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MOKOLO AND CROCODILE RIVER (WEST) WATER AUGMENTATION PROJECT (MCWAP) FEASIBILITY STUDY: TECHNICAL MODULE

Project No. WP9528



SUPPORTING REPORT NO. 8B DETAIL GEOTECHNICAL INVESTIGATIONS

PHASE 2



REPORT NO	DESCRIPTION	REPORT NAME			
	FEASIBILITY STAGE				
P RSA A000/00/8109	Main Report	MCWAP FEASIBILITY STUDY TECHNICAL MODULE SUMMARY REPORT			
P RSA A000/00/8409	Supporting Report 8A	GEOTECHNICAL INVESTIGATIONS PHASE 1			
P RSA A000/00/8709	Supporting Report 8B	GEOTECHNICAL INVESTIGATIONS PHASE 2			
P RSA A000/008509	Supporting Report 9	TOPOGRAPHICAL SURVEYS			
P RSA A000/00/8609	Supporting Report 10	REQUIREMENTS FOR THE SUSTAINABLE DELIVERY OF WATER			
P RSA A000/00/8209	Supporting Report 11	PHASE 1 FEASIBILITY STAGE			
P RSA A000/00/8309	Supporting Report 12	rt 12 PHASE 2 FEASIBILITY STAGE			
PRE-FEASIBILITY STAGE					
P RSA A000/00/8809	Supporting Report 1	WATER REQUIREMENTS			
P RSA A000/00/8909	Supporting Report 2	WATER RESOURCES			
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P RSA A000/00/9109	Supporting Report 4	DAMS, ABSTRACTION WEIRS AND RIVER WORKS			
P RSA A000/00/9209	Supporting Report 5	MOKOLO RIVER DEVELOPMENT OPTIONS			
P RSA A000/00/9309	Supporting Report 6	WATER TRANSFER SCHEME OPTIONS			
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The Mokolo (Mogol) River catchment is part of the Limpopo Water Management Area (WMA). The Mokolo River originates close to Modimolle (Nylstroom) and then drains to the north into the Limpopo River. The Mokolo Dam (formerly known as the Hans Strijdom Dam) is the largest dam in the catchment. The dam was constructed in the late 1970s and completed in July 1980, to supply water to Mathimba Power Station, Grootegeluk Mine, Lephalale (Ellisras) Municipality and for irrigation downstream of the dam. Based on the water infrastructure, the current water availability and water use allows only limited spare yield existing for future allocations for the anticipated surge in economic development in the area.

There are a number of planned and anticipated consequential developments in the Lephalale area associated with the rich coal reserves in the Waterberg coal field for which additional water will be required. These developments include inter alia the development of further power stations by Eskom, the potential development of coal to liquid fuel facilities by Sasol and the associated growth in mining activities and residential development.

The development of new power stations is of high strategic importance with tight timeframes. Commissioning of the first generation unit will start in September 2010 and additional water needs to be available by mid-2011 according to the expected water requirements. A solution addressing the water needs of the Lephalale area must be pursued. The options to augment existing water supplies include transferring surplus effluent return flows from the Crocodile River (West) / Marico WMA to Lephalale and the area around Steenbokpan shown on the map indicating the study area on the following page.

The Department of Water Affairs (DWA) commissioned the Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP) to analyse the options for transferring water from the Crocodile River (West). In April 2008, the Technical Module of this study was awarded to Africon in association with Kwezi V3, Vela VKE and specialists. The focus of the Technical Module is to investigate the feasibility of options to:

- Phase 1: Augment the supply from Mokolo Dam to supply in the growing water requirement for the interim period until a transfer pipeline from the Crocodile River (West) can be implemented. The solution must, over the long term, optimally utilise the full yield from Mokolo Dam.
- Phase 2: Transfer water from the Crocodile River (West) to the Lephalale area. Options to phase the capacity of the transfer pipeline (Phases 2A and 2B) must be investigated.

