment dam alternatives are as follows:

4.6.1 Rock

Good quality dolerite occurs in abundance in the project area. The problem with the site investigated as a potential rock quarry is the thickness of weathered and fractured overburden prevailing above the solid dolerite rock. The possibility of using the overburden in road layer-works construction, if confirmed by testing, has the potential to render the site feasible.

4.6.2 Sand

The sand tested from the Tsitsa River was found to be too finely graded for use as concrete fine aggregate or as filters. This is a problem facing both RCC and earth dam construction, requiring detailed evaluations, which include blending the sand with coarser materials or procuring sand from a suitable off-site source.

4.6.3 Core Material

The material tested generally complies with the specified requirements for core, although clay content and the plasticity product are above the maximum limits specified. The negative implications of this are associated more with its workability rather than suitability as impervious core. The Atterberg limits determinations for the liquid limit, plasticity index and linear shrinkage fall within the specified limits, implying that the material will not be overly active and with careful processing will be workable. It is therefore considered suitable for use as core.

4.6.4 Shell Material

The material tested comprised mudrock with subordinate intercalated sandstone. The Atterberg limits determinations of liquid limit, plasticity index and linear shrinkage fall above the maximum limits specified, which will negatively affect free drainage, as required of pervious and semi-pervious shell material. Investigating a material with a predominantly sandstone component or a weathered dolerite has the potential to produce a less plastic and more free draining material, but possibly with added environmental considerations, as such materials were found to occur at higher elevations in the valley sides.

4.6.5 Water

Tsitsa River water was found to be suitable for use in the manufacture of concrete.

5. PIPELINE GEOTECHNICAL INVESTIGATION

The proposed pipeline alignment runs from downstream of the dam on the southern side of the Tsitsa River before turning eastwards towards the tunnel inlet portal. Trial pits were excavated at 200 metre intervals along the proposed pipeline route by means of a TLB, to investigate the subsurface material characteristics and excavation conditions. The pipeline alignment and location of these trial pits are shown on Figure 5-1.

Eleven (11) trial pits were excavated to TLB refusal along the proposed pipeline alignment. Longitudinal sections illustrating the geological profile are shown in Figure 5-2 and Figure 5-3.

The transported subsurface material varied from alluvial to colluvial along the proposed pipeline route. The residual materials vary from shale, sandstone and dolerite origin. Boulders were found in most of the trial pits. Refusal of the tractor-loader-backhoe was generally experienced in all the trial pits at relatively shallow depths. The test pits profiles are attached to Appendix F1.

Sandstone, shale and dolerite bedrock was found during the pipeline investigation. Massive, hard dolerite bedrock was found at the first 5 test pits (pipeline trial pit (PTP)1 to PTP5) underlying mostly corestone filled, loose to medium dense, silty sand, residual dolerite and transported, loose, silty sand alluvial with abundant dolerite cobbles and boulders.

In trial pits PTP6 and PTP7, fine grained, very thinly bedded, very closely jointed, moderately weathered to unweathered, soft to medium hard sandstone bedrock was found. Above the sandstone bedrock, silty sand, loose, intact colluvial was found with abundant boulders and cobbles.

Trail pits PTP8 and PTP9 exhibited completely weathered to moderately weathered, fine grained, very thinly bedded and very closely jointed, medium hard rock shale. Above the shale bedrock, a loose to medium dense, intact, silty sand, colluvial with abundant boulders and cobbles was found.

In test pits PTP10 and PTP11, highly weathered to slightly weathered, fine grained, very thinly bedded, very closely jointed, medium hard sandstone bedrock was found at a relatively shallow depth underlying silty sand, loose, colluvial with abundant cobbles and boulders.

Excavation conditions along the pipeline route categorise as ‘intermediate’ and ‘hard’ according to SANS 2001-BE1: 2008 *“Classes of excavation”,* as specified for restricted excavation, within the depths investigated. Figure 5-1 and Figure 5-2 also includes a graphical illustration of the excavatability along the pipeline route.

Dolerite boulders will make excavation difficult along the proposed pipeline route. From the dam wall to the test pit location PTP6 ‘hard’ excavation can be expected from the dolerite bedrock. Blasting of the dolerite bedrock may be required to achieve the required invert level for the pipeline.

