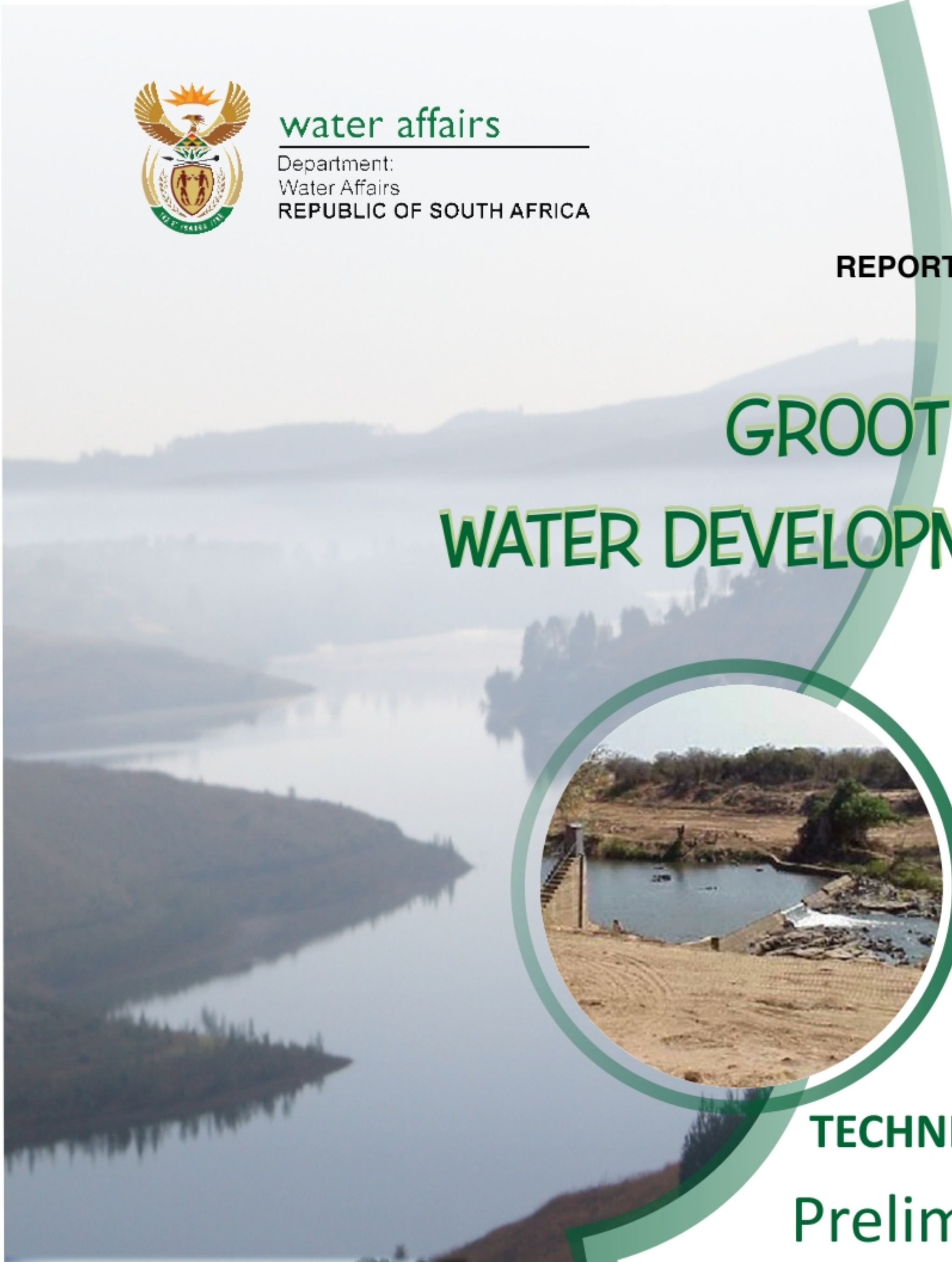
water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA

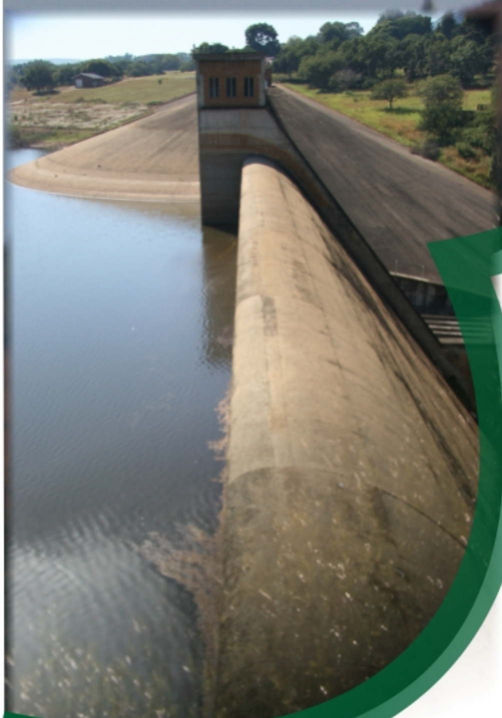
REPORT NO.: P WMA 02/B810/00/1110/2



GROOT LETABA RIVER WATER DEVELOPMENT PROJECT (GLeWaP)

TECHNICAL STUDY MODULE:

Preliminary Design of Nwamitwa Dam



VOLUME 6

Annexure 2: Appendix B (Part 1): Geotechnical Investigations

MAY 2010

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KLM Consulting Services
Urban-Econ Developmental Economics
Schoeman & Associates

W54_2010/H505701

LIST OF STUDY REPORTS IN GROOT LETABA WATER DEVELOPMENT PROJECT (BRIDGING STUDIES)

This report forms part of the series of reports, done for the bridging studies phase of the GLeWaP. All reports for the GLeWaP are listed below.

REPORT NUMBER	REPORT TITLE
P WMA 02/B810/00/0508	**Project Coordination and Management Team**
P WMA 02/B810/00/0508/1	Project Coordination and Management Team: Executive Summary Report: Vol 1
P WMA 02/B810/00/0508/2	Project Coordination and Management Team: Main Report: Vol 2
P WMA 02/B810/00/0508/3	Project Coordination and Management Team: Register of Decisions: Vol 3
P WMA 02/B810/00/0508/4	Project Coordination and Management Team
P WMA 02/B810/00/0508/5	Project Coordination and Management Team
P WMA 02/B810/00/0608	**Technical Study Module**
P WMA 02/B810/00/0608/1	Technical Study Module: Main Report: Vol 1
P WMA 02/B810/00/0608/2	Technical Study Module: Review of Water Requirements: Vol 2
P WMA 02/B810/00/0608/3	Technical Study Module: Groundwater: Vol 3
P WMA 02/B810/00/0608/4	Technical Study Module: Hydrology: Vol 4
P WMA 02/B810/00/0608/5	Technical Study Module: Water Resource Analysis: Vol 5
P WMA 02/B810/00/0608/6	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6
P WMA 02/B810/00/1110/1	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 1: Appendices
P WMA 02/B810/00/1110/2	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 2: Appendix B (Part 1): Geotechnical Investigations
P WMA 02/B810/00/1110/3	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 3: Appendix B (Part 2): Geotechnical Investigations
P WMA 02/B810/00/1110/4	Technical Study Module: Preliminary Design of Nwamitwa Dam: Vol 6 - Annexure 4: Appendix H: Drawings
P WMA 02/B810/00/0608/7	Technical Study Module: Preliminary Design of the Raising of Tzaneen Dam: Vol 7
P WMA 02/B810/00/0608/8	Technical Study Module: Bulk Water Distribution Infrastructure: Vol 8
P WMA 02/B810/00/1110/5	Technical Study Module: Bulk Water Distribution Infrastructure: Vol 8 - Annexure 1 : Appendices
P WMA 02/B810/00/0708	**Environmental Management Module**
P WMA 02/B810/00/0708/1	Environmental Management Module: Scoping Report: Vol 1
P WMA 02/B810/00/0708/2	Environmental Management Module: Environmental Impact Assessment Report: Vol 2
P WMA 02/B810/00/0708/3	Environmental Management Module: Environmental Management Programme for Borrow Area 1 on the Farm Laborie 515: Vol 3
P WMA 02/B810/00/0708/4	Environmental Management Module: Environmental Management Programme for Borrow Area 2 on the Farm La Parisa 729 (Gubits Farm): Vol 4
P WMA 02/B810/00/0708/5	Environmental Management Module: Environmental Management Programme for Borrow Area 3 on the Farm Letaba Drift 727: Vol 5
P WMA 02/B810/00/0708/6	Environmental Management Module
P WMA 02/B810/00/0708/7	Environmental Management Module
P WMA 02/B810/00/0708/8	Environmental Management Module
P WMA 02/B810/00/0708/9	Environmental Management Module
P WMA 02/B810/00/0708/10	Environmental Management Module
P WMA 02/B810/00/0808	**Public Involvement Program**
P WMA 02/B810/00/0808/1	Public Involvement Program: Main Report: Vol 1
P WMA 02/B810/00/0808/2	Public Involvement Program
P WMA 02/B810/00/0808/3	Public Involvement Program
P WMA 02/B810/00/0808/4	Public Involvement Program
P WMA 02/B810/00/0808/5	Public Involvement Program

EXECUTIVE SUMMARY

Geotechnical investigations were conducted at the proposed dam site, as well as for the bulk water supply and the roads to be relocated. Material investigations were also carried out.

Investigations comprised a desk study of available data and aerial photograph interpretation (API), followed by field work which included engineering geological mapping, boreholes, test pits and a programme of laboratory and field testing. Geophysical investigations were conducted, as was a Seismic Hazard Assessment (SHA).

The area of interest is underlain by Mesoaechean granitoid gneisses, specifically the Groot Letaba Gneiss (previously Goudplaats Gneiss) which has been intruded by younger dolerite dykes. No major faults occur in the vicinity of the dam but a number of lineaments are present. The level of seismic hazard may be described as moderate.

At the dam site, shallow colluvial soils cover the left flank while the right flank is partly covered by reworked alluvial gravels and the remainder by colluvial sands; underlain by thin residual soils. Thick alluvial deposits occur within the river section. The granite bedrock generally occurs at shallow depth on the left flank but is deeply and variably weathered. On the right flank the bedrock is generally moderately or highly weathered. Within the river section the underlying bedrock is generally unweathered.

For the dam foundations, founding criteria are defined and expected founding depths for either mass concrete or embankments are determined on the basis of empirically determined Deformation Modulus (E_{mod}) values for the founding rock mass. Lugeon (packer) tests are used to assess the permeability of the founding rock mass which proved mainly to be impervious.

Two potential hard rock quarry sites were investigated as possible sources of coarse aggregate / rip-rap materials. No further explorations of other dam construction materials (semi-pervious and impervious, fine aggregate) were conducted to complement previous work by the Department of Water Affairs and Forestry.

Geological conditions along the proposed re-aligned sections of road were characterised using test pits and laboratory testing, and the suitability of the respective layers for use in the road construction was assessed.

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APPENDICES

Please note: These are contained in a separate Volume entitled *"Technical Study Module: Preliminary Design of Nwamitwa Dam: Volume 6 - Annexure 3: Appendix B (Part 2): Geotechnical Investigations*

Aurecon Electronic Reference: I:\HYDRO\401775 Groot Letaba\Reports\Vol 6 Annex 2 App B Part 1 Geotech Text\Vol 6 Annex 2 App B Part 1 GLeWaP Geotechnical Report text (Final).doc

ABBREVIATIONS

DWA	Department of Water Affairs
GLeWaP	Groot Letaba River Water Development Project
OA	Options Analysis
PCMT	Project Co-ordination and Management Team
PSP	Professional Service Provider
CGS	Council for Geoscience
API	Aerial photograph interpretation
SHA	Seismic hazard assessment
AAR	Alkali aggregate reaction
UCS	Unconfined / uniaxial compressive strength

1 STUDY INTRODUCTION

1.1 BACKGROUND TO PROJECT

The catchment of the Groot Letaba River has many and varied land uses with their associated water requirements. These include significant use by agriculture in the form of irrigated crops, commercial afforestation, tourism (particularly linked to the Kruger National Park, which lies partially within the catchment), as well as primary demands by the population in the catchment. The water resources available in the catchment are limited, and considerable pressure has been put on these resources in the past, with periods of severe and protracted water restrictions occurring over the past 25 years. This situation has been investigated at various levels by the Department of Water Affairs (DWA).

The first major study undertaken for this area was the Letaba River Basin Study in 1985 (DWAF, 1990), which comprised the collection and analysis of all available data on water availability and use, as well as future water requirements and potential future water resource developments. This was followed by a Pre-feasibility Study (DWAF 1994), which was completed in 1994. The focus of the Pre-feasibility Study was the complete updating of the hydrology of the Basin. The next study undertaken was the Feasibility Study of the Development and Management Options (DWAF, 1998), which was completed in 1998.

The Feasibility Study proposed several options for augmenting water supply from the Groot Letaba River. These included some management interventions, as well as the construction of a dam at Nwamitwa and the possible raising of Tzaneen Dam. These options would enable additional water to be allocated to the primary water users, would allow the ecological Reserve to be implemented and could also improve the assurance of supply to the agricultural sector.

This Bridging Study was initiated by the (then) Department of Water Affairs and Forestry (now DWA) in 2006 in order to re-assess the recommendations contained in the Feasibility Study in the light of developments that have taken place in the intervening 10 years. Other contributing factors to the DWA's decision to undertake Bridging Studies were the promulgation of the Water Services Act and the National Water Act in 1997 and 1998 respectively, and the recently completed Reserve Study on the Letaba River.

The study area is shown in **Figure 1.1.** It consists of the catchment of the Groot Letaba River, upstream of its confluence with the Klein Letaba River. The catchment falls within the Mopane District Municipality, which is made up of six Local Municipalities. The four Local Municipalities that lie fully or in part within the catchment area are Greater Tzaneen, Greater Letaba, Ba Phalaborwa and Greater Giyani. The major town in the study area is Tzaneen, with the Limpopo capital city of Polokwane located just outside of the catchment to the West.

The site of the proposed Nwamitwa Dam is also shown in **Figure 1.1**. The focus of the Feasibility Study was the Groot Letaba catchment, with the catchments of the other rivers included to check that the environmental flow requirements at the Kruger National Park, and international agreements regarding flow entering Mozambique were met. This focus was kept for this Bridging Study.

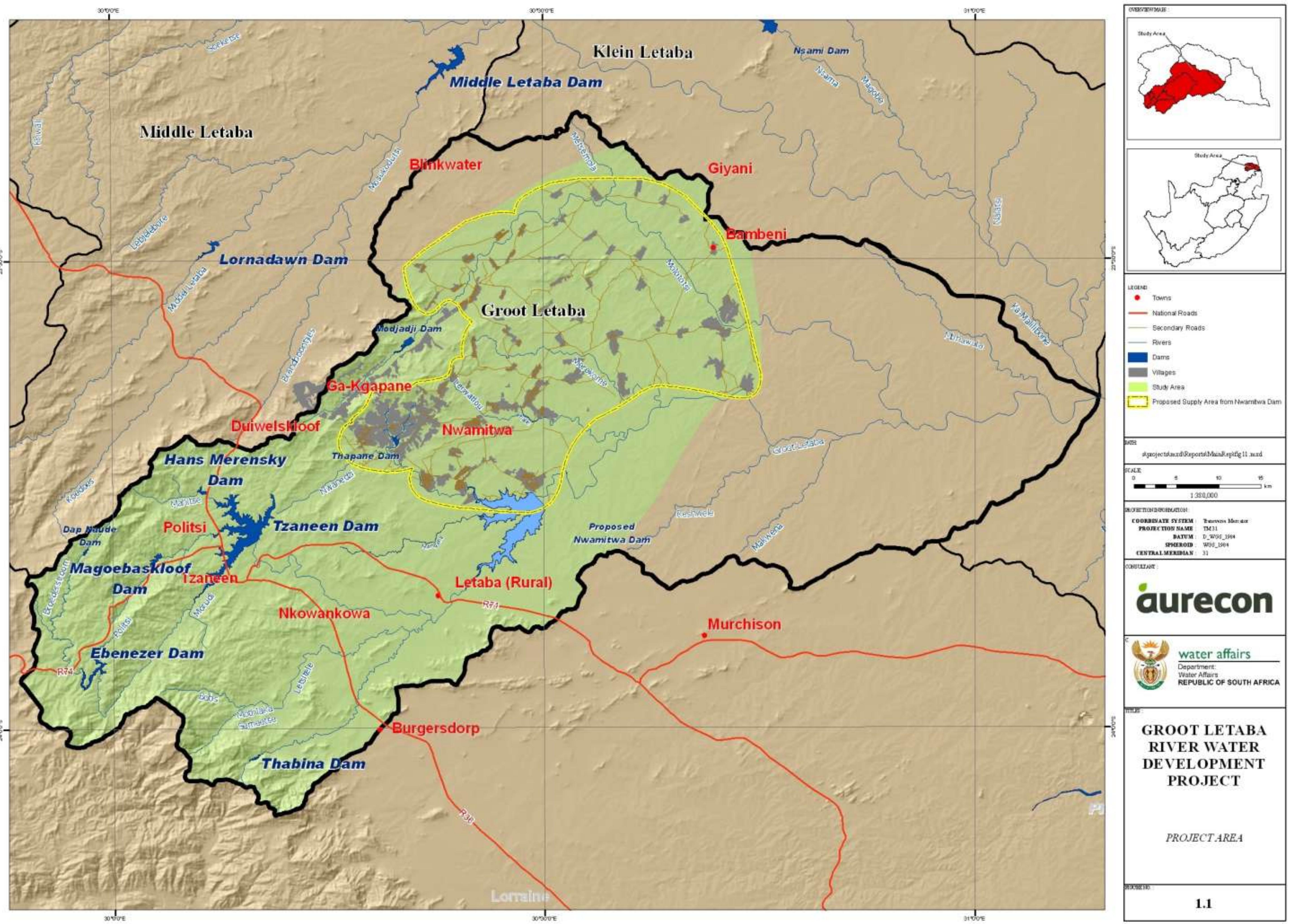
1.2 SCOPE AND ORGANISATION OF PROJECT

The Department's Directorate: Options Analysis (OA), appointed Aurecon in Association with a number of sub consultants (listed below) to undertake this study. The official title of the study is: "The Groot Letaba River Water Development Project: Bridging Studies: Technical Study Module".