The Technical Module has been programmed to be executed at a Pre-feasibility level of investigation to identify different options and recommend the preferred schemes, which was followed by a Feasibility level investigation of the preferred water schemes. Recommendation on the preferred options for Phase 1 and Phase 2 Schemes were presented to DWA during October 2008 and draft reports were submitted during December 2008. The Feasibility Stage of the project commenced in January 2009 and considered numerous water requirement scenarios, project phasing and optimisation of pipeline routes. The study team submitted a draft Feasibility report during October 2009 to the MCWAP Main Report in November 2009.

This report (Report 8 – Feasibility Stage, P RSA A000/00/8709) covers the detail geotechnical investigations that have been performed for Phase 2 of the MCWAP. These include the pump station site, pipeline route and borrow pits.



MOKOLO AND CROCODILE RIVER (WEST) WATER AUGMENTATION PROJECT FEASIBILITY STUDY:

TECHNICAL MODULE

DETAIL GEOTECHNICAL INVESTIGATIONS –

PHASE 2

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LIST OF ABBREVIATIONS

DPL	Dynamic Penetrometer Light
DWA	Department of Water Affairs
MCWAP	Mokolo and Crocodile River (West) Water Augmentation Project
NE	North-East
NW	North-West
PGA	Peak Grand Acceleration
RWR	Raw Water Reservoir
SPT	Standard Penetration Test
TLB	Tractor-Loader-Backhoe
WMA	Water Management Area

1. BACKGROUND AND INTRODUCTION

1.1 Scope of the Investigations of this Report

The project entails two separate phases, Phase 1 and Phase 2.

Phase 1 comprises expansion of the pumping station at the Mokolo Dam, a pipeline to the delivery area at Mathimba Raw Water Reservoir (RWR), near Lephalale, and a pipeline extending west from a point approximately 2 km south-east of the delivery area to Steenbokpan.

Phase 2A comprises abstraction from the Crocodile River at the Vlieëpoort site, and transfer via a pipeline to link with the western leg of the Phase 1 pipeline at Steenbokpan, the flow of which will be reversed to accommodate transfer to the delivery area near Lephalale. Phase 2B is a future phase for increased supply from the Crocodile River (West) to the larger Steenbokpan / Lephalale area.

These phases are contained in separate reports, with this report dealing with Phase 2A.

This report presents the findings of the feasibility-level geotechnical investigations conducted for the various components of Phase 2A of the project, namely:

- Vlieëpoort abstraction weir;
- Vlieëpoort balancing dam and desilting works; and
- Vlieëpoort Steenbokpan pipeline.

The layout of the scheme is shown in Figure 1-1.

Since the time that the geotechnical investigations were carried out and compilation of this report, the alignment of the pipelines has changed (but not yet necessarily finalised). This report thus deals with the alignment as it was at the time of the investigation, and shown as on Figure 1-1. Any additional investigations occasioned by changes to this alignment will have to be carried out in the future and the findings supplied as Addenda to this report.



Figure 1-1: Geotechnical Investigation Routes – Phases 1 & 2A

1.2 Geotechnical Introduction

The study commenced with a desk study of available information; the findings of which were presented in the Project Inception Report.

The various abstraction and conveyance options were subsequently briefly visited by the Technical Team. These preliminary geotechnical assessments of the various possible components for the respective were then considered in the selection of the favoured options for further geotechnical study.

Further geotechnical investigations were then conducted for the favoured options; the results of which are presented in this report. Geotechnical investigations were conducted for the following components:

- Vlieëpoort abstraction weir;
- Vlieëpoort desilting works and balancing dam; and
- Vlieëpoort Steenbokpan transfer and delivery pipeline.

Due to financial and budgetary restraints, test pits on the pipeline were dug at 5 km spacing. Additionally, no borrow pit investigation or laboratory testing was carried out.

1.3 Previous Investigations and Available Information

Available geological information included the published 1:250 000 scale geological maps (Council for Geoscience). The relevant sheets comprised the following:

- Sheet 2326 Ellisras
- Sheet 2426 Thabazimbi

Although a large number of geological investigations, dating back 70 years, had been conducted for the alternative Boschkop weir site, there is only a single record of an earlier geological report on the Vlieëpoort site (Gelletich, 1983).