Figure 5-1: Hydropower Conduit: Pipeline Section Trial Pits

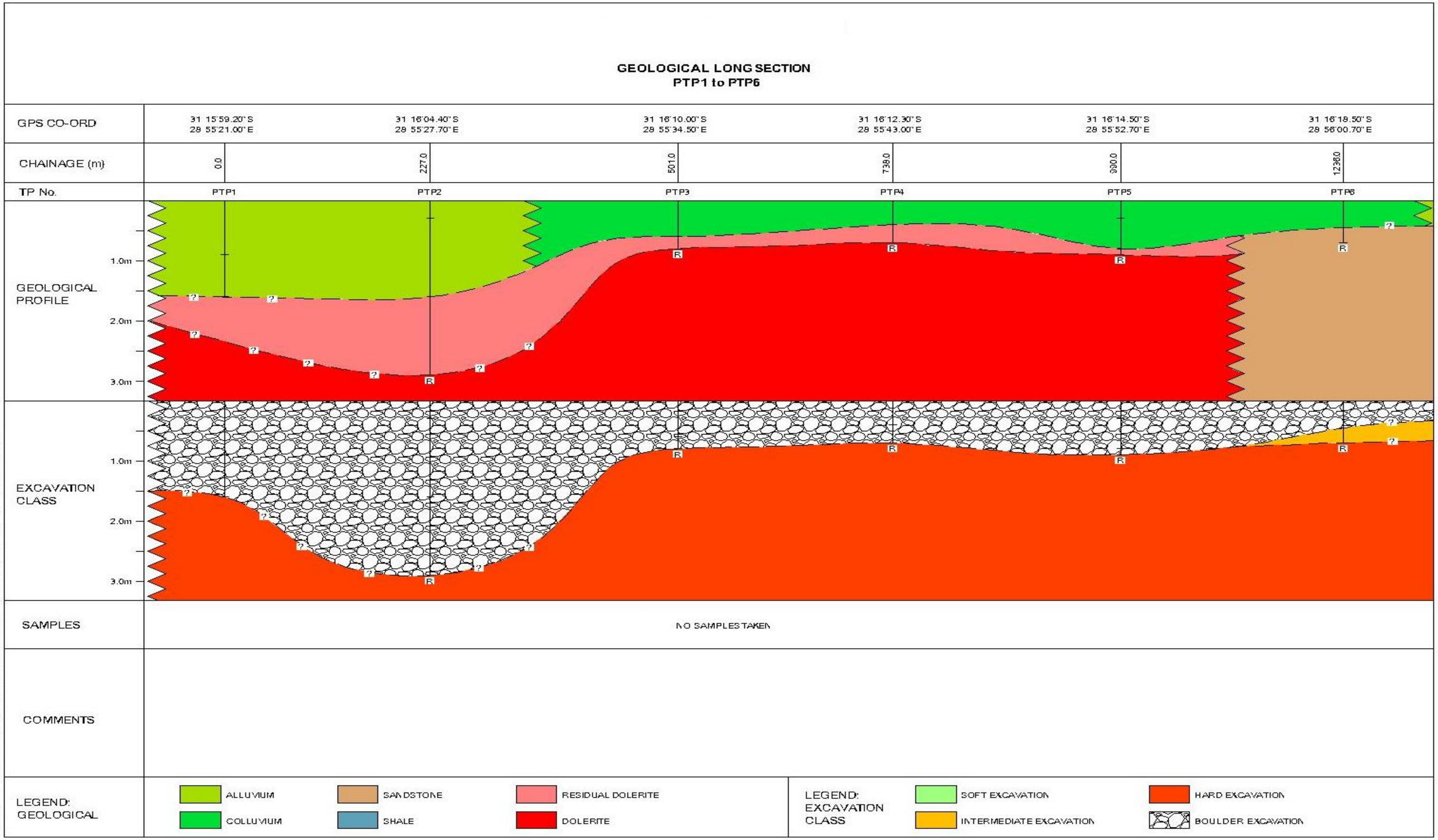


Figure 5-2: Pipeline Geological Longitudinal Section between PTP1 to PTP6

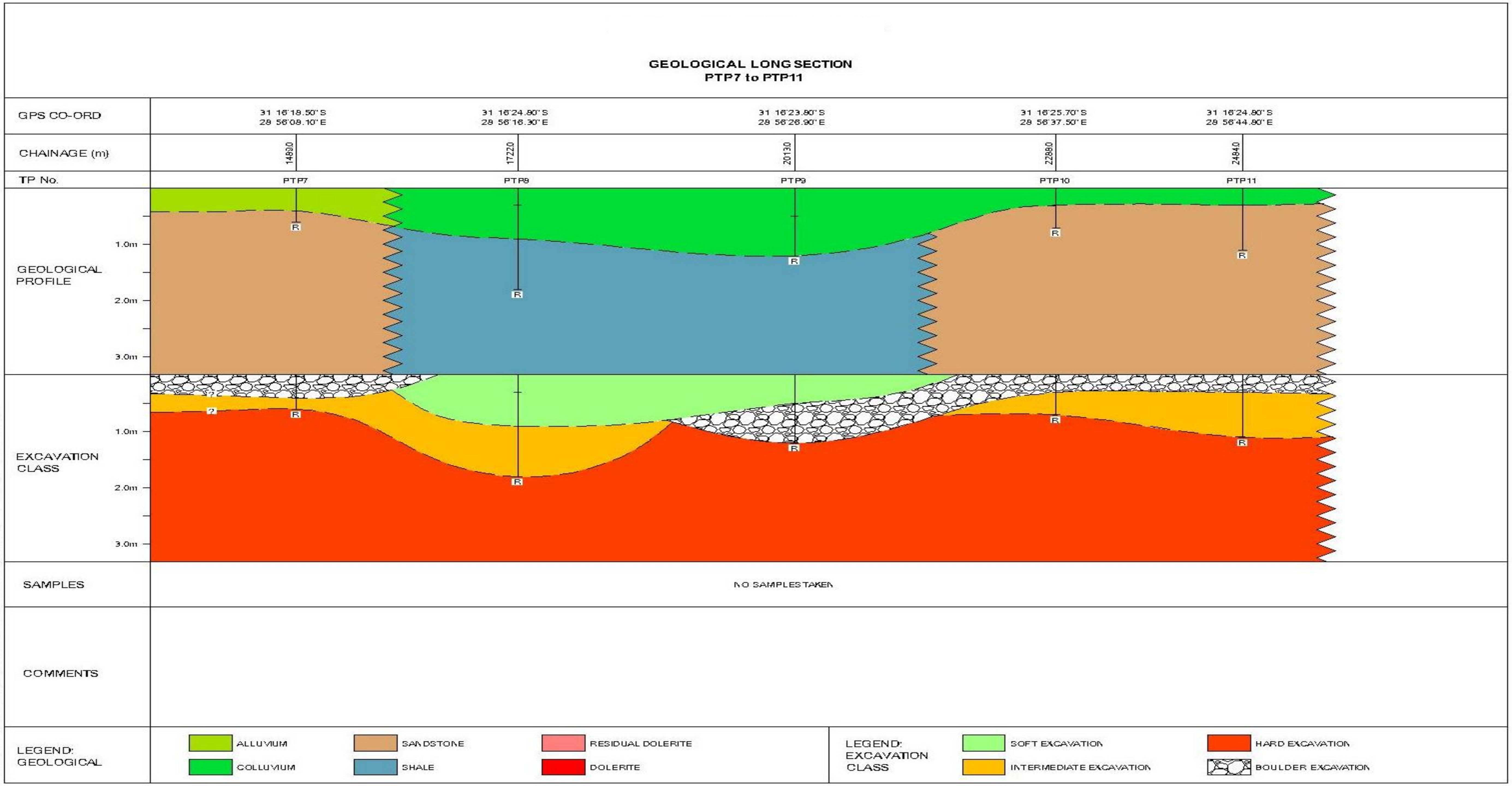


Figure 5-3: Pipeline Geological Longitudinal Section between PTP7 to PTP11

The excavatability of shale bedrock and sandstone bedrock will vary from 'intermediate' to hard excavation between trial pit locations PTP6 and PTP11.

Table 5-1: SANS 2001-BE1: 2008 Excavation Class Descriptions

Excavation Class	Description
Soft	Excavation in material that can be efficiently removed by a back-acting excavator of flywheel power approximately 0.10 kW per millimetre of tined-bucket width, without the use of pneumatic tools such as paving breakers
Intermediate	Excavation in material that requires a back-acting excavator of flywheel power exceeding 0.10 kW per millimetre of tined-bucket width or the use of pneumatic tools before removal by equipment equivalent to that specified for soft excavation.
Hard	Excavation in material that cannot, before removal, be efficiently ripped by a bulldozer of mass approximately 35 ton, fitted with a single-tine ripper suitable for heavy ripping and of flywheel power approximately 220 kW.

6. TUNNEL GEOTECHNICAL INVESTIGATION

A number of alternative hydropower conduit options and alignments were considered. These are illustrated on Figures 6-1 to 6-3 and included:

- a long tunnel option with the inlet portal constructed into the right flank behind the dam and with two vertical alignments, one deep and a shallower one that would require steel segments in an area of shallow overburden cover,
- a part pipeline / part tunnel option. At the time of commencing the investigation this was the favoured option, comprising a 3.5 km long pipeline from the dam to the inlet portal, a 4 km tunnel and a pipeline from the outlet portal to the generating substation. This option had nominal overburden thickness, requiring steel segments over only short sections at the portals, and
- subsequently, a third option has been proposed, similar to the second option, but comprising a deeper tunnel, which would eliminate the second pipeline section to the generating substation.