An association exists between the following consultants for the purposes of this study:

- Aurecon (previously Ninham Shand)
- Semenya Furumele Consulting
- KLM Consulting Services
- Urban-Econ Developmental Economists
- Schoeman & Vennote

The Bridging Study comprises a number of modules, namely: an Environmental Management Module (EMM), a Public Involvement Programme (PIP), and a Technical Study Module (TSM). This Report focuses on part of the scope of work for the (TSM).


Klein Letaba
Middle Letaba Dam
Nsami Dam
Middle Letaba
Blinkwater
Giyani
Bambeni
Lornadawn Dam
Groot Letaba
Modjadji Dam
Ga-Kgapane
Duiwelskloof
Nwamitwa
Thapane Dam
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Politsi
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R71
R36
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National Roads
Secondary Roads
Rivers
Dams
Villages
Study Area
Proposed Supply Area from Nwamitwa Dam
SCALE
1:350,000
PROJECTION INFORMATION :
COORDINATE SYSTEM : Transverse Mercator
PROJECTION NAME : TM31
DATUM : D_WGS_1984
SPHEROID : WGS_1984
CENTRAL MERIDIAN : 31
CONSULTANT :
aurecon
water affairs
Department:
Water Affairs
REPUBLIC OF SOUTH AFRICA
TITLE :
GROOT LETABA RIVER WATER DEVELOPMENT PROJECT
PROJECT AREA
1.1

1.3 SCOPE OF THIS REPORT

This report presents the findings of the geotechnical investigations conducted for;

- The dam footprint,
- Construction materials, including sources of coarse aggregate / rip-rap as well as borrow areas for embankment materials, and the
- Re-aligned roads.

Geotechnical investigations were also conducted for the Bulk Water Infrastructure; the findings of which are presented in a separate report entitled *Groot Letaba River Water Development Project (GLeWaP): Bulk Water Infrastructure*: *Volume 8 - Annexure 1: Appendices (DWAF, 2010).*

2 GEOTECHNICAL INTRODUCTION

Africon (Pty) Ltd were appointed by Ninham Shand (Pty) Ltd (now merged to form Aurecon) in their letter reference 401775/WW/4050/L01/O-2058, dated 17 July 2008, as a sub-consultant for the Groot Letaba River Water Development Project (GLeWaP): Post Feasibility Bridging Studies.

The appointment was to provide technical input and provide deliverables associated with the geotechnical investigations for the proposed Nwamitwa Dam, relocation of roads, and bulk water supply infrastructure.

The results of these investigations are presented in this report.

As mentioned in Section 1.3, the geotechnical investigations conducted for the Bulk Water Infrastructure are presented in a separate report entitled *Bulk Water Infrastructure: Volume 8 - Annexure 1: Appendices (DWAF, 2010).*

3 PREVIOUS GEOLOGICAL / MATERIALS INVESTIGATIONS

A number of previous engineering geological studies for the proposed Nwamitwa Dam had been conducted. These are briefly summarized below, in chronological order.

A **reconnaissance-level appraisal** of the proposed dam site, then known as the Janetsi site was conducted initially (*Dooge, N. 1994. Letaba Water Supply Scheme: Great Letaba River: Janetsi Weir Site: Letaba District. First Engineering Geological Reconnaissance Report. Unpublished Council for Geoscience Report, No 1984-0081*). The investigation comprised a desk study followed by a brief site visit during which an assessment of likely geological conditions along the proposed centre-line was made. Possible sources of construction materials were listed, as derived from an earlier broad study in the area.

Some years later this was followed by **feasibility-level engineering geological investigations** which were conducted in 1996 (*Dooge, 1997. Letaba Water Supply Scheme: Great Letaba River: Nwamitwa Dam Site: Letaba District. First Engineering Geological Feasibility Report. Unpublished Council for Geoscience Report 1997-0009*) during which a total of seventeen exploratory boreholes were drilled (numbered BH 1001 to BH 1017), at two possible centre-lines. Investigations shifted to the upstream centre-line after initial boreholes at the downstream site revealed unfavourable conditions.

The report provided an assessment of expected excavation depths for a central concrete spillway section as well as typical excavation depths for the core trench on the embankments and included geological sections along the respective centre-lines. No investigations of potential sources of construction materials were conducted, although various sources of both fine (i.e. sand) and coarse aggregate were identified.

At approximately the same time **materials investigations** were conducted by the DWA Materials Laboratory (DWAF report no B801/41/IJ01) into sources of earthfill materials as well as fine aggregate (sand). One site for potential earthfill materials was identified, and two sites for fine aggregate. Samples for laboratory testing were recovered by a programme of auger drilling. The results of the testing were presented in the above mentioned report.

4 INVESTIGATION METHODOLOGY

4.1 OUTLINE

A broad outline of the geotechnical investigations conducted for these Post-Feasibility Bridging Studies is presented below.

These geotechnical studies comprised the following:

- Desk study of available geological information,
- Field mapping,
- Geophysical surveys,
- Additional rotary diamond core drilling,
- Test pitting,
- Water pressure (Lugeon) testing and measurement of the water table,
- Laboratory and field testing, and a
- Seismic hazard assessment,

4.2 DESK STUDY

Available geological data was assessed. This included the previous engineering geological reports, as listed above, as well as available maps as listed below;

- Topo-cadastral map 1:50 000 Sheet 2330 CD Letsitele *(Chief Director of Surveys and Mapping)*
- Published geological map 1:250 000 Sheet 2330 Tzaneen *(Council for Geoscience)*
- Published metallogenic series map 1:250 000 Sheet 2330 Tzaneen *(Council for Geoscience)*
- Unpublished geological map 1:50 000 Sheet 2330 CD Letsitele *(Council for Geoscience)*

In addition, aerial photograph interpretation (API) was conducted, using photographs originally flown for this project on behalf of DWA (Job no W501, scale 1:8000). These photographs are listed below in **Table 4.1**. The primary aim of the API was the identification of major structural features, such as faults or lineaments etc, which might negatively impact on the founding of a large dam. The results of the API are included on the engineering geological site plan (Drawing 103577-G1-003).

Table 4.1 List of aerial photographs

Strip no.	Photograph numbers
6	28 – 31
7	31 – 35
8	61 – 67
9A	188 - 193

4.3 ENGINEERING GEOLOGICAL MAPPING

Field mapping of the general area of the proposed dam footprint was conducted in order to produce an engineering geological site plan (Drawing 103577-G1-003) which included geological boundaries and the results of the aerial photograph interpretation as well as borehole and test pit positions.

Where rock outcrops were present, a random discontinuity survey was conducted, using a Breithaupt compass. Joint orientations were adjusted for magnetic declination. The discontinuity data was reworked using DIPS® software.

Although co-ordinates of test pits and the later rotary core boreholes were picked up by means of a hand-held GPS apparatus, using the WGS 84 coordinate system, Hartebeesthoek94 Datum (Lo 31), the base maps supplied by DWA were still in Cape Datum. These DWA maps, in Cape Datum, form the base map for the engineering geological site plan, and the co-ordinates were subsequently all converted back to Cape Datum for sake of consistency.

4.4 GEOPHYSICAL SURVEYS

Geophysical surveys were conducted at the proposed dam site as well as the proposed quarry sites by Engineering and Exploration Geophysical Services cc (EEGS).

The reports by EEGS are included as **Appendix H**, while summarised survey results are discussed in the appropriate sections below.

Three geophysical techniques were employed for this geophysical survey, namely surveys of seismic velocity, electro-magnetics as well as a magnetic survey. Instruments utilised included a Smartseis seismograph, a proton magnetometer and a Geonics EM34-3.

4.5 ROTARY CORE DRILLING

As mentioned, a total of seventeen boreholes (numbered BH 1001 to BH 1017, total length 450,23 m) were drilled in the course of the initial feasibility-level investigations (Council for Geoscience, 1997) in the period February 1996 to December 1996. At the time two alternative centre-lines were investigated; seven boreholes were drilled along Centre-line 1 and ten boreholes were drilled along the upstream alternative, namely Centre-line 2. Two boreholes at the top of the right flank were shared by the two centre-lines. All these boreholes were drilled vertically.

Prior to drilling for these Post-Feasibility Bridging Studies, it was decided that the Bridging Studies would only investigate the upper Centre-line 2 option. It should be noted that the upper right flank alignment has changed slightly as a result of an increased dam height and the need to accommodate the topography.

A further twelve rotary core boreholes were subsequently drilled by the DWA Drilling Branch, in the period September 2007 to December 2008. Boreholes on the upper flank areas were drilled vertically, while boreholes on the respective river banks were angled into the river section. In addition to the boreholes drilled on the dam footprint, a further six rotary boreholes were drilled at two possible hard rock quarry sites (numbered NBH 3001 to NBH 3003, and NBH 3101 to NBH 3103). These quarry sites are shown on Drawing 103577-G1-001.

The new boreholes were primarily drilled to fill in gaps from the previous investigations. Founding conditions for an envisaged side-channel spillway and stilling basin on the left flank were investigated by three boreholes (NBH 1202 to NBH 1204). An additional borehole (NBH 1213) was drilled to investigate founding conditions at the position of an anomaly identified during the geophysical survey.

Borehole details including the boreholes drilled during the previous investigations are summarised below in **Table 4.2**. All borehole positions are shown on Drawing103577-G1-002. Borehole logs and core photographs are presented in **Appendices B** and **C**, respectively. Borehole cores were logged in accordance with accepted South African standards, as per Core Logging Committee (1976) and the borehole logs were prepared using dotPLOT® software.

Note that borehole co-ordinates listed in **Table 4.2** are in accordance with the WGS84 co-ordinate system, while the Cape Datum co-ordinates are listed on the Site Plan (Drawing 103577-G1-002). The positions of the initial boreholes were picked up by detailed surveying and the respective borehole collar elevations are therefore assumed to be accurate. The current boreholes were picked up by hand-held GPS and the collar elevations, in particular, can only be assumed to be approximate. In order to improve the accuracy the collar elevations were adjusted by interpolation from the longitudinal section (Drawing 103577-G1-004) with the previous surveyed levels used as control points.

Table 4.2 Rotary boreholes summary

Borehole No	Co-ordinates (WGS84)		Reduced Level (masl)	BH length (m)	Remarks[1]
	Y	X			
BH 1001	51 698	2 628 033	464,08	24,82	Lower centre-line 1
BH 1002	51 691	2 628 087	460,87	30,40	Lower centre-line 1
BH 1003	51 691	2 628 108	451,35	19,66	Lower centre-line 1
BH 1004	51 693	2 628 142	454,02	30,20	Lower centre-line 1
BH 1005	51 686	2 628 177	455,51	29,80	Lower centre-line 1
BH 1006	51 685	2 628 220	464,11	35,00	Lower centre-line 1
BH 1007	51 689	2 628 288	463,40	36,00	Lower centre-line 1
BH 1008	51 288	2 629 044	475,61	24,13	No Lugeon test
BH 1009	51 921	2 628 390	463,90	35,05	No Lugeon test
BH 1010	52 048	2 628 255	462,88	21,20	No Lugeon test
BH 1011	52 121	2 628 178	463,36	19,55	No Lugeon test
BH 1012	52 292	2 628 011	470,76	27,60	No Lugeon test
BH 1013	52 378	2 627 865	474,68	15,12	
BH 1014	52 877	2 627 856	479,75	20,65	
BH 1015	52 645	2 627 862	477,20	21,05	No Lugeon test
BH 1016	51 688	2 628 637	470,83	30,00	No Lugeon test
BH 1017	51 477	2 628 828	477,40	26,30	No Lugeon test
NBH 1201	52 251	2 627 912	451	21,15	
NBH 1202	52 189	2 627 957	480	15,50	No Lugeon test
NBH 1203	52 019	2 628 004	476	15,65	No Lugeon test
NBH 1204	51 849	2 628 051	437	19,80	No Lugeon test
NBH 1205	51 956	2 628 256	456	50,10	Angled borehole (60°).
NBH 1206	51 995	2 628 288	452	50,42	Angled borehole (60°).
NBH 1207	51 950	2 628 331	448	50,10	Angled borehole (60°).
NBH 1208	51 899	2 628 299	453	50,50	Angled borehole (60°).
NBH 1209	51 855	2 628 415	463	25,15	
NBH 1210	51 732	2 628 557	465	20,50	
NBH 1211	51 576	2 628 735	472	15,50	

Borehole No	Co-ordinates (WGS84)		Reduced Level (masl)	BH length (m)	Remarks[1]
	Y	X			
NBH 1212	51 372	2 628 878	476	15,30	
NBH 1213	52 138	2628 150	463	29,85	Angled borehole (60°). No Lugeon test – packer problems
NBH 3001	62 717	2 622 140	632	19,85	Quarry borehole. No Lugeon test
NBH 3002	62 633	2 622 088	622	15,43	Quarry borehole. No Lugeon test. Drilling difficulties.
NBH 3003	62 569	2 622 052	619	20,18	Quarry borehole. No Lugeon test
NBH 3101	64 993	2 638 324	583	20,60	Quarry borehole. No Lugeon test
NBH 3102	64 883	2 638 363	589	20,75	Quarry borehole. No Lugeon test
NBH 3103	64 952	2 638 364	589	19,70	Quarry borehole. No Lugeon test

Notes:
1 Unless otherwise stated, all boreholes are drilled vertically, and Lugeon tests were conducted.

4.6 TEST PITTING

Prior to these Bridging Studies, only disturbed borehole samples of the upper soil horizons were available from the rotary core drilling programme. In order to assess the undisturbed *in situ* soil conditions, and to provide opportunity for sample collection for laboratory testing, a total of eleven (11) test pits were excavated on the dam footprint.

Test pits were excavated by light TLB from the DWA; either to refusal, or to the maximum reach of the machine, or to a point when the sidewalls were deemed unstable. Soil profiles were described in accordance with accepted South African standards (Jennings, Brink and Williams, 1973). Soil profiles were prepared using dotPLOT® software and are presented in **Appendix D**, and are summarised below in **Tables 4.3** and **4.4** (dam footprint, and realigned roads centre-lines, respectively). Test pit positions on the dam footprint are shown on Drawing 103577-G1-002, and for the re-aligned roads centre-lines on Drawing 103577-G1-006. The co-ordinates are indicated on the respective drawings in Cape Datum (to conform with the base maps), but included below in **Tables 4.3** and **4.4** in accordance with WGS84.

Table 4.3 Test pit summary (dam)

Test pit number	Co-ordinates (WGS84)			Remarks
	Y	X		
NTP01	51 940	2 628 338	River section	Alluvium to depth 3,1 m. No refusal.
NTP02	51 914	2 628 360	River section	Transported soils to 3 m. No refusal.
NTP03	51 739	2 628 550	Lower right flank	Near-refusal at 2,4 m.
NTP04	51 564	2 628 752	Mid-right flank	Refusal at 1,4 m.
NTP05	51 251	2 629 120	Upper right flank	Refusal at 1,25 m.
NTP06	51 059	2 629 328	Upper right flank	Refusal at 1,65 m.
NTP07	50 877	2 629 571	Upper right flank	Refusal at 2,1 m but some collapse at 1,5 m (seepage at 1,65 m).
NTP08	52 726	2 628 874	Upper left flank	Hole collapsing at 2,4 m because of seepage. No refusal.
NTP09	52 379	2 628 879	Upper left flank	Difficult excavation at 2,6 m, but not refusal.
NTP10	52 218	2 628 080	Mid-left flank	Refusal at depth 1,9 m on hard rock boulders.
NTP11	52 007	2 628 241	Lower left flank / river secion	Alluvium to 3,2 m. No refusal.