The report discusses the geology on the farm Hanover 341KQ. No specific reference is made to the founding of a dam and the purpose of this report is uncertain. Potentially relevant comments include those regarding the presence of dolomite beneath the banded ironstone and the occurrence of rubble accumulations (read talus deposits) at the foot of steep slopes. It is further commented that no major tectonic "disturbances" occur, and no dip-faults.

No records of any detailed investigations conducted at the Vlieëpoort site could be sourced, and it is considered unlikely that such investigations were ever conducted.

The pipeline follows a "greenfield" alignment and no site specific data relating to it was identified.



Figure 1-2: Geotechnical Geology – Phases 1 & 2A

2. INVESTIGATION METHODOLOGY

2.1 Outline

A broad outline of the geotechnical investigations conducted for these feasibility studies is presented below.

These geotechnical studies comprised the following:

- Desk study of available information;
- Field verification of the geology;
- Rotary core drilling;
- Test pitting;
- Water pressure (Lugeon) testing of the boreholes and measurement of the water table;
- Standard Penetration Tests (SPT);
- Dynamic Penetrometer Light (DPL) tests (often referred to as DCP tests);
- Assessment of seismic hazard; and
- Assessment of climate and weathering.

2.2 Desk Study

Available geological and geotechnical data was assessed. On a broad level, the published geological maps (Council for Geoscience) were studied, as well as orthophotos and images from Google Earth, while on a more detailed level previous site investigation reports were studied. No stereo-pairs of aerial photographs could be sourced in time for the desk study.

The available sources of information are listed above (Section 1.3).

2.3 Field Verification of the Geology

Brief site visits were undertaken, during which a visual inspection of the outcrops and other geological exposures was carried out.

The coordinates of test pits along the conveyance routes and boreholes drilled at the abstraction weir and pump station sites were picked up using a hand-held GPS instrument, and the usual allowances for accuracy should be made. Coordinates comply with the WGS84 coordinate system, utilising the Hartebeeshoek94 Datum (Lo 27). No detailed levelling of the borehole or test pit positions was conducted, i.e. no reliable information on the elevations was recorded.

2.4 Rotary Core Drilling

Rotary core boreholes were drilled at the site of the proposed Vlieëpoort Abstraction Weir (numbered VBH1 to VBH9), as well as the Balancing Dam (numbered VBH DAM1 to VBH DAM4). Borehole VBH4 was subsequently cancelled.

Drilling had commenced at the initially-proposed site of the balancing dam, but subsequent revision of the layout resulted in the cancellation of this drilling and the relocation of the drill rig.

Boreholes on the right flank of the Crocodile River were drilled by Weppelmann Geotechnical Drilling in the period between March and May 2009. Drilling on the left bank of the proposed Vlieëpoort Weir was carried out by the Drilling Branch of the Department of Water Affairs (DWA) in the period between February and July 2009.

Boreholes were generally drilled vertically, with a single exception where an angled borehole was drilled from the river bank, angled beneath the river in order to investigate the founding conditions in that area.

Rotary core drilling was carried out in accordance with the CSRA standard specifications (CSRA, 1993).

Borehole cores were logged by engineering geologists in accordance with accepted South African practice (Brink and Bruin (Eds.) 2002). Borehole cores were also photographed. Borehole logs were prepared using DotPlot® software and are included in **Appendix B**. Photographs of the borehole cores are included in **Appendix C**. Borehole details are listed below in Table 2-1.