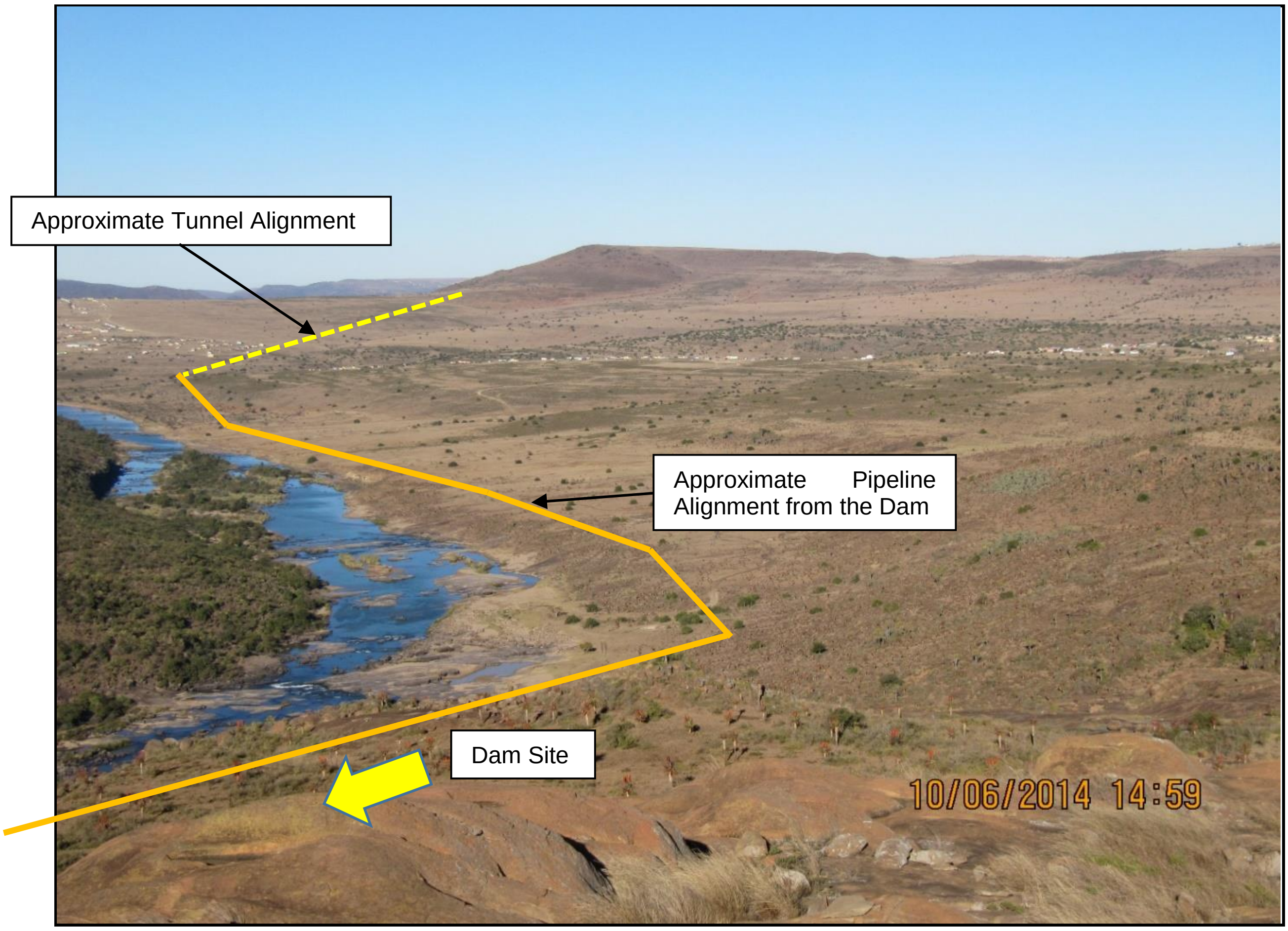


Figure 6-1: Conduit Route Visualisation from above the Dam Right Flank

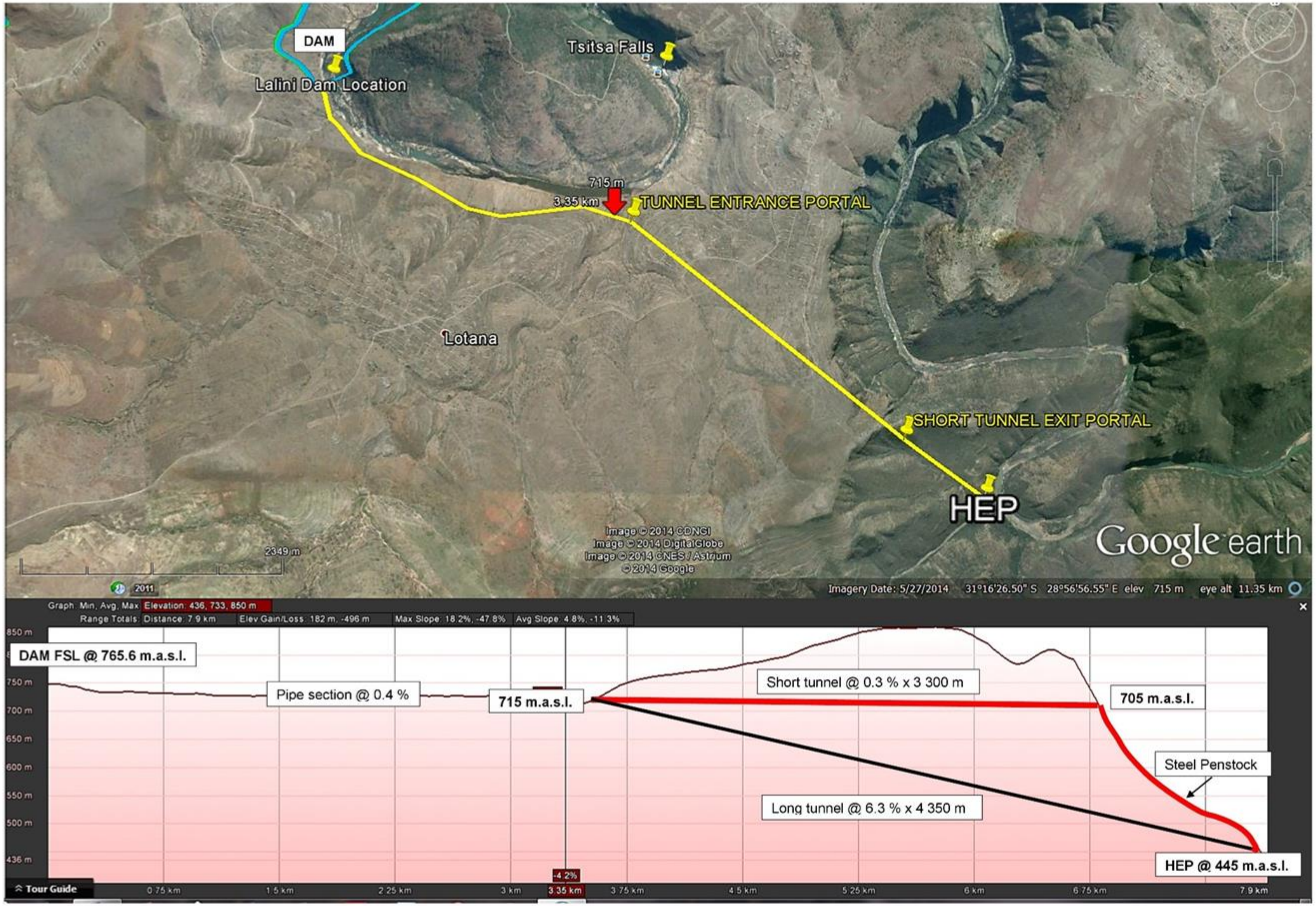


Figure 6-2: Plan and Long Section of Conceptual Pipeline and Tunnel Alignments

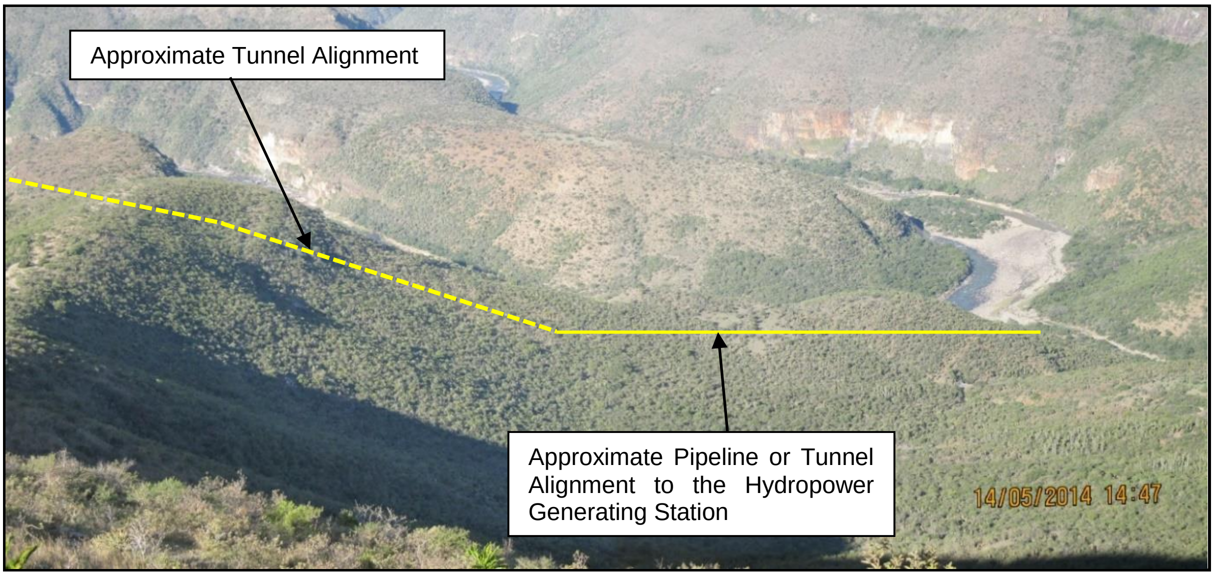


Figure 6-3: View of Tunnel and Pipeline Alignment to Hydropower Station

Within the allowable budget constraints, the tunnel geotechnical investigation entailed the drilling of seven rotary core boreholes, as indicated on Figure 6-4.

Whilst the drilling concentrated on the favoured tunnel option, it aimed at providing an overall evaluation of the subsurface conditions for possible alternative alignments and hence Borehole T1 and T3, as indicated on Figure 6-4, were also drilled off the favoured alignment, which gave an indication of underground conditions laterally from the alignment.

Borehole T7 was drilled, where access permitted, to gain an assessment of geotechnical conditions downstream of the outlet portal for the second pipeline and hydropower generating substation.

Figure 6-6 is a longitudinal section along the tunnel route showing the two alternative vertical alignments and the borehole profiles. This indicates that the upper tunnel alignment for all or most of its length is likely to be excavated through sedimentary sandstone / siltstone geology. The lower tunnel alignment was proposed in order to avoid the construction of the second pipeline over the steep and rough terrain that characterises the area downstream of the outlet portal of the upper alignment.

The borehole details along the horizontal alignment, geology within the tunnel zone of the upper alignment and expected geology within the tunnel zone of the lower alignment are indicated in Table 6-1.