No additional test pitting was conducted for potential sources of embankment materials (either semi-pervious or impervious core materials).

Table 4.4 Test pit summary (relocated roads centre-lines)

Test pit number	Co-ordinates (WGS84)		Total depth (m)	Remarks
	Y	X		
R529				
R01	53 619	2 627 674	1,6	Refusal on very dense residual gneiss
R02	53 902	2 627 974	1,3	Refusal on very dense residual gneiss
R03	54 187	2 628 271	1,8	Refusal on very dense residual gneiss
R31	55 028	2 629 640	1,7	Refusal on very dense residual gneiss
R42	55 229	2 631 239	1,7	Refusal on very dense residual gneiss
R43	55 404	2 631 504	1,1	Refusal on very dense residual gneiss
P43/3				
TP2A	56 332	2 636 009	1,08	Refusal on granite gneiss boulders
TP3A	55 145	2 635 196	1,18	Refusal on granite gneiss boulders
TP6	53 600	2 633 504	1,79	Refusal on soft rock granite gneiss
TP7	52 696	2 632 837	1,80	Refusal on very dense residual granite gneiss/ soft rock
TP8	51 543	2 632 952	0,77	Refusal on granite gneiss boulders
TP9	50 942	2 632 153	1,32	Refusal on granite gneiss boulders
TP10	50 321	2 631 421	1,12	Refusal on granite gneiss boulders
TP11	49 579	2 630 719	1,14	Refusal on granite gneiss boulders
D1292				
R21	55 599	2 629 574	1,7	Refusal on very dense residual gneiss
R22	55 247	2 629 566	1,7	Refusal on very dense residual gneiss
Borrow pits				
BP01	55 742	2 625 351	2,0	Refusal on very dense residual gneiss
Borrow Pit A	49 441	2 630 714	1,02	Refusal on soft rock granite and possibly boulders
Borrow Pit A1	49 466	2 630 745	1,60	Refusal
Borrow Pit B	51 551	2 632 543	1,56	Refusal on possibly granite boulders

4.7 BOREHOLE TESTING

As a rule the exploratory boreholes were water pressure tested (Lugeon tested), although in a number of instances tests could not be carried out due to problems in inserting of the packer.

Tests were conducted in accordance with the methodology described by Houlsby (1976). Detailed test results are presented in **Appendix E**, while summarised results are included in the borehole logs (**Appendix B**).

Water table measurements were recorded in the boreholes after drilling was completed. These results are discussed in Section 7.1.4.

4.8 FIELD TESTING

Apart from testing within boreholes as described above, no other field testing was conducted, with the exception of Dynamic Cone Penetration (DCP) testing conducted along the road alignments. Summarised results are included in Section 6.3, while detailed results are presented in **Appendix F**.

4.9 LABORATORY TESTING

Representative samples recovered from the test pits on the dam footprint were submitted to the DWA Materials Laboratory for testing. Testing of the disturbed samples comprised determination of foundation indicators (i.e. grading analyses including both sieve and hydrometer analyses, as well as determination of the Atterberg limits), as well as determination of compaction properties, shear strengths and permeabilities. In addition, a limited number of undisturbed samples were also submitted for which the field densities and moisture contents were determined. Undisturbed samples could only be successfully recovered from the finer-grained alluvial deposits. Attempts to recover undisturbed samples of the coarser grained palaeo-alluvium or the residual granite soils were not successful.

Point load strength (PLS) testing was conducted on borehole cores from the dam centre-line as well as from the hard rock quarry sites, and a limited number of cores (6 No) were also submitted to Rocklab (Pty) Ltd for uniaxial compressive strength (UCS) testing. In addition, the Elastic Modulus (E_{mod}) as well as the Poisson's ratio and the density of the intact rock material were also determined. Two rock samples from the potential quarry site were submitted to the Council for Geoscience for petrographic analyses.

Disturbed samples from the relocated roads were also submitted to the DWA Materials Laboratory for testing. Tests conducted included the following: road indicators (grading analyses, Atterberg Limits), compaction characteristics (Mod ASSTHO, NRB and CBR's as well as Proctor compaction), and uniaxial compressive strength testing.

Borehole cores drilled at the potential hard rock quarry sites were subjected to the suite of tests defined in SABS 1083 (SABS, 1994), by the DWA Materials Laboratory, to confirm the suitability of the rock for use as coarse aggregate.

Test results are presented in **Appendix F**, and are summarized in the appropriate sections in the body of this report.

4.10 SEISMIC HAZARD ASSESSMENT

A seismic hazard assessment (SHA) for the proposed Nwamitwa Dam was conducted by the Council for Geoscience (CGS, 2008). The probabilistic seismic hazard assessment was performed using the classical Cornell-McGuire procedure; with earthquake recurrence parameters calculated using the procedure of Kijko and Sellevoll. Peak ground acceleration (PGA) values were calculated for the Operating Basis Earthquake (OBE) and the Maximum Credible Earthquake (MCE).

The CGS report is included as **Appendix G**, with the main findings summarised in the text of this geotechnical report (Section 5.3).

5 GENERAL GEOLOGICAL SETTING

5.1 LITHOLOGY AND STRATIGRAPHY

The proposed Nwamitwa Dam is located on the Kaapvaal Craton which describes a major stable area of the earth's crust which in this case is underlain by Archean granitoids, gneisses and migmatites interspersed with greenstone belts. These granitoid rocks represent major crust-forming magmatic episodes which may be associated with the development of Archean greenstone belts.

According to Robb, *et al* (2006), the proposed Nwamitwa Dam is located in an area underlain by granitoid gneisses of the Groot-Letaba Gneiss, which describes all granitoid gneisses occurring between the Murchison and Pietersburg-Giyani greenstone belts. Previously these rocks were considered to be part of the widespread Goudplaats Gneiss.

The Groot Letaba Gneiss encompasses a wide variety of closely intermingled gneisses including minor banded and linear gneisses. Individual units have not yet been delineated. The main constituents of the rocks include oligoclase, quartz, biotite and microcline. The rocks are well to weakly foliated and strongly folded, but can have an almost massive appearance in places. The rocks are generally migmatised. In places the gneiss contains small rafts of mafic to ultramafic greenstone fragments.

The Archean granitoid rocks have been intruded by younger dolerite dykes.

These granitoid gneisses are considered by Robb *et al* (2006) to be Mesoarchean intrusions, with ages between 3 200 Ma and 2 800 Ma.

5.2 STRUCTURAL GEOLOGY

The tectonic history of the area has imprinted a variably-developed foliation on the granitic gneisses which has a general north-easterly trend. The contacts between the greenstone belts and the granitoid rocks may be tectonic. This is further reflected in a number of prominent lineaments or shear zones in the general area which share a common NE-striking trend. Intrusive dolerite dykes might be expected to coincide with these lineaments, as suggested by the geological maps which show a pronounced NE-striking trend for these dolerite intrusions.

The available geological maps do not indicate any major faults in the vicinity of the proposed Nwamitwa Dam. However, aerial photograph interpretation (API) indicates a number of lineaments in the general area of the dam site (Drawing 103577-G1-003). These lineaments reflect a common NE trend, although occasional E-W striking and ESE striking lineaments are also noted. These lineaments may indicate fault- or shear zones, or may equally be zones of prominent jointing, or dolerite intrusions. Several NE-trending lineaments traverse the right flank of the proposed centre-line.

5.3 SEISMICITY

Published seismic hazard maps of Southern Africa (Kijko *et al,* 2003) indicated a peak ground acceleration (PGA) in the order of 0,11 g to 0,13 g for the area in which the proposed Nwamitwa Dam is located. This may be considered a moderate level of seismic hazard.

A site specific seismic hazard assessment was subsequently conducted by the Council for Geoscience (CGS, 2008), as part of this study. Detailed findings are included as **Appendix G**, but are summarised below.

The Peak Ground Acceleration (PGA) was calculated for the Operating Basis Earthquake (OBE) as well as the Maximum Credible Earthquake (MCE), where the OBE is defined as an earthquake having a 50% probability of exceedence in 100 years, i.e. a return period of 144 years. The return period of the MCE is considered to be 10 000 years. The OBE values fall within the range 0,01 g to 0,03 g. The MCE values fall in the range 0,09 g to 0,18 g.

5.4 CLIMATE AND WEATHERING

The area of interest is characterised by a climatic N-value in the order of 2 (Weinert, 1980) which indicates that a process of decomposition predominates. The resulting clay component developed on these acid crystalline rocks is kaolinite. Significant depths of weathering might be expected.

6 INVESTIGATION FINDINGS

6.1 NWAMITWA DAM SITE

6.1.1 Introduction

Earlier drilling results (Council for Geoscience, 1997) have been incorporated into this appraisal where appropriate, and were considered together with the findings of the recent investigations. The findings and conclusions of the previous studies might therefore not necessarily be valid when viewed in the context of additional data, and this report supersedes all previous reports.

Plate 1 A general view of the upper left flank, illustrating the gentle topography

At the time of the geotechnical investigations the proposed dam site was being used for agricultural purposes and primarily comprised citrus orchards although access roads also traverse the site (**Plate 1**).

6.1.2 Site geology

Left flank

(a) Description

The left flank is approximately 1350 m in length. The proposed centre-line is kinked at approximate Chainage 900 in order to make optimal use of the contours. The upper flank areas are very gently sloping (approximate gradient 1:800), steepening slightly in the mid-slope areas before flattening towards the river section (Drawing 103577-G1-002).

The upper left flank was investigated previously by five rotary diamond boreholes (BH 1011 to BH 1015). Recent investigations included three test pits (NTP 08 to NTP 09) and two additional boreholes (NBH 1201 and NBH 1213), where borehole NBH 1213 was drilled specifically to investigate a geophysical anomaly.

A further three boreholes (NBH 1202 to NBH 1204) were also drilled on the left flank, a short distance downstream of the centre-line, in order to investigate conditions for a possible side-channel spillway.

(b) Major structural features

No prominent lineaments traversing the dam footprint on the left flank were recognised during aerial photograph interpretation. It might be noted, however, that north-easterly striking lineaments were recognised upstream of the proposed centre-line (Drawing 103577-G1-003). If extended, these lineaments will pass through the dam footprint. As mentioned in Section 5.2 these lineaments may represent prominent shear zones or intrusive dolerite dykes.

Geophysical traverses (**Appendix H**) were also conducted on the left flank. A number of magnetic and electro-magnetic anomalies were identified on the upper left flank. These anomalies might represent fracture zones or intrusive dolerite dykes. Recorded seismic velocities on the lower flank areas adjacent to the river section indicate deeper ‘troughs’ in the lower velocity materials overlying the slightly weathered bedrock. The possibility that these ‘troughs’ or depressions in the bedrock profile represent buried palaeo-channels cannot be discounted at this stage. The previous report (Dooge, 1997) made mention of palaeo-alluvium but these occurrences could not be verified during the current investigation.

An additional borehole (NBH 1213) was subsequently drilled to investigate these anomalies on the lower left flank. Detailed results will be discussed in the following section, but it might be noted that the occurrence of quartz veins with a brecciated appearance suggests the possible occurrence of faulting.

(c) Geological profile

The left flank is covered by an upper horizon of colluvial soils which may be of hillwash origin. The thickness of this upper horizon varies between 0,25 m and 0,50 m. The materials may be described as comprising slightly moist, grey to occasionally dark grey or black, loose to medium dense, intact, silty sand; in places with minor medium gravels or in roughly equal proportions with medium to coarse gravels and cobbles. A poorly developed pebble marker comprising a thin horizon of rounded gravels is recognised in places.

On the lower flank area adjacent to the current river section previous boreholes indicated that the bedrock is overlain by slightly cemented alluvial materials suggestive of older deposits. Current investigations could not confirm the presence of these ‘older’ alluvial deposits.

An underlying horizon of residual soils, or reworked residual soils, is present beneath the colluvial soil cover. This horizon is generally between 0,45 m and 1,1 m in thickness and extends to depths between 0,7 m and 1,6 m. Typically this material comprises slightly moist, reddish brown or dark grey to orange, loose to dense, silty sand and gravel. This horizon is fairly variable, however, and in places grades into a combination of very soft rock granite, or core-stones of highly weathered, medium hard rock granite (diameter up to 500 mm), and loose sandy residual soils which extend to a depth between 1,6 m and at least 2,4 m (NTP08).

The upper horizon of granite bedrock intersected in the test pits may be described as highly weathered, grey-brown, very soft (friable) rock to medium hard rock granite. Occasional vein quartz is present as well as narrow pegmatite veins (thickness generally 0,4 m to 0,9 m). In places this weathered granite is intersected as shallow as 1,6 m (NTP09) but the profile is variable as confirmed by the exploratory drilling.

The boreholes confirm the variable bedrock depths. For the most part bedrock depths are relatively shallow, varying between 0,4 m and 4 m, but on the lower flank areas bedrock occurs at much greater depths varying between 8 m and 13 m.

The bedrock is characterised by deep, yet variable weathering on the upper flank. Although generally deeply weathered, an improvement in the degree of weathering is typically noted with increasing depth. The upper flank areas (approximate Chainages 0 to 700) are underlain by moderately weathered granite gneiss, becoming slightly weathered granite gneiss at a depth of 13,10 m (BH 1014). Similarly, the mid-flank areas are generally underlain by slightly weathered granite gneiss (approximate Chainages 850 to 1300), although overlying horizons of moderately weathered or moderately to highly weathered granite gneiss thicken in the direction of the river section. Deep weathering is revealed in Borehole NBH 1201 which suggests a 'trough-like' feature between approximate Chainages 700 and 800. In this area, completely weathered, soft rock granite (recovered as sand and gravel) extends to a depth of 4 m, underlain by highly weathered, medium hard rock granite gneiss extending to a depth of 10,45 m; below which the rock comprises moderately to highly weathered, hard rock to very hard rock granite gneiss.

Jointing of the rock mass is similarly variable. While none of the borehole cores were orientated and joint orientations are unknown, it is likely that at least one set of NE-striking joints is present reflecting the regional orientation of the lineaments. The joint spacing is broadly linked to the degree of weathering; the greater the weathering the more prominent the jointing, and vice versa. Numerous exceptions occur, however, and there are no hard and fast rules in this regard. Generally, joint spacing within the slightly weathered rock mass varies between very close and wide, while joints within the moderately to highly weathered horizons are generally very closely to medium spaced. Up to three joint sets are commonly recognised; comprising sub-horizontal joints (dipping between 0° and 20°), shallow- to moderately dipping joints (dipping between 20° and 40°) and sub-vertical joints (70° to 80°). For the most part, joint surfaces are rough and the surfaces are stained. Clay joint infill is uncommon, but is occasionally recorded in the upper rock horizons.

Only in one instance, namely the test pit (NTP 10), was a dolerite intrusion intersected, although additional dolerite dykes are likely. The dimensions and attitude of this dolerite intrusion are not known but at this stage is assumed to be NE-striking. The upper weathered horizon may be described as varying between slightly moist to wet, very dense, greenish grey, intact, clayey sand (completely weathered dolerite) and highly weathered, medium hard, occasionally hard rock dolerite. Machine refusal occurred at a depth of 1,9 m on boulders of hard rock dolerite.

(d) Groundwater conditions

Water seepage was recorded in two of the three test pits excavated during the course of this investigation, namely test pits NTP 08 and NTP 10 at respective depths of 2,3 m and 1,8 m. In both instances this seepage caused sidewall collapse of the test pits. It should be noted, however, that the seepage does not necessarily represent shallow groundwater; there is a possibility that the seepage originates from leaking irrigation pipes but this was not proven.

A water table was also measured at a depth of 6,38 m in NBH 1201 (approximately 444,5 masl), while water table measurements in the previous boreholes generally varied between 4 m and 7 m on the upper flank areas, becoming deeper on the lower flank areas, reflecting a close relationship with the bedrock horizon.

River section

(a) Description

The river section is relatively narrow with a width in the order of 200 m. The river channel itself is approximately 100 m with the river banks rising quite steeply from river level to the boundary with the lower flank areas.

With the exception of two boreholes drilled on the lower slopes of the respective flanks (BH's 1010 and 1009), no boreholes were previously specifically drilled in the river section of this Centre-line 2 option, although a number of boreholes were drilled in the river section of the downstream Centre-line 1 option (Drawing 103577-G1-003). For the current investigations two boreholes were drilled from each of the respective river banks (total 4 No, numbered NBH 1205 to NBH 1208), angled at 60° from horizontal into the river section in order to investigate the geological conditions within the river section.

In addition three test pits were excavated in the vicinity of the river section, namely NTP01 on the right bank of the river, and NTP11 and NTP02 on the lower slopes of the respective left and right flanks.

A general view of the river section is shown in **Plate 2**.

Plate 2 A view of the river section, illustrating the thick alluvial deposits on the river banks and the absence of rock outcrop

(b) Major structural features

Aerial photograph interpretation (API) indicates a NE-striking lineament which may be traced along the river section and across the lower left flank downstream of the proposed centre-line (Drawing 103577-G1-003).