The measured discontinuity orientations are shown plotted relative to the tunnel axis on Figure 6-5, and summarized on Table 6-2.

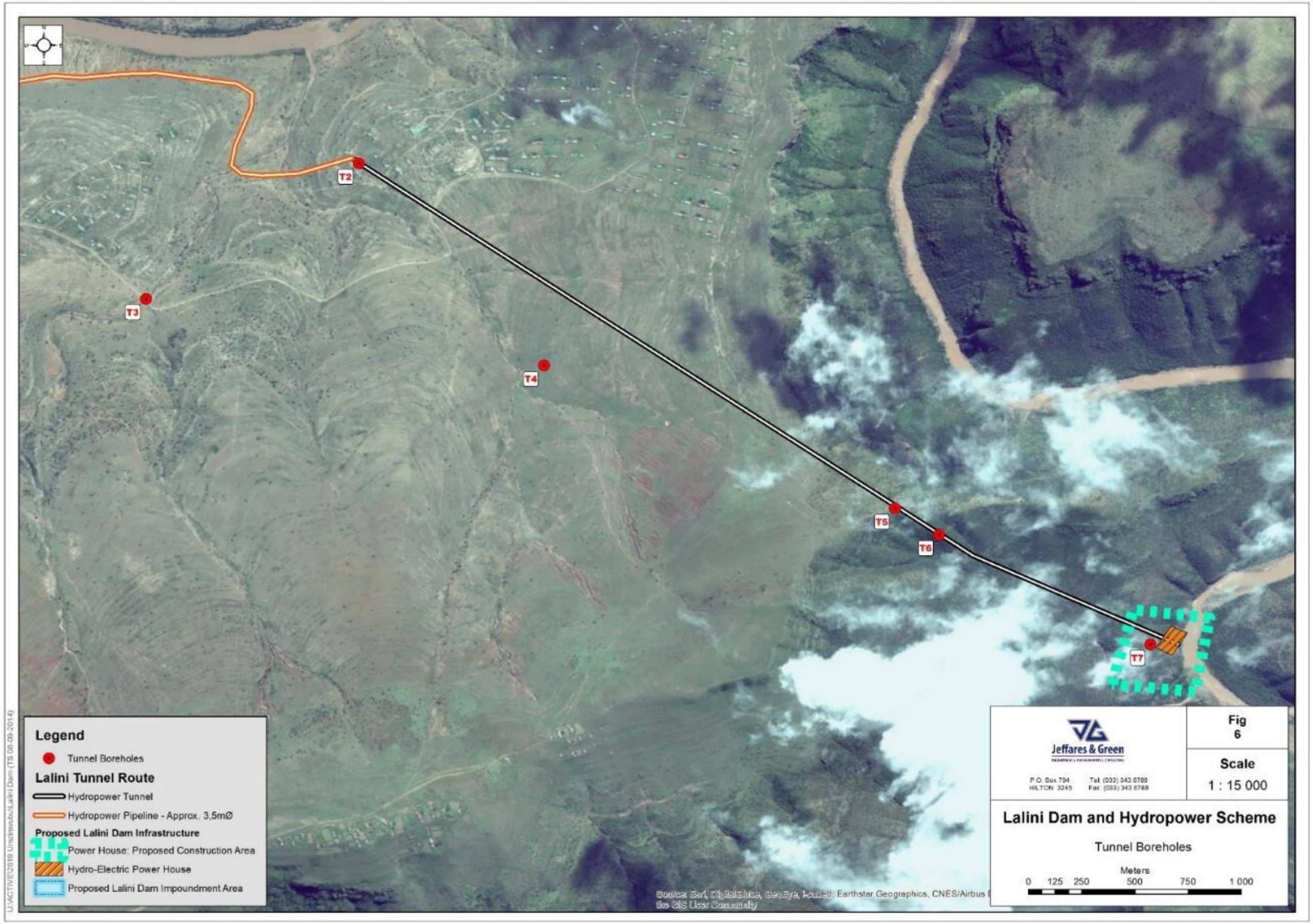


Figure 6-4: Borehole Locations for the Tunnel

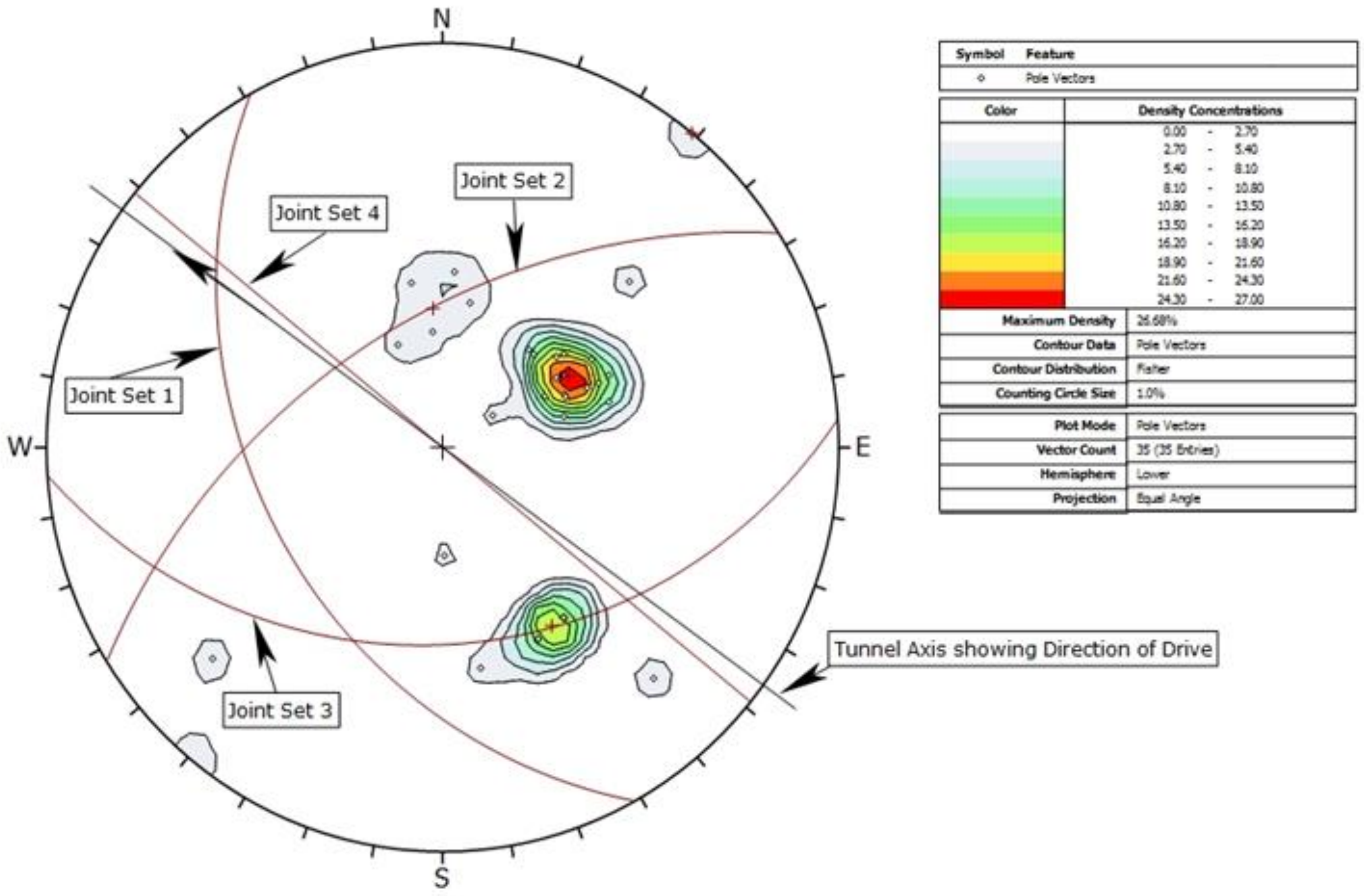


Figure 6-5: Stereographic Projection of Tunnel Discontinuity Plane Orientations

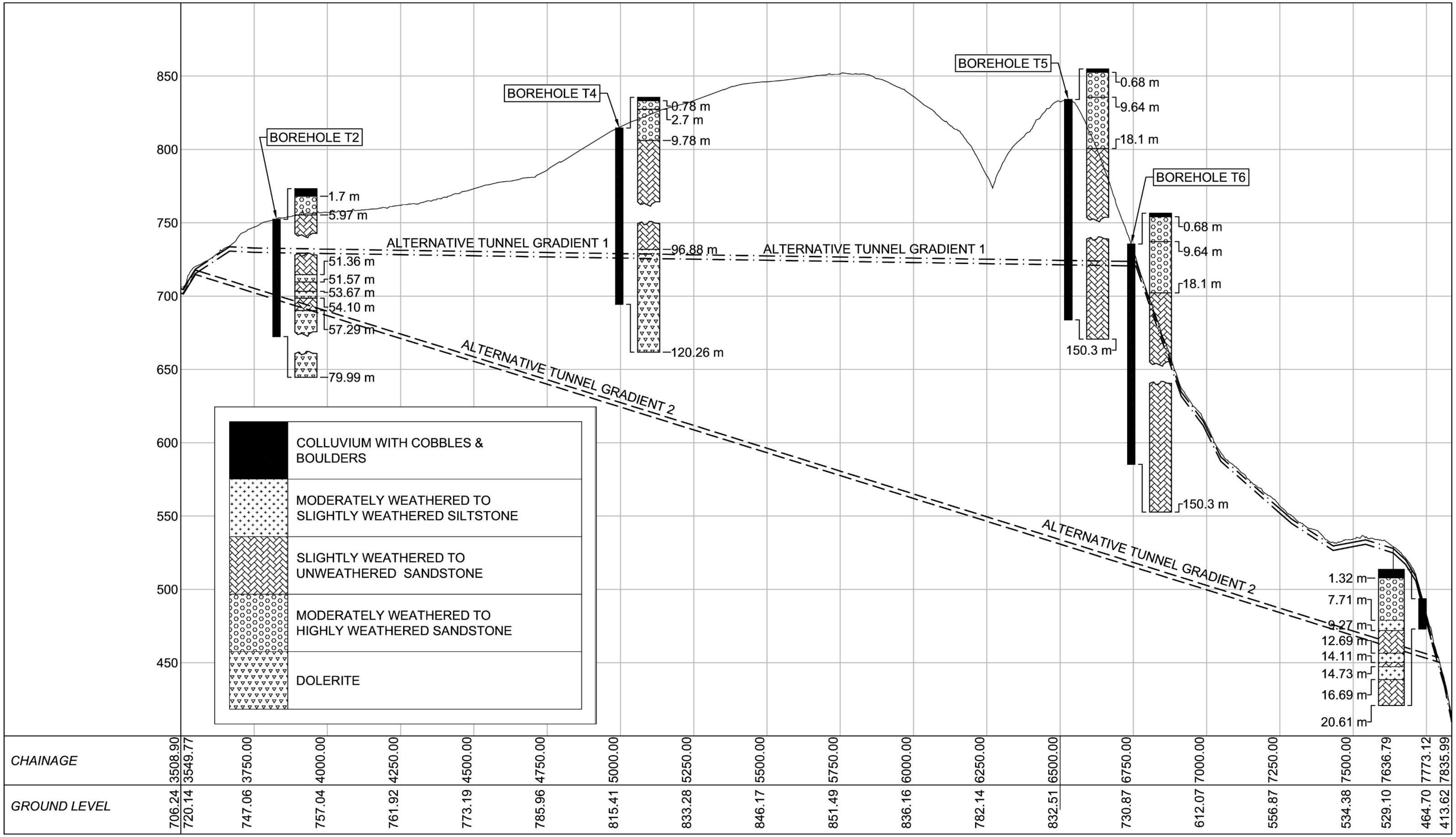


Figure 6-6: Borehole Log Profiles along Tunnel Section

Table 6-1: Tunnel Drilling Results

Borehole No.	Drilling Length (m)	Upper Alignment Tunnel Zone Geology	Lower Alignment Expected Tunnel Zone Geology (Assumed)
T2	79.99	Sedimentary rock comprising laminated / interbedded sandstone / siltstone. RMR = 72	Sedimentary rock. Close to dolerite contact.
T4	120.26	Sedimentary rock comprising laminated / interbedded sandstone / siltstone. RMR = 67	Difficult to evaluate. Either sedimentary rock or dolerite
T5	150.30	Sedimentary rock comprising laminated / interbedded sandstone / siltstone. RMR = 70	Sedimentary rock. The surface geology does not suggest the presence of intrusive dolerite sills along this portion of the tunnel route.
T6	70.03	Sedimentary rock comprising laminated / interbedded sandstone / siltstone. RMR = 40 – 55. Upper part of borehole. Near tunnel outlet portal.	
T7	20.61	N/A Not in the tunnel	

Table 6-2: Tunnel Discontinuity Plane Orientations on Figure 6-5

Joint Set No.	Dip (degrees)	Dip Direction (degrees)
1	40	241
2	55	328
3	38	176
4	90	219
Tunnel Axis and Direction of Drive	-	306

The rock mass rating (RMR) and geological strength index (GSI) of the rock occurring within the tunnel zone (upper alignment) is indicated in Table 6-3.

Table 6-3: RMR and GSI for the Tunnel

Borehole No.	DEPTH (m)	GSI	RMR
T2	10.0 – 25.0	70	72
T4	74.0 - 95.0	67	70
T5	100.0 - 120.0	70	70
T6	8.0 - 12.0	50	40
T6	12.0 - 21.0	50	55

Along the tunnel alignment water was sampled from Boreholes T4, T5 and T6 and was found to be mildly aggressive to non-aggressive towards buried concrete and steel structures.

7. SAMPLING AND TESTING

7.1 Rock Strength Testing

The results of Point Load Strength Index (PLSI) and Unconfined Compressive Strength (UCS) testing carried out on rock core samples from the dam, quarry and tunnel boreholes are summarised in Table 7-2, based upon the classification given in Table 7-1.

Rock hardness, according to the Core Logging Committee (1976), is clarified as follows:

Table 7-1: Rock Hardness Classification Based on UCS and PLSI

UCS Range	PLSI Range	Rock Hardness Classification
1 – 3 MPa	< 1 MPa	Very Soft Rock
3 – 10 MPa	1 – 2 MPa	Soft Rock
10 – 25 MPa	2 – 4 MPa	Medium Hard Rock
25 – 70 MPa	4 – 10 MPa	Hard Rock
70 – 200 MPa	> 10 MPa	Very Hard Rock
>200 MPa		Extremely Hard Rock

Table 7-2: Results of Rock Strength Testing

Results of Point Load Strength Index and Unconfined Compressive Strength Tests						
B/Hole No.	B/Hole Length (m)	Vertical Depth (m)	Rock Type	Point Load Strength Index		Unconfined Compressive Strength (MPa)
				$I_{s(50)}$ (MPa)	Equivalent UCS (MPa)*	
D1	8.12	8.12	Sandstone	1.3 (diametral)	30.0	-
	9.46 – 9.5	9.46 – 9.5	Sandstone	1.5 (axial)	36.3	-
	9.5	9.5	Sandstone	1.8 (diametral)	42.8	-
D2	4.46 – 4.51	4.46 – 4.51	Dolerite	8.7 (axial)	209.4	-
	4.51	4.51	Dolerite	8.6 (diametral)	205.4	-
	4.51 – 4.76	4.51 – 4.76	Dolerite	-	-	192.0
	5.78	5.78	Dolerite	7.5 (diametral)	181.1	-
	5.88	5.88	Dolerite	9.6 (diametral)	231.1	-
D3	3.75 – 3.8	3.75 – 3.8	Sandstone	5.4 (axial)	130.7	-
	3.8 – 3.98	3.8 – 3.98	Sandstone	-	-	110.9
	10.29	10.29	Dolerite	13.9 (diametral)	333.8	-
	10.47	10.47	Dolerite	12.5 (diametral)	299.6	-
D4	6.47	6.47	Dolerite	5.3 (diametral)	128.4	-
	6.67 – 6.8	6.67 – 6.8	Dolerite	-	-	145.2
	7.23	7.23	Dolerite	7.1 (diametral)	171.2	-
	15.77	15.77	Dolerite	8.9 (diametral)	214.0	-
Q2	5.89 – 6.09	5.89 – 6.09	Dolerite	-	-	189.0
Q3	4.49 – 4.63	4.49 – 4.63	Dolerite	-	-	170.6
T2	50.48	50.3	Sandstone	6.3(diametral)	151.2	-
	50.77 – 50.93	50.6 – 50.7	Sandstone	-	-	131.2
	51.16	51.0	Sandstone	8.4 (diametral)	201.6	-
	51.16 – 51.22	50.9 – 51.1	Sandstone	5 (axial)	120	-
	57.32	57.1	Dolerite	16.9 (diametral)	405.6	--
	57.32 – 57.58	57.1 – 57.3	Dolerite	-	-	306.6
	74.05 – 74.22	73.8 – 73.9	Dolerite	-	-	146.2
T3	75.76	75.5	Dolerite	12.8 (diametral)	307.2	-
	75.76 – 75.82	75.47 – 75.53	Dolerite	7 (axial)	168 edge break	-