The nature of the lineaments cannot be deduced from the API, and may be associated with dolerite intrusives or fault- or fracture zones or zones of well-developed jointing. Borehole NBH 1205 intersects a minor fault, however, with evidence of some shearing between depths of 36 m and 42,5 m and it may reasonably be assumed that a fault is associated with this lineament.

(c) Geological profile

Significant deposits of alluvium are recognised on the respective river banks, and are also visible within the river channel during periods of low flow. The respective boreholes drilled from the river banks indicate alluvium thicknesses varying between 4 m and 9 m (vertical depths). The boreholes do not reflect the total thickness of the river banks, however, which is in the order of 13 m to 16 m (**Plate 2**).

In situ conditions within the alluvial deposits were investigated by means of the test pits, although only the upper 3 m of the profile could be inspected. On the river bank (NTP01) two horizons of alluvium are recognised; an upper horizon with thickness of 0,7 m comprising slightly moist, light brown, intact but horizontally laminated, loose sand, with roots. This is underlain by slightly moist to dry, brown, shattered, intact to shattered, medium dense clayey silt, with roots. Test pits excavated from the top of the river bank reveal the alluvium to comprise slightly moist, reddish brown or dark grey-brown becoming brown, medium dense or dense, silty sand or clayey to silty sand. Occasional rounded to sub-rounded cobbles and boulders are present, with diameter varying between 60 mm and 300 mm.

Previous boreholes intersected a horizon of partly cemented alluvial gravels and boulders immediately above the bedrock, which was interpreted as representing older alluvial deposits. Current boreholes did not intersect this horizon, however, and the test pits could not be excavated deep enough to verify the presence of this partially-cemented alluvium.

Bedrock within the river section occurs between approximate levels of 447 masl and 454 masl. The boreholes only intersected granite gneiss, with the exception of a minor, 1,8 m thick amphibolite band in NBH 1208. Other narrow pegmatite veins as well as amphibolite-rich veins also occur. The possibility that minor dolerite dykes may be present cannot be discounted, however.

For the most part the granite gneiss underlying the river section comprises unweathered, very hard rock, although upper, weathered horizons are present. Where present, this upper weathered horizon varies in thickness between 3 m and 4 m. The upper horizons typically comprise highly weathered or moderately to highly weathered, hard rock occasionally to soft rock granite gneiss, occasionally grading to slightly weathered or slightly to moderately weathered, hard rock granite gneiss before becoming unweathered granite gneiss at depths below 8,8 m to 9,5 m. An exception is noted in the case of Borehole NBH 1207 where the rock mass beneath the upper, moderately to highly weathered horizon remains slightly weathered, hard rock to very hard rock to a minimum depth of at least 50 m.

The jointing within the unweathered rock mass is generally widely to very widely spaced, and occasionally medium spaced. The upper, weathered horizons typically possess a closer joint spacing, with joints within the moderately to highly weathered horizon described as very closely to closely spaced, occasionally medium spaced, and the joints within the slightly to moderately weathered horizons being very closely to medium spaced. More importantly, zones of very closely spaced joints are encountered within the unweathered rock mass. These zones are generally narrow (100 mm to 300 mm).

Joint infill was generally absent and was restricted to staining of partial staining. The possibility that potential infill materials have been washed away cannot be discounted, but based on the material recoveries any such losses can only be minor. In places staining is prominent, such as via the 60° joints in Borehole NBH 1207, which might represent a sub-vertical joint set, where the prominent staining suggests a possible seepage path. Prominently stained joints were also evident in other places, for example between depths of 26 m and 28 m and 36,5 m and 42,5 m (NBH 1205) where the staining was not restricted to a specific joint set.

A minor fault was recognised between depth of 36 m and 42,5 m in NBH 1205, where some shearing was noted via the 60° joints (possibly representing sub-vertical joints). Although largely recemented and intact, this zone of shearing was open and stained in places and might represent a potential seepage path.

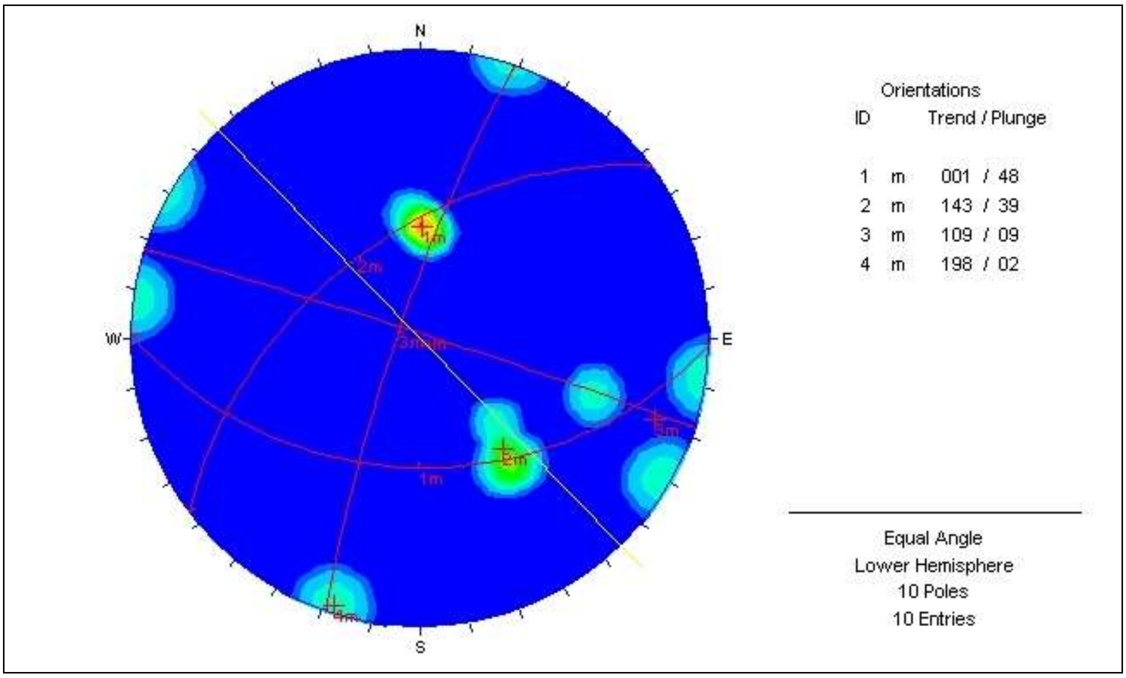

Figure 6.1 Stereographic discontinuity data (river section). The orientation of the centre-line is indicated, and the respective joint sets are defined.

Up to three or four joint sets are generally recognised within the borehole cores. In the absence of oriented cores, no firm conclusions can be made on the attitude of the respective joint sets, however. Sub-vertical and sub-horizontal joint sets may nevertheless be expected. Bedrock outcrop a short distance downstream of the proposed centre-line enabled a limited discontinuity survey to be conducted. The stereoplot of the measured discontinuities is included below (**Figure 6.1**).

Four discontinuity sets are recognized, although none of these respective sets are well developed and continuity is limited.

(d) Groundwater conditions

Water levels were recorded in all the boreholes drilled into the river section. The measured water levels may be assumed to indicate the river level, with differences in water level possibly reflecting different river levels, measured at different times in the course of the investigation programme.

Right flank

(a) Description

The right flank is roughly 2000 m in length. The flank is gently sloping, initially rising from the river section at an approximate gradient of 1:40 to an elevation of 477 masl at approximate Chainage 2200, continuing via very flat slopes and a minor saddle to an elevation of 482 masl at approximate Chainage 3000, and a further minor saddle and flattish slopes to an elevation of 486 masl at approximate Chainage 3500 (Drawing 103577-G1-003).

Initial investigations comprised the drilling of four rotary diamond boreholes, numbered BH’s 1008 and 1009, and BH’s 1016 and 1017. Current investigations comprised the drilling of a further four boreholes (numbered NBH 1209 to NBH 1212) as well as the excavation of six test pits (numbered NTP02 to NTP07).

A general view of the right flank is presented in **Plate 3** below.

Plate 3 A general view of the upper right flank, viewed from the high point at approximate Chainage 3000 (m) looking towards the saddle in the middle distance and the local topographic high at Chainage 2200.

(b) Major structural features

Aerial photograph interpretation indicated a number of minor NE-striking lineaments traversing centre-line on the lower right flank (Drawing 103577-G1-003). In addition, an E-W striking lineament is recognised which traverses the saddle on the mid-flank areas (approximate Chainage 2700). These lineaments might represent fault- or shear zones, or dolerite intrusions which are known to possess a common NE-striking regional trend.

The geophysical survey also indicated an anomaly on the mid-slopes area in the vicinity of Borehole NBH 1211. This borehole intersected dolerite and it might be assumed that this anomaly is therefore associated with a dolerite intrusion. Dolerite was also intersected in BH 1008.

(c) Geological profile

Deposits of alluvium occur in areas adjacent to the river section. The lower flank areas as well as the mid-slope areas to approximate Chainages 2800 are covered by rounded to sub-rounded alluvial gravels and cobbles in a silty sand matrix, representing earlier alluvial deposits (palaeo-alluvium). A degree of reworking is assumed to have occurred and the materials are likely partly colluvial in origin. The upper flank areas, between approximate Chainages 2800 and 3500, and beyond, are covered by a poorly developed horizon of silty sand with minor gravel mainly of colluvial origin (Drawing 103577-G1-003). The respective soil horizons are described in more detail below.

The alluvial deposits adjacent to the river section comprise reddish brown to brown, clayey to silty sand with occasional rounded to sub-rounded cobbles and boulders. The material exposed in test pit NTP02 only represents the upper 3 m of the alluvial profile, however, and the total thickness of river bank and therefore the alluvial horizon is estimated at 13 m to 16 m.

The palaeo-alluvial deposits, now reworked to some degree and including some mixing with colluvial materials, cover the greater part of the right flank, extending from the boundary with the river section at approximate Chainage 1550 to approximate Chainage 2800. This horizon may be described as comprising a matrix of slightly moist to moist, dark grey to brown, loose, silty sand, and rounded to sub-rounded gravels and cobbles (diameter 20 mm to 200 mm) of hard rock quartzite. This upper horizon varies in thickness between 0,35 m and 0,6 m.

The upper flank areas, i.e. beyond approximate Chainage 2800, are covered with shallow soils primarily of colluvial origin. These soils (NTP07) may be described as slightly moist, dark brown, medium dense, intact, silty sand with an estimated 5% to 10% gravel comprising angular to rounded granite and quartz.

Pedogenic material in the form of nodular ferricrete was intersected in two test pits and is assumed to be patchily developed. In test pit NTP03 this horizon was intersected between a depth of 0,5 m and 1,1 m while in test pit NTP06 the ferricrete nodules occurred in an horizon between depths of 0,6 m and 0,85 m, i.e. thickness varied between 0,25 m and 0,6 m. This horizon may be described as moist to slightly moist, grey brown, medium dense to dense to very dense, clayey, silty sand and gravel and nodular ferricrete.

The upper horizons of transported or pedogenic soils are underlain by residual soils derived from the underlying granites. These materials are intersected at depths varying between 0,3 m and 1,1 m. Horizon thickness varies between 0,15 m and 1,3 m. In places the materials can be distinguished from the underlying weathered granitic bedrock but in other areas the material is seen to vary between a residual soil without a recognisable structure, and highly or completely weathered, soft rock granite gneiss. The material may be described as slightly moist, reddish brown to red to grey, loose to dense, silty to sandy gravel to clayey- or silty sand. This horizon is typically quite variable.

Within the test pits, the upper horizon of weathered bedrock was intersected at depths varying between 0,55 m and 0,8 m. Bedrock therefore occurs at shallow depths, but is deeply weathered. Bedrock generally comprises granite or granite gneiss. Dolerite dykes were intersected in two places, but the possibility that other dolerite dykes also traverse the dam footprint cannot be discounted. Dolerite was intersected in Boreholes NBH 1211, between depths of 12,43 m and 15,5 m but could be wider than this, and in BH 1008 between respective depths of 3,98 m and 7,32 m, and between 11,1 m and 14,56 m.

For the most part the rock mass is described as highly or moderately weathered, medium hard rock to hard rock granite gneiss. Only in two instances was unweathered, very hard rock granite gneiss encountered, namely in Boreholes BH 1009 and BH 1017, below respective depths of 25,45 m and 23,95 m. In places, horizons of soft rock also occur. In other areas material losses were noted within the weathered horizons; where these losses are ascribed to horizons of soft rock or very soft rock which have been lost in the drilling process.

Typically, up to three joint sets are recognised within the weathered granite gneiss rock mass, where the overall joint spacing generally varies between very close and medium. These include shallow-dipping joints as well as steeply- dipping and sub-vertical joints. As a rule the joint surfaces are stained. In places joint infill material comprising weathered rock is recorded, but such material is likely to have been lost in many instances and the recovered joint infill material is not likely representative of in situ conditions.

The contacts between the granite gneiss host rock and the intrusive dolerite dykes are all fractured and are not intact.

(d) Groundwater conditions

Groundwater levels were recorded in all the exploratory boreholes, at depths varying between 2,5 m and 10 m. No major anomalies in the level of the water table were evident, although local variation cannot be excluded.

It should further be noted that the proposed centre-line traverses an area of irrigated citrus orchards, and seepage from leaking irrigation pipes may locally influence the groundwater table.

6.1.3 Test results

Soil

(a) Description

Representative samples of the various soil horizons were tested by the DWA Materials Laboratory. The test results have been grouped on the basis of the respective soil horizons and are summarized below. Detailed results are presented in **Appendix F**. Grading and Atterberg limit results are summarised in **Tables 6.1** while shear strength, compaction as well as permeability data are presented in **Table 6.2**.

(b) Grading and Atterberg limits

Table 6.1 Summarised foundation indicator test results (dam footprint)

Hole No	Depth (m)	Material horizon	Soil composition				GM	Atterberg limits		
			Clay %	Silt %	Sand %	Gravel %		LL %	PI	LS %
NTP01	1,5 – 2,5	Alluvium	34	53	13	0	0,04	51,1	23,1	15,0
NPT02	1 - 2	Alluvium	13	27	54	6	0,74	23,0	8,6	4,8
NPT11	1 - 2	Alluvium	16	23	61	0	0,54	21,5	4,4	2,7
NPT05	0,2 – 0,4	Reworked palaeo-alluvium	7	9	39	46	1,88	23,5	7,0	5,7
NTP03	0,5 - 1	Mixed colluvium / pedogenic	20	6	29	45	1,72	49,1	28,4	14,0
NTP09	0,5 – 1,5	Residual soils from granite	3	7	33	57	2,14	20,1	6,1	3,3
NTP07	0,3 – 0,6	Reworked residual soil from granite	6	11	39	45	1,87	26,3	10,0	7,0
NTP04	0,5 – 0,8	Reworked residual soil from granite	7	9	47	37	1,78	24,5	11,6	5,3
NTP08	1 - 2	Very soft rock granite / residual soil	3	7	52	38	1,95	18,1	6,4	2,7

Where:
GM = Grading modulus
LL = Liquid limit
PI = Plasticity Index
WPI = Weighted Plasticity Index

The above results may be summarised as follows;

The recent **alluvial deposits** which occur within the present-day river channel comprise finer-grained sediments varying between clayey silt and clayey to silty sand, with clay content varying between 13% and 34% and sand content varying between 13% and 61%. The grading modulus varies between a very low value of 0,04 and moderate values of 0,74. The Liquid Limit is generally low (21% to 23%) and occasionally high (51%). The weighted Plasticity Index values generally vary between very low and low (4 and 7) but occasionally high values (23) are recorded. Similarly, Linear Shrinkage values are typically low (3% to 5%) but occasionally high (15%).

The **older alluvial deposits** which cover extensive areas on the right flank comprise a mixture of sand and gravels. The clay content is a low 7%. Testing of a single sample reveals the grading modulus to be very high (1,88). The Liquid Limit is low (24%), while the weighted Plasticity Index is very low (3). The Linear Shrinkage is a moderate 6%.

In places, the soil profile contains horizons which are **partly pedogenic** in origin as indicated by the presence of ferricrete nodules. These horizons are expected to be patchily developed, in the form of lenses or pockets, and comprise clayey to sandy gravels. The clay content of the single sample tested was 20% while the sand content was 29% and the gravel content was 45%. The grading modulus is very high (1,72). Values for the Liquid Limit and the Linear Shrinkage are both high (49 and 14, respectively). The weighted Plasticity Index is moderate (12).

The transported soils are underlain by **residual soils** generally derived from the granite gneiss. In places, however, residual soils developed on the dolerite might also be expected. This residual soil horizon is not entirely uniform; in places there is no recognisable structure within the soil while in other areas the horizon partly grades into completely weathered, very soft rock granite/granite gneiss. These horizons predominantly comprise a mixture of sand and gravel in roughly equal amounts (sand fraction varying between 33% and 52%, while the gravel fraction varies between 37% and 57%). The clay fraction is typically low (3% to 7%). As a rule, values of the Grading Modulus are very high (1,78 to 2,14). Liquid Limit values are generally low (18% to 25%) but occasionally moderate (26%). The weighted Plasticity Index values are mainly very low (2 to 5), while the Linear Shrinkage values vary between low and moderate (3% to 7%).