Results of Point Load Strength Index and Unconfined Compressive Strength Tests

B/Hole No.	B/Hole Length (m)	Vertical Depth (m)	Rock Type	Point Load Strength Index		Unconfined Compressive Strength (MPa)
				$I_{s(50)}$ (MPa)	Equivalent UCS (MPa)*	
T4	95.9	95.5	Sandstone	8.2 (diametral)	196.8	-
	96.0	95.6	Sandstone	4.3 (diametral)	103.2	-
	96.13 – 96.27	95.7 – 95.9	Sandstone	-	-	106.2
	97.1 – 97.23	96.7 – 96.8	Dolerite	-	-	115.1
	101.11	100.7	Dolerite	8.2 (diametral)	196.8	-
	101.39	101.0	Dolerite	9.7(diametral)	232.8	-
	107.04 – 107.09	106.6 – 106.7	Dolerite	10 (axial)	240	-
	107.09	106.7	Dolerite	9.5 (diametral)	228	-
	107.09 – 107.29	106.7 – 106.9	Dolerite	-	-	90.1
	111.48 – 111.54	111 – 111.1	Dolerite	8 (axial)	192	-
	115.7 – 115.76	115.2 – 115.3	Dolerite	7.3 (axial)	175.2	-
	115.76	115.3	Dolerite	10.8 (diametral)	259.2	-
	117.48	117.0	Dolerite	14.5 (diametral)	348	-
	118.49 – 118.53	118.0 – 118.1	Dolerite	10.4 (axial)	249.6	-
	118.53	118.1	Dolerite	11.2 (diametral)	268.8	-
T5	4.79 – 4.82	4.77 – 4.8	Sandstone	3.7 (axial)	88.8	-
	6.91	6.9	Sandstone	3 (diametral	72	-
	10.48	10.4	Sandstone	0.4 (diametral)	9.6	-
	10.55	10.5	Sandstone	0.6 (diametral)	14.4	-
	10.87	10.8	Sandstone	3.7 (diametral)	88.8	-
	29.33	29.2	Sandstone	3 (diametral)	72	-
	29.33 – 29.37	29.2 – 29.3	Sandstone	5.1 (axial)	122.4	-
	29.37 – 29.7	29.3 – 29.5	Sandstone	-	-	121.7
	29.73	29.6	Sandstone	0.2 (diametral)	4.8	-
	44.15 – 44.18	43.97 – 44.0	Sandstone	5.1 (axial)	122.4	-
	44.18	44.0	Sandstone	5 (diametral)	120	-
	71.58 – 71.61	71.29– 71.3	Sandstone	6.7 (axial)	160.8	-
	71.61	71.3	Sandstone	2.4 (diametral)	57.6	-
	101.14	100.7	Sandstone	4.3 (diametral)	103.2	-
	101.17 – 101.41	100.8 – 101.0	Sandstone	-	-	130.1
	113.22	112.77	Sandstone	6.7 (diametral)	160.8	-
	113.22 – 113.25	112.7 – 112.8	Sandstone	7.8 (axial)	187.2	-
	120.95	120.5	Sandstone	4.3 (diametral)	103.2	-
	120.99 – 121.31	120.5 – 120.8	Sandstone	-	-	182.5
	134.41 – 134.45	133.8 – 133.9	Sandstone	6.9 (axial)	165.6	-
	134.45	133.9	Sandstone	3.9 (diametral)	93.6	-
	146.59	146.0	Sandstone	5.8 (diametral)	139.2	-

**Equivalent UCS = 24.$I_{s(50)}$ (Broch and Franklin, 1972).*

Diametral tests refer to tests conducted perpendicular to the long axis of the core.

Axial tests refer to tests conducted parallel to the long axis of the core.

7.2 Water Pressure Tests

The results of the water pressure tests have been interpreted according to the method of Houlsby (1976) and are presented in Table 7-3. A lugeon is defined as the loss of water in litres per minute and per metre borehole at an over pressure of 1 MPa.

Table 7-3: Water Pressure Test Results

BH No.	Stage Depth (m)	Permeability			Comments
		Lugeon per Test Interval	Representative Lugeon Value from Tests	Flow Group	
D1	10.0 – 16.3	9/7/5/8/7	7	Void filling	
	16.3 – 21.5	5/5/7/9/9	8	Laminar / turbulent	3rd stage leakage
	21.5 – 25.4	7/5/4/5/6	4	Turbulent	
D2	7 – 12	1/1/3/12/23	23	Wash out	
	12 – 18.4	1.3/2/2/2	2	Laminar	2nd stage leakage
	18.5 – 25.8	11/?	No result	-	Leakage & pressure loss
D3	3.5 – 7.2	56/24/17/21/30	17	Turbulent	
	7.5 – 13.5	13/7/5/5/1	1	Void filling	
	13.5 – 24.2	0/2/1/2/0	1	Laminar / turbulent	
D4	15 – 20.5	20/13/7/0/20	13	Not defined	3rd & 4th stage leakage and pressure loss
	20.5 – 24.8	30/?	No result	-	
T5	17 – 150.3	1/0/0/0/1	1	Laminar	
	23 – 150.3	0/0/0/0/0	0	-	3rd stage pressure loss
	37.5 – 150.3	0/0/0/?/0	No result	-	2nd & 3rd stage leakage & pressure loss
	Double packer stuck at 50 m and 110 m – fracture zones. Tested with single packer to bottom of borehole				

7.3 Water Chemistry

The results of chemical tests on water sampled from the Tsitsa River and Boreholes T4, T5 and T6 are summarised in Table 7-4.

Table 7-4: Results of Chemical Tests on Water Samples

Results of Chemical Tests on Water Samples from Tsitsa River and Boreholes					
Property	**Unit**	**Tsitsa River**	**BH T4**	**BHT5**	**BHT6**
pH	pH	8.51	8.24	7.55	7.64
$CaCO_3$ Saturated pH	pH	-	8, 11	7.3	7.45
Calcium Hardness ($CaCO_3$)	mg/l	71	79	146	144
Total Ammonium (NH_4)	mg/l	-	< 1	< 1	< 1
Magnesium (Mg)	mg/l	-	25	35	33
Sulphate (SO_4)	mg/l	-	21	17	20
Sulphate (SO_3)	mg/l	Nil	-	-	-
Chloride (Cl^-)	mg/l	-	101	169	176
Chloride (Cl^-)	mg/l	16	-	-	-
Total Dissolved Solids	mg/l	105	607	876	767

7.4 Rock Mineralogy

The modal percentages of the quartz, alkali feldspar and plagioclase feldspar constituents of the dolerite, as obtained by petrographic analysis and XRD, were normalised and plotted on the IUGS (International Union of Geological Sciences) QAP diagram, as indicated by the modal proportions of quartz, alkali feldspar and plagioclase on Figure 7-1. The rocks generally classify as quartz bearing micro-gabbro, which is synonymous with a dolerite composition. The star symbols represents the test results.

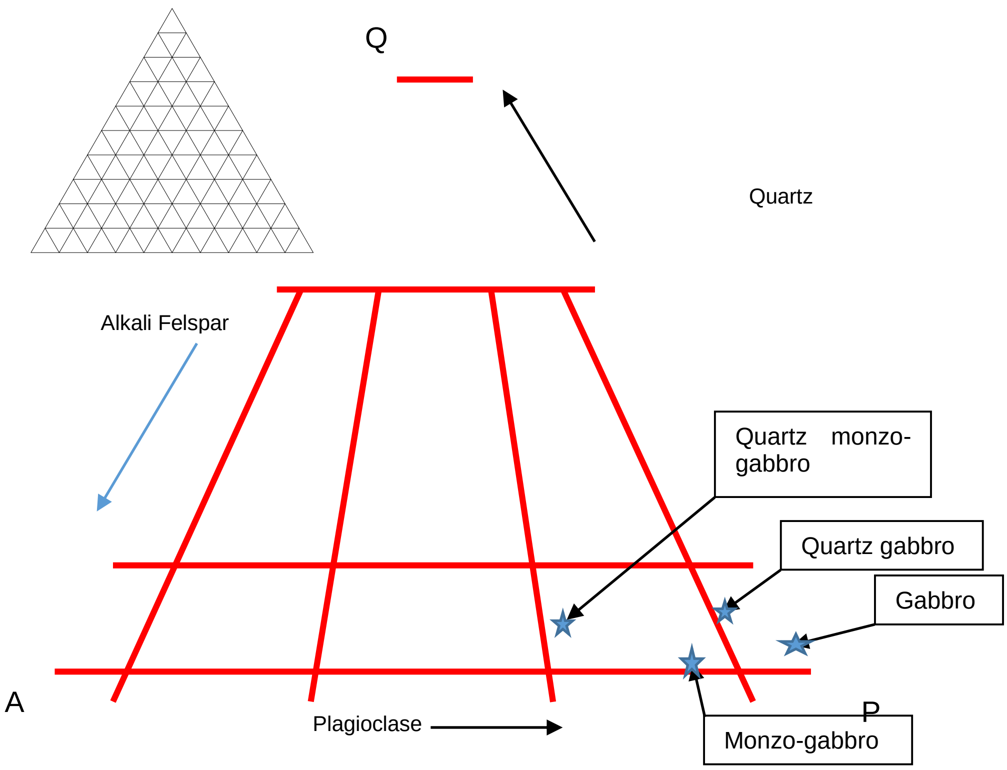


Figure 7-1: Dolerite Plots on QAP Diagram

A summary of the mineral compositions is presented in Table 7-5.

Table 7-5: Mineral Compositions of Rock Samples

Sample	Major Mineral Constituents	Minor Mineral Constituents	Accessory Minerals	Rock Name	Comments
Dolerite					
Borehole Q2 6.24-6.36 m	Plagioclase 56% Pyroxene 31%	Quartz 5% Muscovite 6% Hornblende 2%	Opaque minerals	Quartz bearing Dolerite	Crystalline rock of dolerite composition; Low degree of alteration; No deleterious smectite clays present (see footnote 2 on Page 23)
Borehole T2 57.5-57.58 m	Plagioclase 49% Pyroxene 31%	Quartz 7% Orthoclase 5% Hornblende 5% Muscovite 3%	Opaque minerals		
Borehole T4 97.23-97.31 m	Plagioclase 33% Pyroxene 31%	Orthoclase 12% Quartz 6% Amphibole 4% Prehnite 13% Epidote 1%	Opaque minerals		
Borehole T4 115.97-116.02 m	Plagioclase 51% Pyroxene 25%	Microcline 9% Quartz 4% Muscovite 5% Hornblende 4%	Opaque minerals (ilmenite) 2%		
Sandstone					
Borehole D3 3.61-3.75 m	Quartz 48% Plagioclase 22%	Microcline 8% Muscovite 7% Chlorite 8% Laumontite 7%		Quartzo-feldspathic sandstone	
Borehole T2 50.63-50.77 m	Quartz 42% Plagioclase 19% Muscovite 17%	Microcline 9% Chlorite 9% Laumontite 4%		Micaceous sandstone / siltstone	
Borehole T4 96.04-96.13 m	Quartz 36% Plagioclase 19% Muscovite 19%	Microcline 9% Chlorite 11% Laumontite 5.5%		Micaceous sandstone / siltstone	
Borehole T5 112.77-113.22 m	Quartz 38% Plagioclase 40%	Microcline 9% Muscovite 7% Chlorite 6%		Quartzo-feldspathic sandstone	

8. GEOTECHNICAL APPRAISAL OF DAM

8.1 Dam Foundation

The recommended dam foundation excavation profile depicted on Figure 3-1 varies from 8.08 m at Borehole D1 on the upper left flank to virtually zero in the river section to 6.4 m at Borehole D4 on the upper right flank. This places the foundation in a rock hardness category of medium hard to hard rock and is generally commensurate with the excavation limits of large mechanical excavators and bulldozers. The profile depicted on Figure 3-1 is based upon the drilling of only four boreholes.

The literature emphasises the avoidance or minimisation of blasting at or near to the foundation invert level, due to the adverse effects of blast fracturing. Van Schalkwyk *et al* (2009) recommend that bulk blasting be terminated at least 1 m above invert level, with excavation below this by means of powerful excavation equipment or highly controlled blasting. Recommendations for a RCC dam foundation, according to van Den Berg and Parrock (2009), are summarised in Table 8-1.

Table 8-1: Foundation Design Criteria for RCC Dam

E_{mod} (GPa)	RMR	Weathering	UCS (MPa)	RQD %	Joint Spacing	Joint Condition
>4.5	>40	Medium to slightly weathered or better	>20	>30	>300 mm	Rough, unaltered

As the foundation is the key element in the integrity of the dam, it is essential that the inferences made in this investigation are verified during detailed design by the undertaking of more detailed investigations, including additional drilling, water pressure testing and rock strength testing. It is also important for the foundation invert to be visually appraised and approved by a geotechnical professional during construction.

8.2 Grouting

The water pressure tests results summarized in Table 7-3 indicate that the dam foundation will require grouting as lugeon values are in excess of 3 for an RCC dam according to Houlsby (1983).

It is recommended that grouting comprise a single row grout curtain with primary grout holes at 6 m intervals to a minimum length of 20 m. Due to the high angle joints in the dolerite it is recommended that the grout holes are inclined at 70° into the flanks and stitched across the river section to provide a transitional zone for the change of direction.

At this feasibility stage it is recommended that secondary grout holes are installed to depths of 20 m or a lesser depth as determined by additional drilling and water pressure testing. Secondary grout holes to be equally spaced, midway between the primary holes.

The need for tertiary grouting to achieve satisfactory closure of the grout curtain requires to be more conclusively proved by further investigation or confirmed on site during construction by the undertaking of water pressure tests. The sequence of grouting must be such that no subsequent phase is commenced prior to the completion of the previous phase, namely that all primary grouting must have been completed prior to the commencement of secondary grouting.

Due to the presence of fracture zones normally associated with the contact areas between dolerite and sandstone, it is highly likely that consolidation grouting will be required, but confined to the lower flanks.

This could take the form of additional grout lines on both the upstream and downstream sides of the grout curtain and extending to a depth of about 6 m below foundation invert. This would have a two-fold effect of reducing permeability and improving rock quality in the upper part of the foundation on the lower flanks. This needs to be verified by further drilling during detailed design to identify and quantify zones of weakness in the foundation and optimise the extent of consolidation grouting.

The recommendations made regarding foundation treatment are based upon very limited information acquired from the drilling of only four boreholes. As a result, the recommendations made cannot be applied rigidly at this stage, until verified during the detailed design stage investigations, as actual conditions may vary from those assumed. The detailed design geotechnical investigations should be considered as the “minimum standard” for assumptions to be made.

Even following this level of detail in respect of the investigations, appropriate foundation treatment may still need to be developed as necessary to cater for the actual conditions encountered during construction.

9. GEOTECHNICAL APPRAISAL OF TUNNEL

Applying the Finite Element Analysis (FEA) method, the internal stability of the tunnel was assessed according to the Generalized Hoek-Brown material model. This was used to evaluate the behaviour of the in-situ rock mass surrounding the tunnel structure. The critical material parameters required by the material model were selected based upon the borehole data retrieved from Boreholes T2, T4, T5 and T6.

From the borehole data the classification of the surrounding rock mass was carried out according to Bieniawski's Rock Mass Rating system (1973).

The Geological Strength Index (GSI) of the surrounding rock mass was also determined by using the software program RocLab based on Bieniawski's Rock Mass Rating system (1973).

The most critical scenario exists where sandstone with interbedded and interlaminated siltstone occurs around the tunnel structure. This is of particular importance also at the portal areas where the rock is exposed and is more fractured and weathered. A finite element method was used to model the joint network located in the sandstone / siltstone profile.

The results of this provided evidence that the combined joint network in the surrounding rock mass could undergo considerable yielding. This indicated that the tunnel would be unstable at such locations if no supporting elements are used.

Installing a composite liner consisting of shotcrete, mesh and selective rock-bolting where necessary resolved the instability issues.

This issue must be revisited at the detailed design stage and once more geotechnical investigations have been undertaken. Allowance should also be made in the works contract contingencies for unforeseen additional lining as this need could also be discovered during the drill and blast construction process.