(c) Compaction, shear strength and permeability data

The results of laboratory testing of compaction characteristics as well as shear box strengths and permeabilities are summarised below in **Table 6.2**.

Table 6.2 Summarised compaction, shear strength and permeability test results – disturbed samples (dam footprint)

Hole No	Depth (m)	Material horizon	Compaction		Shear strength		Permeability (cm/sec)
			OMC (%)	MDD kg/m^3	Φ´ (degrees)	c´ (kPa)	
NTP01	1,5 – 2,5	Alluvium	16,5	1605	37,7	112,0	$8,4 \times 10^{-7}$
NPT02	1 - 2	Alluvium	11,6	1887	44,5	61,7	$2,0 \times 10^{-7}$
NPT11	1 - 2	Alluvium	12,0	1850	45,4	50,0	$2,2 \times 10^{-5}$
NPT05	0,2 – 0,4	Reworked palaeo-alluvium	8,7	2000	52,4	16,8	$1,5 \times 10^{-5}$
NTP03	0,5 - 1	Mixed colluvium / pedogenic	13,4	1876			
NTP09	0,5 – 1,5	Residual soils from granite	8,6	2007	47,6	47,7	$2,0 \times 10^{-5}$
NTP07	0,3 – 0,6	Reworked residual soil from granite	10,6	1945			
NTP04	0,5 – 0,8	Reworked residual soil from granite	9,2	1893	46,2	39,2	$6,6 \times 10^{-6}$
NTP08	1 - 2	Very soft rock granite / residual soil	9,8	1942	52,2	20,9	$1,9 \times 10^{-5}$

Where:
OMC = *optimum moisture content*
MDD = *maximum dry density*
Φ´ = *internal angle of friction*
c´ = *cohesion intercept*

The above results may be briefly discussed as follows;

The recent **alluvial deposits,** comprising finer-grained sediments varying between clayey silt and clayey to silty sand, are characterised by moderate to very high values for the optimum moisture content (11,6% to 16,5%) and corresponding moderate to very low values for the maximum dry density (MDD) which vary between 1605 kg/m^3 and 1850 kg/m^3. A limited number of undisturbed samples of alluvium were recovered and the field densities and moisture contents determined (**Table 6.3**). Comparison with the disturbed samples recompacted to optimum moisture content and maximum dry density reveal that the in situ alluvial materials were dry of optimum (moisture contents varying between 4,6% and 6,4%) and at lower densities (1476 kg/m^3 to 1609 kg/m^3). Shear strength parameters yielded very high friction angles varying between 37,7° and 45,4°, and cohesion between 50 kPa and 112 kPa. Results of permeability tests conducted on recompacted samples yielded values between $2,0 \times 10^{-7}$ cm/sec and $2,2 \times 10^{-5}$ cm/sec.

Table 6.3 Summarised compaction results - undisturbed samples (dam footprint)

Hole No	Depth (m)	Material horizon	Compaction	
			OMC (%)	MDD kg/m^3
NTP01	1,5 – 2,5	Alluvium	6,4	1476
NPT02	1 - 2	Alluvium	5,3	1887
NPT11	1 - 2	Alluvium	4,6	1609

The **older alluvial deposits** which comprise a mixture of sand and gravels are characterised by a moderate optimum moisture content (8,7%) and a corresponding high maximum dry density (2000 kg/m^3). Shear strength parameters include a very high friction angle of 52° and cohesion of 16,8 kPa. A permeability of 1,5 x 10^{-5} cm/sec was determined from a recompacted sample.

The patchy**, partly pedogenic** horizons which comprise clayey to sandy gravels (and ferricrete nodules) possess a high optimum moisture content (13,4%) and a moderate maximum dry density of 1876 kg/m^3. No shear strength parameters or permeability values could be determined on the single sample submitted.

The underlying **residual soils** which predominantly comprise a mixture of sand and gravel are characterised by moderate optimum moisture contents (8,6% to 10,6%) and maximum dry densities (MDD) which are generally moderate (1893 kg/m^3 to 1945 kg/m^3) and occasionally high (2007 kg/m^3). The shear strength parameters of these residual soils comprise very high friction angles which vary between 46,2° and 52,2°, and cohesion values which vary between 21 kPa and 48 kPa. Permeabilities determined from recompacted samples vary between 6,6 x 10^{-6} cm/sec and 2,0 x 10^{-5} cm/sec.

Rock

Initially no testing of the rock materials was conducted. Point load strength indices (PLSI) were subsequently determined on borehole cores in order to characterise the rock material strengths and the statistical variability of these strengths. The point load testing was further complimented by Uniaxial Compressive Strength (UCS) testing on a limited number of rock cores. At the same time values for the Elastic Modulus (E_{mod}) as well as the Poisson's ratio and the density of the intact rock material were also determined. Test results are summarised below in **Tables 6.4** and **6.5**, with detailed results included in **Appendix F**.

Table 6.4 Summarised Point Load Strength Index (PLSI) values I_{s50} (dam foundation)

Material description	Minimum (MPa)	Maximum (MPa)	Mean (MPa)	Std dev	n
Granite, granite gneiss; highly weathered	0,04	2,83	0,90	0,717	43
Granite, granite gneiss; slightly weathered	1,01	8,47	3,24	2,998	5
Granite, granite gneiss; unweathered	1,35	11,21	4,86	2,454	20
Dolerite, moderately weathered	1,04	4,78	2,31	1,682	4
Dolerite, slightly weathered			3,02		1

Table 6.5 Summarised rock material properties (dam foundation)

Hole No	Depth (m)	Material horizon	Density g/cm^3	UCS MPa	Emod[1] GPa	Poisson's ratio[2]
BH1201	6,9 – 7,1	Highly weathered granite	2,61	46,4	6,9	0,29
BH1201	18,7 – 19,0	Highly weathered granite	2,64	75,6	18,5	0,19
BH1204	8,4 – 8,6	Highly weathered granite	2,60	95,4	14,3	0,15
BH1205	18,23 – 18,46	Unweathered granite gneiss	2,64	112,2	62,4	0,19
BH1206	14,0 – 14,15	Unweathered granite gneiss	2,73	155,5	65,0	0,26
BH1206	16,05 – 16,25	Unweathered granite gneiss	2,74	149,1	57,6	0,21

Where:
[1] = *Secant modulus at 50% UCS*
[2] = *Secant value at 50% UCS*

With regard to the rock material strengths determined from UCS testing; it should be borne in mind that samples are selected not only to withstand the rigors of sample preparation prior to testing but also to be free of discontinuities or weaknesses which could enable premature failure. In other words the selected samples would therefore fail at the highest applied loads, and the material strengths must therefore be seen as upper bound values.

Rock mass properties are discussed in Section 7.1.1.

6.2 MATERIALS FOR DAM CONSTRUCTION

The various types of construction materials, including hard rock sources for concrete aggregate and rip-rap, other dam embankment materials, backfill materials for the bulk water infrastructure as well as road construction materials are discussed below.

6.2.1 Hard rock for concrete aggregate and rip-rap

Description

No previous investigations of potential hard rock sources were conducted, although the initial engineering geological reconnaissance report (Dooge, 1994) made mention of potential coarse aggregate quarry sites. Brief verification of these potential sites was subsequently conducted by means of a brief drive-over survey. The rock exposures which presumably led to their identification as potential quarry sites were located at relatively high elevations in rugged topography, i.e. in highly visible localities. Furthermore these exposures were located more than 70 km from the proposed Nwamitwa Dam, and further investigations were therefore not considered to be warranted.

It was recognised that locating a usable hard rock quarry site in the dam basin would be unlikely due to the general deep weathering and the absence of rock outcrop. Potential quarry sites located further afield were identified on the basis of visible outcrop during drive-over and walk-over surveys, and are discussed briefly below. Exploratory investigations were subsequently conducted at the closest two potential sites (designated **Quarry Sites A and B**, respectively, see Drawing 103577-G1-001) and these investigation findings are discussed separately. Geophysical investigations were conducted, followed by limited exploratory drilling at the two sites with the aim of confirming the geological conditions at depth, and therefore verifying the potential for quarry development. The rock material properties were further investigated by Point Load Strength testing, and selected cores were subjected to the SABS 1083 suite of tests (SABS, 1994) to assess their suitability for use as coarse aggregate. In addition, limited petrographic analyses were conducted to investigate the mineralogy and petrography.

Potential quarry site 1, comprising a granite koppie which has been intruded by dolerite, was identified along the road to Giyani, at a distance of approximately 50 km for the dam site. This site had been worked previously for dimension stone and the dolerite is described as unweathered, very widely jointed, extremely hard rock dolerite.

Another abandoned quarry site (**potential quarry site 2**) is located near the T-junction between roads R81 and R529 near Giyani. This site is approximately 63 km from the dam site. The geology comprises granite and dolerite, but both lithologies are deeply weathered and would have to be quarried selectively.

Apart from the above potential quarry sites, and the two sites which were drilled, other options for coarse aggregate would be to purchase the required materials from **commercial sources**. Such sources are available in Tzaneen; a distance of approximately 45 km from the dam site. The town of Gravelotte is also located approximately 45 km from the dam site and waste rock dumps from various mines in the vicinity represent potential sources of aggregate. No reconnaissance of these potential sources has been conducted.

Quarry site A

Potential Quarry Site A is located to the north-west of the dam site, approximately 15 km by road, in the hills above the settlement of Babanana (Drawing 103577-G1-001). Access to the site is via the R529 tarred road and a network of existing gravel roads between the various settlements. A general view of the site is shown below (**Plate 4**).

Three boreholes were drilled at this site, numbered NBH 3001 to NBH 3003. Borehole details are included in **Table 4.1**.

Plate 4 A general view of Potential Quarry Site A, illustrating scattered outcrop. A concrete reservoir may be discerned in the distance

(a) Site description

The potential quarry site is located amongst low koppies in a high-lying area above the village of Babanana, on the original farm known as Meadowbank. The village is however some distance from the potential quarry site and is unlikely to be a constraint on quarrying activities. The potential site is bisected by a gravel road and the hills are also traversed by power lines on both sides of this road. It is estimated that these power lines are located more than 500 m from possible quarrying activities, implying that their proximity is not a constraint on quarry development, but this would need to be confirmed. A concrete reservoir is located near the potential quarry site (**Plate 4**). The effects of blasting in relative proximity to this reservoir will have to be assessed.

(b) Geophysical survey

The findings of the geophysical survey are presented in **Appendix H**, and may be briefly summarised as follows :

There were some point anomalies identified in the magnetic field, but no significant trend in the data set. Apparent conductivity values were low, but there was a weakly-defined NE-SW trend in the data. The variable acoustic velocities suggest a greater degree of weathering than the alternative Quarry Site B.

(c) Site geology, and drilling results

The initial walk-over survey revealed scattered outcrop of granite, comprising moderately weathered, medium- to coarse-grained, hard rock to very hard rock. A negative feature in the topography suggests the area might be traversed by a dyke or a fault.

The overlying colluvial materials are poorly developed; in places the bedrock occurs as outcrop and no soil cover is present while in other areas pockets of shallow colluvial soils are present. Where present the colluvial overburden comprises dark grey, silty to clayey sand. A thickness of 0,1 m was recorded in borehole NBH 3001, but locally thicker accumulations are possible.

The underlying bedrock comprises granite gneiss. No intrusive dolerite dykes were intersected by the exploratory boreholes, but may still be present. A narrow amphibolite band (width 600 mm) was intersected in NBH 3001.

The granite gneiss rock mass is characterised by an irregular weathering profile; In places unweathered granite gneiss bedrock occurs at surface (NBH 3003), while in other areas an upper horizon of highly weathered granite gneiss occurs with the underlying unweathered, or slightly weathered granite gneiss occurring at depths between 5,5 m and 6,5 m. Occasional narrow weathered zones (300 mm wide, in NBH 3001) occur within the unweathered rock mass.

The unweathered or slightly weathered granite gneiss comprises extremely hard rock, while the highly weathered horizons vary between soft rock or very soft rock and hard rock.

Joint spacing within the unweathered (or partly slightly weathered) rock mass is generally widely to very widely spaced (600 mm to more than 1000 mm) and occasionally medium spaced. A closer joint spacing is typically associated with the highly weathered horizons; varying between close or very close spacing becoming medium spaced in places (less than 100 mm up to 500 mm).

(d) Groundwater

Water levels were recorded in the exploratory boreholes and are summarised below in **Table 6.6**.

Table 6.6 Water levels – Quarry Site A

BH No	Depth of water table (m)	Approximate level of water table (masl)	Remarks
3001	14,89	617	
3002	?	?	Hole blocked – unable to measure water table
3003	8,13	611	

(e) Test results

Two hand samples were submitted to the Petrographic Laboratory at the Council for Geoscience for detailed petrographic analyses. Summarised results are presented below in **Table 6.7** and detailed results are included in **Appendix F**.

Table 6.7 Summarised mineral assemblages

Mineral	%	Remarks
Alkali feldspar	59 - 63	
quartz	22 – 24	
Plagioclase	8 – 12	
Biotite	2 – 3	Mica – deleterious mineral
Muscovite	1 - 2	Mica – deleterious mineral
Epidote	<1 - 2	
Opaques	<1	
Sphene	<1	

As part of the petrographic analyses, there was confirmation of the presence of undulatory extinction of the quartz grains. The presence of strained quartz is known to be indicative of potential alkali-aggregate reactivity (Gogte, 1973). Gogte concluded that the crystalline rocks such as granites which contain 40% or more of strongly undulatory quartz are highly reactive, while 30% to 35% strained quartz indicates moderately reactivity. The total strained quartz in the two samples was approximately 60% and 48%, respectively, which translates to approximately 13% and 11,6% of the total samples. In terms of undulatory extinction (UE) angles, the one sample indicated undulatory extinction angles which varied between 10° and 46°, with an average of 28°; while the other sample indicated undulatory extinction angles between 27° and 49°, with an average of 34°. It is accepted that the greater the extinction angles, the more strongly reactive are the rocks, and that UE angles smaller than 15° characterise non-reactive rocks (Dolar-Mantuani, 1981). The above results therefore indicate that these granites are potentially susceptible to alkali-aggregate reaction (AAR).

A programme of Point Load Strength Index (PLSI) testing was conducted in order to further characterise the rock material properties. The results are summarised below in **Table 6.8** and included in **Appendix F**.

Table 6.8 Summarised Point Load Strength Index (PLSI) values I_{s50} (Quarry Site A)

Material description	Minimum (MPa)	Maximum (MPa)	Mean (MPa)	Std dev	N (No of tests)
Granite, granite gneiss; highly weathered	3,29	4,35	3,94	0,467	4
Granite, granite gneiss; unweathered	2,99	6,92	4,58	1,293	12

In addition, borehole cores were submitted to the Materials Laboratory of the DWA in order to determine whether the crushed rock would satisfy the standard specifications for coarse aggregate (SANS 1083). Test results are summarised below in **Table 6.9** while detailed results are presented in **Appendix F**.

Table 6.9 Summarised coarse aggregate test data

Property		NBH 3001 (5,4 m – 19,85 m)	NBH 3003 (0 – 20,18 m)	Requirement (SANS 1083)
Grading (mass percentage of material passing nominal sieve sizes in mm)	53	100	100	
	37,5	100	100	
	26,5	100	100	
	19,0	99,4	99,0	
	13,2	91,4	92,8	
	9,5	57,1	62,3	
	6,7	36,2	41,1	
	4,75	26,4	30,0	
Aggregate crushing value (ACV), mass percentage max		18,0	21,1	29
10% FACT, kN, min		229	184	110 (where subject to surface abrasion, or structural elements)
Flakiness Index, max		11,8	10,3	35
Absorption		0,13	0,10	
Loose bulk density (kg/m^3)		1370	1523	
Compacted bulk density (kg/m^3)		1617	1641	

The test results indicate the granite gneiss from Quarry Site A is suitable for the manufacture of coarse aggregate.

Quarry site B

This potential hard rock quarry site, designated Quarry Site B, is located to the south-west of the dam site near Letsitele, approximately 19 km by road from the proposed centre-line, and would be accessed by the main R71 and R529 tarred roads. The location of this potential site is shown on Drawing 103577-G1-001.

The potential quarry site was investigated by the drilling of three boreholes, numbered BH3101 to BH3103. Borehole details are included in **Table 4.1**. A general view of the proposed quarry site is shown below in **Plate 5**.

Plate 5 A view of the granite outcrop at Potential Quarry Site B

(f) Site description

The site comprises a series of koppies where outcrop of massive, slightly to moderately weathered, hard rock granite gneiss is noted (**Plate 5**). The site is located relatively close to the main R71 road, and a chicken farm is located at the foot of the koppie slopes, on the original farm known as Portion 57 California 507. A minor road cutting on the R71 which has been blasted through the unweathered granite gneiss provides an indication of the material at depth; but cognisance should be taken of the constraint on quarrying within a distance of at least 500 m from this road and associated services (telephone lines, etc). Cognisance should also be taken of other developments located in the broader area surrounding this site, including dwellings and a chicken farm.