10. GEOTECHNICAL APPRAISAL OF THE PIPELINE

Eleven (11) trial pits were excavated to TLB refusal along the proposed pipeline alignment. Shallow bedrock was found in all eleven (11) trial pits, ranging in hardness of soft to very hard rock.

Dolerite boulders were also visible in trial pits close to the Tsitsa River. Excavation conditions along the pipeline route categorise as ‘intermediate’ and ‘hard’ according to SANS 2001-BE1: 2008 *“Classes of excavation”,* as specified for restricted excavation, within the depths investigated.

Blasting of the dolerite bedrock may be required to achieve the required invert level for the pipeline.

Excavation in sandstone and shale material will require a back-acting excavator of flywheel power exceeding 0.10 kW per mm of tined-bucket width or the use of pneumatic tools before removal by equipment equivalent to that specified for soft excavation.

Large diameter steel pipelines require close attention to be made to bedding and backfill specification and design. It is expected that a high proportion (greater than 50%) of the material excavated from the pipeline trench will not be suitable as bedding or backfill, and granular/free draining material of suitable specification would need to be imported and the unsuitable excavated material disposed of to spoil.

The cost of this has been allowed for in the capital cost estimates, and further bedding and backfill materials sourcing investigations will be required at detailed design stage.

11. CONCLUSION

This report presents the results of geotechnical investigations undertaken for the feasibility study of the Lalini Dam and Hydropower Scheme. The project is located on the lower Tsitsa River near the village of Lalini. The proposed dam is located 3.5 km upstream of the Tsitsa Falls and the hydroelectric plant (HEP) is located some 15 km downstream in the river valley below the Tsitsa Falls.

The feasibility level geotechnical investigations involved selective rotary core drilling of the proposed dam centre-line, tunnel alignment and a potential rock quarry site. Trial pitting was undertaken along the pipeline route between the dam and the tunnel inlet portal, as well as in the investigation of core and shell borrow pits.

The investigation of potential construction material sources generally indicated that an earthfill dam option is feasible. The major constraint in respect of materials, applicable to both dam types, is the lack of suitable naturally occurring sand within an economic haulage distance. The sand occurring in the Tsitsa River course was found to be too fine grained for use as either a concrete aggregate or a filter medium.

Whilst the investigation considered two dam types, namely embankment fill types or an RCC dam, it was concluded in associated reports that the RCC dam is preferred.

The drilling investigation along the proposed dam centre-line indicated founding levels to vary from between about 6 m and 8 m on the upper flanks to between about 3 m and 4 m on the lower flanks. Dolerite outcrops across the river section.

A cut-off grout curtain is considered necessary and selective consolidation grouting on the lower flanks may be required. Additional drilling during the detailed design phase will be required for infill purposes and to further assess conditions at transitions, such as at contacts between dolerite and sandstone and to verify grouting requirements. Infill drilling will be required on the embankment footprint and at the positions of appurtenant structures.

The area investigated as a potential rock quarry, although ideally located, was found to have an excessively thick overburden cover. The feasibility of the site as a rock quarry depends upon the usability of the overburden materials in roads construction and it is recommended that this assessment forms part of the detailed design geotechnical investigations.

The roads construction programme must be properly timed to be able to use the material before basin impoundment to avoid double handling.

The investigation for the tunnel considered a number of alternatives in terms of the horizontal and vertical alignments, but concentrated on an alignment favoured at the time comprising part tunnel / part pipeline. This consisted of an approximately 3.5 km long pipeline from the dam to the tunnel inlet portal, an approximately 4.4 km long tunnel with a short pipe conduit from the tunnel outlet portal to the hydropower generating substation.

Based upon the proposed vertical alignment the tunnel would be excavated in a competent sandstone, through a laminated / interbedded, sandstone / siltstone sedimentary sequence, and with a significant proportion in competent dolerite.

Whilst the budgetary constraints did not allow very deep boreholes to be drilled through and below the deepest central section of the deeper alignment, indications were that the central sections of the deeper tunnel alignment would be within similar competent dolerite sill structures encountered elsewhere in the geotechnical investigations, and that each end of the tunnel would again be excavated in the laminated / interbedded, sandstone / siltstone sedimentary sequence.

The results thus indicate that the tunnelling conditions in both sandstone and dolerite should not be problematical and that the tunnel should only require rock bolting and shotcrete lining arch support where necessary.

Please see the Feasibility Design: Lalini dam and Hydropower Scheme Report No. P WMA 12/T30/00/5212/19.

Given that this drilling investigation was undertaken only at a feasibility study level, it is considered essential that further holes are drilled along the tunnel route to fully inform the optimisation of the tunnel at the detailed design stage.

12. REFERENCES

1. Bieniawski, Z T (1973), Engineering Classification of Jointed Rock Masses, Transactions of the South African Institution of Civil Engineers, Vol. 15, No. 12.

2. COLTO (1998), Standard Specifications for Road and Bridge Works for State Road Authorities.

3. Core Logging Committee of the South Africa Section of the Association of Engineering Geologists (1976), A Guide to Core Logging for Rock Engineering. Proceedings of the Symposium on Exploration for Rock Engineering, Vol. 1.

4. Elges, H F W K (1985), Dispersive Soils. Problem Soils in South Africa – State of the Art. The Civil Engineer in South Africa – July 1985.

5. Elges, H F W, Legge, W C S and Legge, K R, (1994), Earth and Rockfill Dams. South African National Committee on Large Dams.

6. FEMA (2011), Filters for Embankment Dams. Best Practices for Design and Construction. Federal Emergency Management Agency.

7. Fernandez, L M and Guzman, J A (1979), Earthquake Hazard in Southern Africa. Geological Survey of South Africa.

8. Houlsby, A C (1976), Routine Interpretation of the Lugeon Water-Test. Quarterly Journal of Engineering Geology, Vol. 9.

9. Houlsby, A C (1983), Cement Grouting. Water Resources Commission, NSW Australia.

10. Jennings, J E, Brink, A B A and Williams, A A B, (1973), Revised Guide to Soil Profiling for Civil Engineering Purposes in Southern Africa. Transactions of the South African Institution of Civil Engineers, Vol. 15, No. 1.

11. Jermy, C A and Walker, D J H, (1999), Assessing the Dispersivity of Soils. In Geotechnics for Developing Africa, Wardle G R, Blight, G E and Fourie, A B (editors). Proceedings of the Twelfth Regional Conference for Africa on Soil Mechanics and Geotechnical Engineering, Durban, South Africa.

12. Johnson, M R, Anhaeusser, C R and Thomas, R J (2006), Geology of South Africa. Council for Geoscience and Geological Society of South Africa.

13. Melvill, A L, (2002), Foundations and Construction Materials. Design and Rehabilitation of Dams: Short Course. SANCOLD / Institute of Water and Environmental Engineering, University of Stellenbosch.

14. Mouton, D J (2010), Engineering Geological Site Evaluations and Terrain Investigations for Dams. South African National Committee on Large Dams. Short Course: Basic Principles of Design, Construction and Evaluation of Small to Medium Dams, Especially Embankment Dams.

15. Portland Cement Institute, (1994), Fulton's Concrete Technology, Seventh (Revised) Edition, Addis, B J (editor). Portland Cement Institute.

16. Romana, M (2004), DMR (an adaptation of RMR), A new Geomechanics Classification for Use in Dam Foundations. STMR 9th Congresso Luso de Geotecnia, Aveiro.

17. SANRAL (2010), Standard Specifications for Subsurface Investigations.

18. SANS 1083 (2006), Aggregates from Natural Sources – Aggregates for Concrete. South African National Standards.

19. SANS 2001 – BE1 (2008), Classes of Excavation. South African National Standards.

20. USBR, (1974), Earth Manual. US Department of Interior Bureau of Reclamation, A Water Resources Technical Publication.

21. Van Den Berg, M and Parrock, A L (2009), Major Dam Foundation Design and Validation. Sustainable Development of Dams in Southern Africa, SANCOLD Conference, 4 – 6 November 2009.

22. Van Schalkwyk, A, Mouton, D and Van Der Merwe, A, (2009), Pitfalls in the Interpretation of Information from Rotary Core Drilling for the Prediction of Foundation Conditions for Dams. Sustainable Development of Dams in Southern Africa, SANCOLD Conference, 4 – 6 November 2009.

23. Weinert, H H, (1980), The Natural Road Construction Materials of Southern Africa. Academia, Cape Town.