(g) Geophysical survey results

The detailed results of the geophysical survey are presented in **Appendix H**, and are summarised below.

The magnetic survey reveals a NNW-SSW trend which defines an anomaly and is interpreted as a possible dyke. The apparent conductivity data does not reflect this anomaly. The seismic sections are interpreted as typical profiles reflecting improved rock mass conditions, in terms of less jointing and less weathering, with increasing depth.

(h) Site geology and drilling results

As mentioned above, this potential quarry site comprises a hilly area where extensive bedrock outcrop occurs amongst the series of koppies. From the initial walk-over survey it was noted that the lower slopes of these koppies comprise moderately weathered, brown, medium-grained granite with pegmatite veins. Near the crest of these koppies the outcrop comprises slightly to moderately weathered, grey, medium grained granite. There were indications that minor dolerite dykes might traverse the granite, as well as expectations that occasional deeply weathered zones might be present.

Where rock outcrop occurs the overlying soil cover is obviously absent. In other areas the soil cover which comprises silty sand and gravel is poorly developed, with a thickness of only 150 mm intersected in NBH 3103. Thicker accumulations are expected in the low-lying areas, but no boreholes were drilled in these parts.

The boreholes only intersected granite or granite gneiss bedrock, but the possibility the narrow intrusive bodies of dolerite intersect the granite cannot be excluded. In places there is no discernable alignment of the rock mineralogy and the rock is described as granite, while in other areas the fabric displays a preferred orientation and the rock is described as granite gneiss. Typically the fabric changes between granite and gneiss are random and perhaps repetitive and this is reflected in the description of the rock type as granite gneiss.

The weathering profile comprises an upper weathered horizon, with unweathered to slightly weathered bedrock intersected at depths varying between 4,5 m and 10,7 m. The overlying horizon is weathered to varying degrees; from highly to completely weathered, soft rock to medium hard rock granite to moderately to slightly weathered, very hard rock granite. The boreholes reveal a distinct improvement in the degree of weathering with increasing depth, thus confirming the results of the seismic survey.

Joint spacing within the upper slightly weathered horizons varies between medium to wide (generally 300 mm to 1000 mm) in the case of borehole BH 3103 and generally close (60 mm to 150 mm) in the case of borehole BH 3102. The underlying unweathered granite gneiss is generally medium to widely jointed, or widely to very widely jointed (100 mm up to 2000+ mm). One or two, and occasionally three joint sets are recognised.

(i) Groundwater

Water levels were recorded in the three exploratory boreholes and are summarised below in **Table 6.10**.

Table 6.10 Water levels – Quarry Site B

BH No	Depth of water table (m)	Approximate level of water table (masl)
3101	16,09	567
3102	15,31	574
3103	15,08	574

Two of the boreholes (BH 3102 and BH 3103) recorded instances of total water losses during drilling. If these losses are indicative of the general permeability of the rock mass then it can be assumed that the water levels recorded reflect the water table.

(j) Test results

A programme of Point Load Strength Index (PLSI) testing was conducted in order to further characterise the rock material properties. The results are summarised below in **Table 6.11** and included in **Appendix F**.

Table 6.11 Summarised Point Load Strength Index (PLSI) values Is50 (Quarry Site B)

Material description	Minimum (MPa)	Maximum (MPa)	Mean (MPa)	Std dev	N (No of samples)
Granite, granite gneiss; highly weathered			0,68		1
Granite, granite gneiss; moderately weathered	1,0	4,79	2,53	1,414	5
Granite, granite gneiss; slightly weathered	2,38	5,15	3,81	1,387	3
Granite, granite gneiss; unweathered			4,05		1

In addition, borehole cores were submitted to the Materials Laboratory of the DWA in order to determine whether the crushed rock would satisfy the standard specifications for coarse aggregate (SANS 1083). Test results are summarised below in **Table 6.12** while detailed results are presented in **Appendix F**.

Table 6.12 Summarised coarse aggregate test data

Property		NBH 3101 (1,15 m to 20,6 m)	NBH 3102 (4,5 m to 19,7 m)	Requirement (SANS 1083)
Grading (mass percentage of material passing nominal sieve sizes in mm)	53	100	100	
	37,5	100	100	
	26,5	100	100	
	19,0	99,5	99,0	
	13,2	93,5	91,0	
	9,5	65,4	58,0	
	6,7	45,7	38	
	4,75	34,8	29,0	
Aggregate crushing value (ACV), mass percentage max		22,3	22,5	29
10% FACT, kN, min		138	164	110 (where subject to surface abrasion, or structural elements)
Flakiness Index, max		8,9	7,1	35
Absorption		0,17	0,19	
Loose bulk density (kg/m^3)		1511	1441	
Compacted bulk density (kg/m^3)		1723	1641	

The crushing strength test results indicate the granite gneiss from Quarry Site B is suitable for the manufacture of coarse aggregate.

6.2.2 Embankment materials and fine aggregate

Investigations into sources of embankment materials as well as fine aggregate (sand) were previously conducted by the DWA Materials Laboratory (2006). A decision was made by the Project Team that additional investigations into sources of these materials would not be conducted at this stage.

Salient points from the previous investigation are summarised below. The report by the DWA Materials Laboratory is included as **Appendix J**.

Impervious material

The identified borrow area for the embankment material is located within the dam basin, on the right bank of the Groot Letaba River. The higher-lying areas are underlain by weathered granite and a dolerite intrusion, while the low-lying areas mainly comprise alluvial material on the floodplain.

The fine-grained material yielded the following average values for the Atterberg Limits, a Liquid Limit of 38,9%, a Plasticity Index of 18,6 and a Linear Shrinkage of 9,0%. Standard Proctor compactions indicated optimum moisture contents varying between 15,1% and 23,5% while the maximum dry densities vary between 1592 kg/m^3 and 1810 kg/m^3. An average permeability coefficient of 3,0 x 10^{-8} cm/sec was obtained, and shear testing yielded an average internal friction angle of 24,6°.

The investigations proved an available volume of 952 000 m^3.

Semi-pervious material

A borrow area for semi-pervious material is also located within the dam basin. The coarsely-graded material is described as weathered granite.

Average values for the respective Atterberg Limits included the following: Liquid Limit of 30,9%, Plasticity Index of 14 and Linear Shrinkage of 6,5%. Standard Proctor compactions indicated optimum moisture contents which vary between 10,2% and 12,6%, and maximum dry densities (MDD) between 1931 kg/m^3 and 2060 kg/m^3. Shear testing indicated internal angles of friction which varied between 34,7° and 39,2° (mean 37,0°). Dispersivity tests indicated the material classified as non-dispersive, with the exception of a single sample which indicated a ‘marginally dispersive’ material.

Available volumes of 936 000 m^3 were indicated. The report did conclude, however, that additional reserves also occur on the left bank of the river.

Fine aggregate (sand)

Two potential borrow areas for fine aggregate were identified in two rivers namely the Lervattou River located approximately 15 km from the dam site, and the Merekome River located approximately 20 km from the dam site.

The samples comprise quartzitic river sand. All samples tested complied with the SABS specifications for fine aggregate (SABS 1083:1994).

A total volume of 162 000 m^3 was proved.

If required, additional sources of fine aggregate do occur. Only four samples were recovered for the two areas for testing; it is likely that additional sand reserves occur within the same rivers.

Extensive sand deposits were also noted to occur within the Molototsi River, located approximately 35 km from the dam site. No sampling of this source has been conducted to date. Other sand deposits have been noted along the route of the bulk water pipeline. These deposits were sampled to determine their suitability for use as backfill materials; the results of this testing being presented in the separate report which deals with the findings of the geotechnical investigations conducted for the bulk water infrastructure.

6.3 RELOCATED ROADS

6.3.1 D1292 and R529 Section

A total of ten test pits were excavated at positions along the R529 and D1292 relocated road centreline.

Geological profile

The geological profile encountered along this section of the road essentially comprises the following horizons:

Topsoil/Hillwash

Most of the test pits encountered hillwash or topsoil material with an average thickness of 0,5 m over the entire deviation routes. The material consisted of intact, medium dense, silty sand to clayey silty sand or firm, silty sandy clay material.

The maximum thickness of the horizon measured within the test pits is 2,0 m and the minimum thickness is 0,1 m.

The Pebble Marker

A pebble marker which consisted of medium dense, quartz cobbles and gravel with a matrix of silty sand was encountered in five test pits along the relocated routes. The average thickness of the layer is 0,3 m with a maximum thickness of 0,7 and minimum thickness of 0,2 m.

Reworked residual granite gneiss

Reworked residual material was encountered in four test pits of the deviated route and in one borrow pit (BP01). The material consisted of medium dense, silty clayey sand, silty sand and sandy clay with abundant tightly packed quartz gravel. The average thickness is 0,61 m with a maximum thickness is 1,3 m and minimum thickness is 0,2 m.

Residual granite gneiss

Residual granite gneiss was encountered in nine test pits and consisted of firm, sandy clay or medium dense, silty sand with quartz gravel and cobbles. The average thickness of the material before refusal was encountered is 0,7 m with a maximum thickness of 1,3 m and minimum thickness of 0,2 m.

Groundwater

Seepage was only encountered in one of the test pits (R03). The seepage depth was encountered at 1,7 m. Problems due to ground water seepage are therefore expected in places, especially during and after a very wet rainy season.

Laboratory results

Detailed laboratory results are included in **Appendix F** and are summarised below.

(a) Grading and Atterberg Limits

Summarised results of the foundation indicator testing and determination of the Atterberg Limits are presented below in **Table 6.13**.

Table 6.13 Summarised foundation indicator test results (D1292 & R529)

Hole No	Depth (m)	Material type	Soil composition				GM	Atterberg limits			Activity
			Clay (%)	Silt (%)	Sand, (%)	Gravel (%)		LL (%)	PI (%)	LS (%)	
Hillwash and topsoil											
R02	0 - 1	Mixed hillwash and topsoil	6	7	39	48	2,06	25	9,7	4,7	Low
R02 (water)	0 - 1	Mixed hillwash and topsoil	2	11	39	48	2,07	0	0	0	Low
R31	0,5	Mixture of top soil and pebble marker	2	14	37	47	1,94	28	12,1	6	Low
Reworked residual granite gneiss											
R22	0,4-1,7	Gravelly silty sand	2	6	52	40	1,98	26	10,5	3,7	Low
BP04a	0-1,0	Reworked gneiss and topsoil	4	6	41	50	2,11	21	4	2,7	Low
Residual granite gneiss											
R01	0,7- 1,3	Sandy clay	25	10	39	26	1,39	40	17,5	8,7	Low
R21	1,0-1,7	Gravelly silty sand	10	6	24	60	2,14	34	14,4	9,7	Low
BP01	0-1,0	Residual gneiss	4	6	41	50	2,11	21	4	2,7	Low

Where:
GM = Grading modulus
LL = Liquid limit
PI = Plasticity Index

These results may be summarised as follows:

The **Hillwash and topsoil** consisted of silty sandy clay to sandy clay and have very high grading moduli of 1,94 to 2,07. The Liquid Limit values are very low to moderate with values between 0% to 28%, with moderate Plastic Index values between 9,7% and 12,1%. From these results, in conjunction with the low clay fraction, it may be concluded that the material has a low potential expansiveness according to Van der Merwe, 1964.

The **reworked residual granite gneiss** consists of gravelly silty sand to clayey silty sand and has very high grading modulus values between 1,39 and 2,14. The Liquid Limit values are low to moderate with values between 21% and 40% and very low to moderate Plastic Index (PI) values between 4% and 10,5%. From these results, in conjunction with the very low clay content, it is concluded that the material possesses a low potential of expansiveness according to Van der Merwe, 1964.

The **residual granite gneiss** consisted of silty sand and has very high grading modulus of 1,98 to 1,82. The Liquid Limit values are low to moderate with values between 21% to 26%, while the Plastic Index (PI) values are very low to low (2,7% to 9,7%). From the results, and considering the generally low clay content, it is evident that the materials possess a low potential expansiveness according to Van der Merwe, 1964. Although the clay content in test pit R01 is high (25%), the weighted PI is low (11), which generally suggests low potential expansiveness.

(b) Compaction test results

Table 6.14 Compaction test results (D1292 and R529)

Hole no.	Depth (m)	Material type	OMC (%)	MDD (kg/m^3)	CBR at various densities			TRH 14 Classification
					90 %	93 %	95 %	
Hillwash and topsoil								
R02	0 - 1	Mixed hillwash and topsoil	6,8	2146	47	66	83	G5
R31	0,5	Mixture of top soil and pebble marker	7,8	2112	49	71	91	G10
Reworked residual granite gneiss								
R22	0,4-1,7	Gravelly silty sand	6,0	2113	60	60	60	G6
Residual granite gneiss								
R21	1,0-1,7	Gravelly silty sand	5,8	2173	90	93	95	G10
BP01	1,0-2,0	Residual gneiss	7,0	2095	61	66	69	G5

The test results indicate that the **hillwash and topsoil** have high maximum dry densities and moderate to low optimum moisture content values. The tests generally yielded high CBR values at densities typically specified in the field (93% to 95%). These materials can be classified, according to the TRH 14 guidelines, as a G5 or G10 material (plasticity index >12). It must be noted that the materials only classify as G10 materials on occasion due to the PI values being greater than 12. G5 material is considered suitable for the construction of engineered fills of high stiffness and the use as a sub-base layer for road layer material. G10 material is suitable for the construction of engineered fills of low stiffness and the use as a sub-grade layer for road layer material. It should however be noted that the CBR strengths suggest that a high stiffness will be achievable.

The **reworked residual granite gneiss** horizon has a high maximum dry density and moderate to low optimum moisture content value. The tests generally yielded high CBR values at densities typically specified in the field (93% to 95%). This material can be classified, according to the TRH 14 guidelines, as a G6 material and is suitable for the construction of engineered fills of moderate stiffness and the use as a selected or sub-base layer for road layer material.

The test results indicate that the **residual granite gneiss** has high maximum dry density values and moderate to low optimum moisture content values. The tests generally yielded high CBR values at densities typically specified in the field (93% to 95%). This material can be classified, according to the TRH 14 guidelines, as a G5 (plasticity index <10) or G10 material (plasticity index >12). G5 material is considered suitable for the construction of engineered fills of high stiffness and for use as a sub-base layer for road layer material. G10 material is suitable for the construction of engineered fills of low stiffness and the use as a sub-grade layer in the construction of road pavements.

6.3.2 P43/3

Geological profile

The geological profile along the proposed P43/3 road section is summarised below.

Hillwash

The route is covered by dark brown, loose clayey silty sand, with scattered fine gravels, which extends to depths between 0,2 m and 0,4 m.

Residual granite gneiss

The underlying horizon comprises red brown to orange brown, medium dense to very dense, clayey gravelly sand and contains occasional hard rock granite cobbles and boulders and soft rock granite gneiss. This horizon is generally encountered between 0,2 m and 1,2 m (0,4 m average thickness). Refusal generally occurred in this horizon or on granite gneiss boulders.

Groundwater

Seepage was encountered in test pits TP2A and TP6 at 0,7 m and 1,4 m, respectively. Problems due to ground water seepage are therefore expected in places, especially during and after a very wet rainy season.

Laboratory

Detailed laboratory results are included in **Appendix F** and are summarised below.

(a) Grading and Atterberg Limits

Summarised results of the foundation indicator testing and determination of the Atterberg Limits are presented below in **Table 6.15**.

Table 6.15 Summarised foundation indicator test results (P43/3)

Hole No	Depth (m)	Material type	Soil composition				GM	Atterberg limits			Activity
			Clay (%)	Silt (%)	Sand, (%)	Gravel (%)		LL (%)	PI	LS (%)	
Hillwash											
TP6	0 – 0,4	Hillwash	29	21	49	0	0.71	31,8	18,0	8,0	Low
TP7	0 – 0.4	Hillwash	5	11	70	13	1.41	NP	NP	NP	Low
Residual granite gneiss											
TP2A	0,3 – 1,1	Residual granite	13	13	54	20	1.51	31,6	18,0	8.7	Low
TP3A	0,4 – 0,9	Residual granite	9	10	58	24	1.71	31,6	19,3	6,7	Low
TP6	0,4 – 1,8	Residual granite	18	10	49	23	1.51	34,2	18,9	8,7	Low
TP7	0,4 – 1,8	Residual granite	14	10	55	22	1.57	40,5	18,9	9,3	Low
TP8	0,2 – 0,7	Residual granite	8	6	47	39	1.98	34,4	19,0	7,7	Low
TP10	0.3 – 0,9	Residual granite	8	9	29	55	2.07	23,3	14,0	5,0	Low
TP11	0.2 – 1,1	Residual granite	7	7	41	45	2.01	42,1	19,3	5,7	Low
BPA	0,2-1,0	Residual granite	9	7	41	44	2,00	35	16	8,0	Low
BPB	0,7-1,6	Residual granite	9	11	46	34	1,76	28	11	5,7	Low

Where:
GM = Grading modulus
LL = Liquid limit
PI = Plasticity Index
NP = Not plastic

The **hillwash** material consists of silty clayey sand and has very high to moderate grading moduli of 0,71 to 1,41. Atterberg limit testing was only possible for the sample from test pit TP6. The test yielded moderate Liquid Limit, Plasticity Index and Linear Shrinkage values. From the results above it is concluded that the material has a low potential expansiveness according to Van der Merwe, 1964.

The **residual granite gneiss** consists of clayey gravelly sand and has very high grading modulus values of 0,51 to 2,07. The tests yielded low to moderate Liquid Limit and Linear Shrinkage values, and moderate Plasticity Index values. From these results above it is evident that the material has a low potential expansiveness according to Van der Merwe, 1964.

(b) Compaction Results

Test results are summarised below in **Table 6.16**.

Table 6.16 Compaction test results (P43/3)

Hole no.	Depth (m)	Material type	OMC (%)	MDD (kg/m^3)	CBR at various densities			TRH 14 Classification
					90%	93%	95%	
Residual granite gneiss								
TP2A	0,3 – 1,1	Residual granite gneiss	1,9	2088	1,9	9,7	29	G10
TP3A	0,4 – 0,9	Residual granite gneiss	9,0	2178	24	29	32	G10
TP6	0,4 – 1,8	Residual granite gneiss	8,9	2065	8,9	19,5	33	G10
TP7	0,4 – 1,8	Residual granite gneiss	7,0	2102	24	41	58	G10
TP8	0,2 – 0,7	Residual granite gneiss	6,0	2155	27	42	56	G10
TP10	0.3 – 0,9	Residual granite gneiss	6,5	2190	15,2	25	34	G6
TP11	0.2 – 1,1	Residual granite gneiss	7,6	2116	2,8	2,5	2,3	G10
BPA	0,2-1,0	Residual granite gneiss	6,4	2144	5,7	21	50	G10

The test results indicate that the **residual granite gneiss** has a high maximum dry density and moderate to very low optimum moisture content value. The tests generally yielded high CBR values at densities typically specified in the field (93% to 95%). The Plasticity Index (PI), however, is very high (>12). This material can therefore be classified, according to the TRH 14 guidelines, as a G10 material and is therefore considered suitable for the construction of engineered fills of low stiffness and for use as a sub-grade layer for road construction. The material in TP10 is classified as a G6 material and is suitable for use as sub-base or selected layer in road pavements or engineered fill of moderate stiffness.

6.3.3 Field Testing - DCP (Dynamic Cone Penetration) results

DCP (Dynamic Cone Penetration) tests were conducted in close proximity to selected test pits to evaluate and correlate the consistency of the soil profile. The dynamic cone penetration (DCP) test is conducted by driving a 20 mm diameter, 60° cone into the ground by an 8 kg hammer. The hammer is lifted by hand and dropped a distance of 575 mm and the results are expressed as the penetration rate (PR) in mm per blow. The results obtained from these tests are summarised as follows:

Table 6.17 DCP results

Test no	Total depth (m)	Depth where 5 < PR < 15 (mm/blow)	Depth where 15 < PR < 30 (mm/blow)	Depth where 30< PR < 75 (mm/blow)	Remarks
R529					
DR01	0,96	0,25 – 0,96	0 – 0,25	-	Refusal
DR02	0,40	0 – 0,21 0,31-0,4	0,21-0,31	-	Refusal
DR03	0,80	0-0,4		0,4-0,8	Refusal
DR04	0,81	0-0,4		0,4-0,81	Refusal
DR05	0,80	0-0,35		0,35-0,80	Refusal
DR31	0,27	0-0,23	0,23-0,27		Refusal
DR41	0,80	0,78-0,80		0-0,78	Refusal
DR42	0,80		0-0,23	0,23-0,80	Refusal
DR43	0,54			0-0,54	Refusal
P43/3					
TP 2A	0,86		0,64-0,86	0,0-0,64	Refusal
TP 3A	0,74	0,41-0,74	0,0-0,41	-	Refusal
TP 6	0,81		0,7-0,81	0,0-0,7	Refusal
TP 7	0,81	0,13-0,81	0,0-0,13		Refusal
TP 8	0,16	0-0,16			Refusal
TP10	0,57	0,52-0,57		0,0 – 0,52	Refusal
TP11	0,80	0,30-0,80	0,0 – 0,30		Refusal
D1292					
DR22a	0,10	0-0,1			Refusal
DR22b	0,10	0-0,1			Refusal
DR23	0,80	0-0,12		0,12-0,8	Refusal

Results obtained from the dynamic cone penetrometer (DCP) gives a **rough indication** of the consistency of the soil. The relationship between the penetration rates and material description as shown in **Table 6.18**, is regarded as a guideline for **non-cohesive soils**.

Table 6.18 Indicative dynamic cone penetration correlation with soil consistency (non-cohesive soils)

Penetration rate (mm/ blow)	Soil Consistency
> 75	Very loose
40 – 75	Loose
15 – 30	Medium dense
5 – 15	Dense
< 5	Very dense

The graphs of the DCP results (**Appendix F**) also illustrate the variance of the soil consistency along the route. The consistency varies from less than 5 mm/blows to more than 30 mm/blow. According to **Table 6.18**, the upper ~0,4 m of material is generally described as medium dense, with loose zones. Thereafter, the material is expected to be of medium dense to dense consistency. Generally the material consistency is very variable across the route.

7 GEOTECHNICAL CONSIDERATIONS

7.1 NWAMITWA DAM SITE

7.1.1 Rock mass properties

Because of obvious difficulties with direct measurement of *in situ* rock mass properties, an empirical approach is generally followed. The Hoek-Brown Failure Criterion (Hoek and Brown, 1997) is utilized here.

According to the approach outlined by Hoek and Brown (1997), and summarised in **Appendix I**, rock <u>mass</u> properties can be estimated on the basis of three input parameters, namely the rock <u>material</u> strength (uniaxial compressive strength, UCS, in MPa), the Hoek-Brown constant m_i and the Geological Strength Index (GSI).

As discussed above (Section 6.1.2), the rock material strengths were primarily assessed by means of Point Load Strength Index (PLSI) tests, supplemented by limited Uniaxial Compressive Strength (UCS) tests conducted on the better quality materials which could stand up to the rigors of this destructive testing. Prior to the availability of the rock material strength test data reliance was placed on *estimated* rock material strengths for the initial assessment of the rock mass properties. Although UCS values for various weathering grades of granite from the Kleinplaas Dam are available in the literature (Brink, 1981), estimated material strengths were allocated to the various weathering grades on the basis of the described rock hardness (**Table 7.1**, after Core Logging Committee, 1976).

These estimated material strengths were subsequently checked against values determined by limited laboratory UCS testing, while further statistical evaluation of the strength variability was possible from the Point Load Strength testing. As discussed above in Section 6.1.2 the laboratory-determined UCS results must be considered to represent upper-bound values.

Table 7.1 Rock material strengths (estimated values as well as confirmed results)

Lithology	Weathering grade	Hardness (as described)	Estimated UCS value (MPa)	UCS (lab) MPa	PLSI (I_{S50}) MPa
Granite gneiss	Highly weathered	Soft rock Medium hard rock	5 20	72,5	0,90
	Moderately weathered	Soft rock Medium hard rock Hard rock	5 25 50		
	Slightly weathered	Soft rock Medium hard rock Hard rock	5 25 50		3,25
	Unweathered	Medium hard rock Hard rock	70 150	140	4,86
Dolerite	Highly weathered	Soft rock Medium hard rock	5 20		

The input parameter comprising the Hoek-Brown constant m_i is usually determined from laboratory triaxial tests. In the absence of test data, however, estimated values provided by Hoek and Brown (1997) are considered adequate for preliminary design purposes. The m_i constant for granite and gneiss is given as 33, while an estimated m_i value for dolerite / dolerite is given by Hoek and Brown (1997) as 19.

Values for the Geological Strength Index (GSI), which equate to Rock Mass Rating of Bieniawski (1976), can be determined from the borehole cores, using guidelines set out by Hoek and Brown (1997).

With regard to the GSI and the RMR; Hoek and Brown (1997) have stated that, for better quality rock masses the value of the GSI can be estimated directly from Bieniawski's 1976 Rock Mass Rating (RMR) – with the Groundwater rating set to 10 i.e. "Completely Dry". For dam foundations such as considered in this report, this assumption is not considered realistic or representative of actual circumstances. The approach has therefore been followed where the Groundwater parameter equating to "Severe Water Problems" (see **Appendix I**) is selected, following a "worst case scenario", with an ascribed rating of 0. This approach was also followed by Meintjes, *et al.* (1997). The argument could likely also be made that the parameter "Water under Moderate Pressure" is more appropriate, with a rating of 4, but a more conservative position is taken here, namely that of the 'worst case scenario'.

Note that the above assumption has implications on the calculated RMR values (i.e. whether or not an additional rating of ‘10’ is accrued or not) and this follows through to the selection of appropriate founding criteria.

Outputs from determination of the rock mass strength parameters include the rock mass tensile strength (in MPa), rock mass compressive strength (in MPa), rock mass shear strength parameters comprising cohesive strength in MPa as well as friction angle, and the deformation modulus of the rock mass (in GPa).

Strength and deformation parameters for each of the defined horizons have been calculated for the respective boreholes. The appropriate rock mass deformation modulus (E_{mod}) values for each horizon are summarized below in **Table 7.2**. Note that the E_{mod} values for assumed Groundwater conditions of “Dry” as well as the “Severe Water Problems” are presented. In addition, a *range* of appropriate deformation modulus values are given in each instance, rather than a single value, to better reflect the nature of geological materials which, by definition, do not lend themselves to such precision (Hoek, 1999). This range of values is determined by bracketing the calculated RMR value by a nominal ± 5.

It is emphasized that the material strength (σ_i) input value as reflected above in **Table 7.1** is considered an overly conservative assessment which has a follow-through effect on calculation of the rock mass parameters.

Table 7.2 RMR$_{76}$ and calculated E_{mod} values for the various rock horizons

Borehole No	Approximate Chainage (m)	Horizon depths[1] (m)	Horizon description[2]	RMR values[3]	E_{mod} (GPa)
Right flank					
1014	380	0 - 3,0	soil	0	N/A
		3 - 6,44	mod w, cj, SR-MHR gr gn	36	1,5 – 2,7
		6,44 - 13,1	mod w, c-mj, MHR-HR gr gn	59	7,5 – 13
		13,1 - 20,65	sl w, mj, HR-VHR gr gn	61	14 – 22
1015	615	0 - 1,75	soil	0	N/A
		1,75 - 9,1	mod w, vc-cj, MHR-HR gr gn	27	1,2 – 2
		9,1 - 21,05	mod w, vc-cj, MHR-HR gr gn	51	4,7 – 8,4
1201	760	0 - 0,3	soil	0	N/A
		0,3 - 4	cw, SR gr	14	< 0,5
		4 - 10,45	hw, MHR gr gn	37	1,6 – 2,8
		10,45 - 21,15	m-hw, HR-VHR gr gn	47	5,5 – 9,8
1013	870	0 - 0,4	soil	0	N/A
		0,4 - 1,4	sw, MHR - HR gr gn	22	0,9 – 1,6

Borehole No	Approximate Chainage (m)	Horizon depths[1] (m)	Horizon description[2]	RMR values[3]	E_{mod} (GPa)
		1,4 - 15,12	sw, HR-VHR gr gn	66	16 – 29
1012	1015	0 - 0,8	soil	0	N/A
		0,8 - 4,7	mw, MHR-HR gr gn	29	1,3 – 2,4
		4,7 - 20,3	sw, HR gr gn	64	12 – 21
		20,3 - 22,12	sw, MHR, gr gn	35	1,6 – 2,8
		22,12 - 27,6	sw, HR gr gn	61	10 – 18
1213	1220	0 – 9,1	soil	0	N/A
		9,1 – 20,80	hw, c – cj, MHR – HR gr gn	18	0,5 – 0,8
		20,80 – 29,85	unw, w – vwj, EHR gr gn	82	42 – 55
1011	1250	0 - 4,7	soil	0	N/A
		4,7 - 8,5	soil, slightly cemented	0	N/A
		8,5 - 12,94	alluvium, sl cement	0	N/A
		12,4 - 16,88	mw, vcj, MHR - SR gr gn	26	0,9 – 1,5
		16,88 - 19,55	sw, wj, VHR gr gn	69	22 – 40
1010	1360	0 - 6,4	overburden	0	N/A
		6,4 - 9,03	colluvium	0	N/A
		9,03 - 11,60	alluvium	0	N/A
		11,6 - 12,2	mw, cj, HR, gr gn	23	1,1 – 2
		12,2 - 21,2	unw, m-wj, VHR gr gn	69	22 – 40
River section					
1205	1420	0 - 6,1	alluvium	0	N/A
		6,1-9,3	hw, vc - cj, MHR-SR gr gn	15	0,5 – 0,8
		9,3-11	s-mw, vc - mj, HR, gr	37	2,5 – 4,5
		11-50,1	unw, w-vwj, VHR, gr gn	69	22 – 40
1206	1420	0-7,3	alluvium	0	N/A
		7,75-10,63	m-hw, vc-mj, HR-SR gr gn	21	0,6 – 1,1
		10,63-50,42	uw, w-vwj, VHR gr gn	69	22 – 40
1207	1480	0-4,2	alluvium	0	N/A
		4,2-8,8	m,-hw, vc-cj, MHR-HR gr gn	22	0,9 – 1,6
		8,8-19,5	sw, c-mj, HR - VHR g	42	4,2 – 7,3
		19,5-50,1	sw, mj, HR-VHR gr	47	5,5 – 9,7
1208	1480	0-10,2	alluvium r	0	N/A
		10,2-26,9	uw, mj, VHR gr gn	55	10 – 18
		26,9-28,7	uw, m-wj, HR amph	54	7,9 – 14
		28,7-50,5	uw, m-wj, VHR gr gn	59	13 – 22
Right flank					
1009	1550	0-10,7	overburden	0	N/A
		10,7 - 13,66	cemented deposits	0	N/A
		13,66-19	mw, vc-cj, MHR gr gn	27	1 – 1,8
		19-25,45	sw, cj, VHR, gr gn	50	7,5 – 13,3
		25,45-35,05	unw, c-mj, VHR, gr gn	69	22 – 40
1209	1610	0-2,75	alluvium	0	N/A
		2,75-25,15	hw, vc-mj, MHR-HR gr gn	21	0,8 – 1,4

Borehole No	Approximate Chainage (m)	Horizon depths[1] (m)	Horizon description[2]	RMR values[3]	E_{mod} (GPa)
1210	1800	0-2,5	colluvium	0	N/A
		2,5-20,50	hw, vc-mj, MHR-HR gr gn	22	0,8 – 1,5
1016	1890	0-4,85	overburden	0	N/A
		4,85-14,66	mw, vcj, SR-MHR gr gn	15	0,5 – 0,8
		14,66-24,35	mw, vcj, MHR-HR gr gn	27	1,2 – 2,1
		24,35-30,0	m-sw, cj, MHR-HR gn	40	2,5 – 4,5
1211	2030	0-2,65	colluvium	0	N/A
		2,65-12,43	hw, vc-cj, HR-SR gr gn	21	0,6 – 1,0
		12,43-15,5	hw, vc-mj, MHR dolerite	16	0,5 – 0,8
1017	2170	0-2,4	overburden	0	N/A
		2,4-3,3	cw, VSR gr gn	8	< 0,5
		3,3-4,55	mw, vcj, HR gr gn	16	0,8 – 1,3
		4,55-6,95	cw, assume VSR / residual	8	< 0,5
		6,95-14,58	hw, vcj, SR-MHR gr gn	21	0,6 – 1
		14,58-17,35	mw, vcj, MHR gr gn	22	0,8 – 1,3
		17,35-23,95	mw, cj, MHR-HR gr gn	40	2,5 – 4,4
		23,95-26,30	unw, wj, VHR gn	77	35 – 63
1212	2280	0-2,3	colluvium / reworked alluvium	0	N/A
		2,3-4,83	h-cw, vcj, HR-VSR gn	14	< 0,5
		4,83-15,3	hw, c-vcj, SR-MHR gr gn	21	0,6 – 1
1008	2450	0-3,98	overburden	0	N/A
		3,98-7,32	hw, vcj, SR dolerite	12	< 0,5
		7,32-11,1	mw, vcj, MHR gr gn	22	0,7 – 1,2
		11,1-14,56	hw, vcj, MHR dolerite	30	1 – 1,9
		14,56-24,13	mw, vcj, MHR gr gn	35	1,4 – 2,5

Where:
1 = measured along the borehole
2 = summarised from detailed borehole logs (Appendix B)
3 = after Bieniawski (1976),
And cw = completely weathered, hw = highly weathered, mw = moderately weathered, sw = slightly weathered, uw = unweathered, vcj = very closely jointed, cj = closely jointed, mj = medium jointed, wj = widely jointed, vwj = very widely jointed, VSR = very soft rock, SR = soft rock, MHR = medium hard rock, HR = hard rock and EHR = extremely hard rock.

7.1.2 Founding criteria and founding levels

At the time of writing, no firm decisions had been made regarding the type of structure best suited to the prevailing founding conditions. Logic, however, suggests that a central, mass concrete structure incorporating a central spillway would be favoured, while the flanks would likely comprise an embankment. It follows that the various structure types are associated with the requirements of certain minimum values of foundation Deformation Moduli (E_{mass}). A concrete gravity structure would typically require a minimum Deformation Modulus (E_{mass}) of 10 GPa, while for embankment flanks a minimum Deformation Modulus (E_{mass}) of 1 GPa is typically required for rockfill and

0,2 GPa for earthfill. A clay core would typically require a minimum Deformation Modulus (E_{mass}) of 2 GPa.

Knowledge of the required Deformation Modulus allows back-calculation of the Rock Mass Rating (RMR), using the approach described by Hoek and Brown (1997), and therefore the input parameters from which the RMR is determined. The RMR defined by Bieniawski (1976) is used here, although it is also possible to incorporate newer versions published by Bieniawski.

From this back-calculation, and using the Rock Mass Rating (RMR) or Geological Strength Index (GSI) values recorded in the various boreholes (see **Table 7.2**), empirical *minimum* founding criteria can be defined for the envisaged dam components. These criteria are listed below in **Table 7.3**.

Table 7.3 Minimum founding criteria

Foundation criteria	Envisaged dam components		
	Mass concrete, gravity	**Core**	**Rockfill shells**
Deformation Modulus (E_{mass})	10 GPa	2 GPa	1 GPa
RMR or GSI value	50	30	20
Degree of weathering	Moderately weathered to unweathered	Moderately to slightly weathered	Highly weathered to moderately weathered
Rock material strength	100 MPa	50 MPa	25 MPa
RQD (%)	> 80	> 40	> 20
Joint spacing (mm)	Medium spacing (> 200 mm)	Closely spaced (> 60 mm)	Very closely to closely spaced (from < 60 mm)
Joint condition	Slightly rough surfaces, separation < 1 mm, medium hard joint wall rock	Open, continuous joints, or infill material < 5 mm in thickness, or slickensided surfaces	Continuous joints, or open joints > 5 mm, or soft infill material > 5 mm thick

The various input parameters in Bieniawski's rock mass classification methodology should not be considered in isolation but should rather be seen as being inter-related. Poorer joint conditions might be balanced by a wider joint spacing, for example, with a similar end result of an overall RMR value which still complies with the stated minimum guidelines.

The above founding criteria have been developed as *minimum* guidelines. In instances where individual values of the deformation modulus E_{mod} are slightly less than the required 10 GPa, it is assumed that the rock mass properties will be improved as a result of a consolidation grouting programme.

The minimum founding criteria thus developed could then be applied to the drilling results; the resulting founding depths are summarised below in **Table 7.4**. These founding depths should be considered as a guideline only. The availability of additional data will likely result in further optimisation of these expected founding levels, and the importance of final verification during construction is emphasized.

Table 7.4 Summarised excavation depths

	Mass concrete, gravity structure	Core trench	Rockfill shells
Left flank, upper	Var 1 m to 13+ m	Est. 0,5 m to 4 m	Nominal 0,5 m to 1 m
Left flank, lower	Var. 12 m to 17 m	5 m to 12 m	Nominal 1 m to 3 m
River section	8 m to 9,5 m	-	-
Right flank	-	Est 4 m	Nominal 0,5 m to 1 m

7.1.3 Water table and foundation permeability

As a rule the water table was measured in the completed boreholes. These measured water levels are summarised below in **Table 7.5**.

Table 7.5: Water table elevations

Borehole number	Collar elevation (masl)	Water table depth* (m)	Reduced level of water table (masl)
BH 1014	479,75	4	475,75
BH 1015	477,2	5,3	471,9
NBH 1201	475,5	6,38	469,1
BH 1013	474,68	3,8	470,88
BH 1012	470,76	4	466,76
NBH 1213	463	6,85	456
BH 1011	463,36	13	450,36
BH 1010	462,88	11,2	451,68
NBH 1205	452,3	3,16	449,1
NBH 1206	454,5	3,18	451,3
NBH 1207	458	3,71	454,3
NBH 1208	456	7,14	448,9
BH 1009	463,9	10,1	453,8
NBH 1209	463	4,23	458,8
NBH 1210	467,5	2,6	465
BH 1016	470,83	9	461,83
NBH 1211	474	3,79	470

Borehole number	Collar elevation (masl)	Water table depth* (m)	Reduced level of water table (masl)
BH 1017	477,4	5,8	471,6
NBH 1212	476	7,81	468
BH 1008	475,61	8,2	467,41

Where:
* = *as measured along borehole length.*

The measured water levels are indicated on the geological longitudinal section (Drawing 103577-G1-004). In general the water table occurs at relatively shallow depths, varying between 4 m and 10 m and is seen to mirror the gentle topography of the site. None of the boreholes picked up sudden fluctuations in the level of the water table – which might indicate highly pervious zones. The possibility that such zones do occur and have therefore been missed by the widely spaced boreholes cannot be excluded, however.

Several of the test pits apparently intersected the water table at relatively shallow depths (test pits NTP07 and NTP08). While the recorded seepage might represent a shallow water table there is also a possibility that these seepages are linked to leaking irrigation pipes.

In addition to measurement of the water table, water pressure or water acceptance or packer (Lugeon) tests were conducted in many of the exploratory boreholes. With the exception of three of the current boreholes (NBH 1202 to NBH 1204) Lugeon tests were conducted in all of these boreholes. Only two of the original boreholes (BH 1013 and BH 1014) were apparently water pressure tested. For at least the current boreholes, the absence of Lugeon test data may be considered to indicate generally poor rock mass conditions on account of the inability of the packers to seal the bedrock. Detailed results are presented in **Appendix E**, while summarised results are listed below (**Table 7.7**), where a five-fold classification of the degree of permeability is followed, based on the determined Lugeon value (**Table 7.6**).

Table 7.6 Classification of degree of permeability

Degree of permeability	Lugeon range	Sensitivity of Lugeon value (after Houlsby, 1990)
Impervious	0 – 5 Lugeons	1 Lugeon
Slightly pervious	5 – 10 Lugeons	2 Lugeons
Pervious	10 – 15 Lugeons	5 Lugeons
Highly pervious	15 – 50 Lugeons	10 Lugeons
Extremely pervious	50 – 100 Lugeons	30 Lugeons

Table 7.7 Summarised Lugeon test results

Borehole No	Remarks
Right flank	
BH 1014	Not possible to test uppermost 13,70 m Impervious to 19,70 m
NBH 1201	No tests to 10 m Impervious 11 m to 21 m
BH 1013	Not possible to test uppermost 6 m Impervious between 6 m and 15 m
NBH 1213	No tests because of packer jamming
River section	
NBH 1205	No tests to 12 m Impervious to 42 m
NBH 1206	No tests to 12,5 m Impervious to 15,5 m Pervious zone 15 m to 18 m Impervious 18 m to 50 m
NBH 1207	No tests to 5 m Highly pervious to extremely pervious 5 m to 12 m Impervious 12 m to 49 m
NBH 1208	No tests to 11,7 m Impervious to 29 m No tests below 29 m (packer sticking)
Right flank	
NBH 1209	No tests to 5,7 m Highly pervious 4 – 7 m Impervious between 7 m and 21,5 m Highly pervious 22 m to 25 m
NBH 1210	No tests to 6,45 m Slightly pervious 6,45 m to 9,45 m Impervious to 16,8 m Pervious 17,5 m to 20,5 m
NBH 1211	No tests to 6,4 m Impervious to 12,43 m Invalid test 13,5 m to 15,5 m
NBH 1212	No test to 6,4 m Slightly pervious 6,4 m to 9,4 m Impervious t o 15,3 m

The summarized water acceptance (Lugeon) results indicate the founding rock mass is generally impervious. Only in two instances (NBH 1207 and NBH 1209) were highly pervious or extremely pervious horizons identified.

In borehole NBH 1207 the highly to extremely pervious zone occurs within the upper 5 m to 12 m. While it is possible that the high water acceptances might be linked to a poor sealing of the packer, it might be noted that the rock mass is closely to very closely jointed in this zone and that prominent staining via certain joints is also associated with a clay film which might suggest a seepage zone. Material losses were also recorded, however, and these losses might also indicate a potential seepage zone.

In borehole NBH 1209 highly pervious horizons are located between depths of 4 m and 7 m, and 22 m and 25 m, respectively. Borehole logging revealed the presence of horizons of soft rock associated with both pervious zones. While it is possible that these soft rock horizons might indicate pervious zones, other soft rock horizons also occur which are not associated with higher water acceptances.

Although the founding rock mass might therefore be assumed to be relatively impervious the possibility that pervious zones might traverse the founding rock mass cannot be excluded. A programme of curtain grouting is therefore recommended along the dam centre-line, where the aim would be the intersection of any potential pervious zones which might occur. The optimum orientation of the grout holes will only be confirmed during dam construction when the attitude of the main discontinuities can be determined and the optimum hole orientation selected to maximise intersection of the main joint sets.

A programme of low pressure consolidation grouting of the dam footprint is further recommended as a means of densifying or improving the upper, weathered horizon of the founding rock mass.

7.1.4 Slope stability

No specific slope stability investigations have been conducted in the vicinity of the envisaged dam basin. Some general comments are pertinent, however. The natural slopes which would define the dam basin are typically only gently sloping, and the likelihood of catastrophic slope failure occurring to the extent of endangering the dam structure is considered near-impossible and therefore of no real concern.

Consideration would also have to be given to the stability of the temporary slopes required during construction. As detailed site layouts are not known at this stage, it follows that delineation of potentially unstable temporary slopes is premature. Typical areas of concern during construction would include the following:

- Excavations in overburden and residual soils particularly if oversteep and if saturated,
- Excavated slopes in soft rock horizons where seepage is noted,
- Excavations in the rock mass where locally-unfavourable jointing is present and slopes are either oversteep or allow ‘daylighting’ of the discontinuities.

7.2 RELOCATED ROADS

7.2.1 D1292 / R529

- The general profile along the route comprises top soil and hillwash underlain in some areas by a pebble marker horizon, which is underlain by reworked and residual granite gneiss.
- Seepage was only encountered in one of the test pits (R03). Seepage was encountered at 1,7 m depth. Problems due to ground water seepage are therefore expected in places, especially during and after a very wet rainy season.
- The **hillwash** and **topsoil** material classifies as a G5 (plasticity index <12) or G10 material (plasticity index >12), according to TRH 14 guidelines. G5 material is considered suitable for the construction of engineered fills of high stiffness and the use as a sub-base layer in road pavements. G10 material is suitable for the construction of engineered fills of low stiffness and the use as a sub-grade layer in road pavements. The **reworked residual granite gneiss** horizon can be classified as a G6 material, according to the TRH 14 guidelines, and is suitable for the construction of engineered fills of moderate stiffness and the use as a selected or sub-base layer in road pavement layers. The **residual granite gneiss** can be classified as a G4 (plasticity index <6) or G10 material (plasticity index >12). G4 material is considered suitable for the construction of engineered fills of high stiffness and the use as a base layer in road pavements. G10 material is suitable for the construction of engineered fills of low stiffness and the use as a sub-grade layer in road pavement layers. It must be noted, however, that if the material is classified as G10 purely on the basis of the PI values, a high stiffness fill will still be achievable.

7.2.2 P43/3

- The general profile along the road centre-line comprises loose to medium dense, silty clayey sand (hillwash) underlain by medium dense to dense clayey gravelly sand (residual granite). Refusal generally occurred in the residual granite gneiss due to scattered cobbles and boulders and soft rock sections.
- Seepage was encountered in test pit TP2A and TP6 at 0,7 m and 1,4 m respectively. Problems due to ground water seepage are therefore expected in places, especially during and after a very wet rainy season.
- The **residual granite gneiss** material classifies as a G10 (due to the high plasticity index, PI>12) material, and is suitable for the use in the construction of

engineered fills of low stiffness and the use as a sub-grade layer in road pavements. The material in TP10 is classified as a G6 material and is suitable for use as sub-base or selected layer in road pavements or engineered fill of moderate stiffness.

7.2.3 Borrow pit sources

Possible borrow pit sources will be in the area of test pit BP01 (coordinates X 55742; Y2625351); see drawing 103577-G1-06.The residual material classifies as a G5 material.

8 REFERENCES

Bieniawski, Z T. 1976. *Rock Mass Classification Systems in Rock Engineering. Proc. Symp.* "Exploration for Rock Engineering". Johannesburg

Brink, A B A. 1981. *Engineering Geology of Southern Africa. Volume 2. Rocks of 2000 to 300 million years in age.* Building Publications, Pretoria.

Core Logging Committee of the South African Section of the Association of Engineering Geologists. 1976. *A Guide to Core Logging for Rock Engineering.* Proc. Symp. "Exploration for Rock Engineering". Johannesburg. November 1976.

Council for Geoscience. 2008. *Stage 1 Probabilistic Seismic Hazard Assessment for the site of Nwamitwa Dam, Limpopo Province, South Africa.* Council for Geoscience report no 2008-0173.

Dooge, N. 1997. Letaba Water Supply Scheme: *Great Letaba River: Nwamitwa Dam Site: Letaba District.* First Engineering Geological Feasibility Report. Unpublished CGS report no 1997-0009.

Dooge, N. 1994. Letaba Water Supply Scheme*: Great Letaba River: Janetsi Weir Site: Letaba District.* First Engineering Geological Reconnaissance Report. Unpublished CGS report no 1994-0081.

DWAF, Materials Laboratory. 2006. *Letaba Waterhulpbron Ontwikkeling. Uitvoerbaarheidsondersoek na die Beskikbaarheid van Konstruksiemateriaal vir die Voorgestelde Janetsidam.* Report No B801/41/IJ01.

DWAF. 2010. *Groot Letaba River Development Project (GLeWaP). Bulk Water Infrastructure: Volume 8 - Annexure 1: Appendices.* Undertaken by Aurecon for Options Analysis. DWA Report No P 02/B810/00/1110/5.

Gogte, B S. 1973. *An evaluation of some common Indian rocks with special reference to alkali-aggregate reaction*. Eng. Geol. 7: 135 – 153.

Hoek, E. 1999. Rock Engineering – course notes. From website http://www.rocscience.com.

Hoek, E and Brown, E T. 1997. Practical Estimates of Rock Mass Strength. Int. J. Rock Mech. Min. Sci. Vol 34, No 8, pp 1165 – 1186.

Houlsby, A C. 1990. *Construction and Design of Cement Grouting. A Guide to Grouting in Rock Foundations.* Wiley Interscience.

Houlsby, A C. 1976. *Routine Interpretation of the Lugeon Water-Test. Q. Jnl. Engng. Geol. Vol. 9, pp 303 – 313.*

Jennings, J E J, Brink, A B A and Williams, A A B. 1973. *Revised Guide to Soil Profiling for Civil Engineering Purposes in South Africa.* Trans. S. A. Ins t. Civ. Eng. Vol. 15, No 1. January 1973.

Kijko, A, Graham, G, Bejaichund, M, Roblin, D L and Brandt, M B C. 2003. *Probabilistic Seismic Hazard Maps for South Africa.* Council for Geoscience.

Meintjes, H A C, De Graaf, P J H and Van der Vlugt, R. 1997. *Assessment of In-situ Modulus of Deformation for Dam Foundations using Rock Mass Rating.* Proc. Conf. "Geology for Engineering, Urban Planning and the Environment". Midrand. November 1997.

National Institute for Transport and Road Research. (1985). *Technical Recommendations for Highways. Guidelines for Road Construction Materials.* TRH14. Council for Scientific and Industrial Research, Pretoria.

Partridge, T C. 1975. *Some Geomorphic Factors influencing the Formation and Engineering Properties of Soil Material in South Africa.* Proc. 6th Regional Conf. Africa, SMFE, Durban.

Robb, L J, Brandl, G, Anhaeusser, C R and Poujol, M. 2006. *Archean granitoid intrusions. In: Johnson, M.R., Anhaeusser, C.R. and Thomas, R.J. (Eds).* The Geology of South Africa. Geological Society of South Africa, Johannesburg / Council for Geoscience, Pretoria.

South African Bureau of Standards (SABS). 1990. Standardized Specification for Civil Engineering Construction – D:Earthworks (SABS 1200-D-1990)

South African Bureau of Standards (SABS). 1994. *Specification: Aggregates from Natural Sources – Aggregates for Concrete*. (SABS 1083:1994).

Van der Merwe, D H. 1964. *The Prediction of Heave from the Plasticity Index and Percentage Clay Fraction of Soils.* The Civil Engineer in South Africa. June 1964.

Weinert, H H. 1980. *The Natural Road Construction Materials of Southern Africa.* Academica, Pretoria, Cape Town.