Borehole	Coordinates (WG27)		Reduced	BH	Remarks			
NO	Y	Х	(masl)	(m)				
Vlieëpoort a	Vlieëpoort abstraction weir							
VBH 1	- 31 910	2 725 590		18,50	Vlieëpoort abstraction weir			
VBH 2	- 31 915	2 725 540		31,75	Vlieëpoort abstraction weir			
VBH 3	- 31 990	2 725 570		47,80	Vlieëpoort abstraction weir			
VBH 4	- 32 000	2 725 525	899	-	Vlieëpoort abstraction weir – BH not drilled			
VBH 5	- 32 040	2 725 480	898	44,21	Vlieëpoort abstraction weir			
VBH 6	- 32 050	2 725 510	901	31,32	Vlieëpoort abstraction weir			
VBH 7	- 32 090	2 725 515	899	21,93	Vlieëpoort abstraction weir			
VBH 8	- 32 090	2 725 460	899 (?)	34,69	Vlieëpoort abstraction weir			
VBH 9A	- 32 036	2 725 441		16,75	Vlieëpoort abstraction weir			
VBH 15	- 31 890	2 724 690		3,27	High lift pump station – borehole cancelled due to layout change			
Vlieëpoort b	alancing dam							
VBH DAM1	- 31 295	2 722 607	919	9,66	Balancing Dam / High Lift Pumping Station			
VBH DAM2	- 31 726	2 722 474	932	8,91	Balancing Dam			
VBH DAM3	- 31 482	2 723 110		7,24	Balancing Dam			
VBH DAM4	- 31 790	2 723 191		9,66	Balancing Dam			

Table 2-1: Borehole Details

2.5 Test Pitting

Test pits were dug along the pipeline routes in order to assess the quality of the in-situ material. The test pits were dug using a Hydromek 102B tractor-loader-backhoe (TLB) as this would give a direct assessment of the excavatability of the materials present and allow their inspection in an undisturbed state. The profile encountered was logged by a geospecialist. Profiles were prepared using Winlog® software and are included in **Appendix D**. Photographs of the profiles are included in **Appendix E**.

After logging, the holes were immediately backfilled using the TLB. Where appropriate, DCP tests were carried out in order to obtain a quantitative assessment of the consistency of the soils encountered. The DCP soundings have been reduced to equivalent SPT N-values (blows per 300 mm penetrated) and presented graphically as N-value versus depth on the test pit profiles.

Where seepage had been encountered, and it was safe to do so, holes were left open for 24 hours to allow the water level to be measured before the hole was closed up. This provided a more quantitative assessment of the inflow that may be expected during construction.

Holes were generally dug to a depth of 4 m (approximately the invert of the pipe) or to refusal of the TLB.

At the time of profiling, an assessment of the conditions encountered in each hole was made and notes recorded relating to:

- Depth of refusal and nature of material on which refusal took place;
- Stability of trench sides;
- Likely longer term (safe) side slopes during construction;
- The presence of groundwater/seepage, level of occurrence, initial inflow and rest level after 24 hours;
- The anticipated utilisation (as bedding or soft backfill) of the soils encountered;
- Any other observations relevant to construction of the pipeline; and
- Burrow sources.

Sources of material suitable for use as bedding or soft backfill to the pipe were identified at a nominal spacing of 5 km along the pipeline. The target volume of material was 50,000 m³ per borrow pit. This approximates to 200% of the volume of material required as bedding/backfill for 5 km of pipeline. This is obviously a conservative approach, as it ignores the fact that much of the backfill could come from the pipe trench, except for rocky areas. However, it does allow for material and depth variability and for backup in the event that certain of the sources will not, for various reasons (environmental, socio-political, financial, etc.) be available during construction and will allow the distribution of sources that are actually employed during construction to be optimised.

SABS 1200 LB and SABS 0120: Part 3 LB give the standard specifications for selected granular material (used in construction of Class B, C and D bedding cradles) as follows:

- Grading requirements:
 - No material retained on 37.5 mm sieve
 - Less than 5% material retained on 19 mm sieve
 - More than 95% material retained on 0.6 mm sieve
- Compatibility requirements:
 - Up to and including 0.1: material suitable
 - Over 0.1 up to and including 0.4: material suitable (except for flexible pipes that may be subject to waterlogged conditions), but require extra care in compaction
 - Over 0.4: material unsuitable

Selected fill material as defined in SABS 1200 LB is defined as material that has a PI not exceeding 6 and is free from vegetation and lumps and stones of diameter 30 mm.

It was decided that these specifications are too stringent and would exclude most if not all of the material excavated from the pipe trench and borrow pits, requiring the sourcing of all selected granular and fill material from commercial sources (i.e. washed sand), thus increasing the cost of the project exponentially. Therefore, for this project the specification applied to the bedding and selected backfill material is that the maximum particle size is 10 mm and the maximum PI is 12. The compact requirements for the selected granular material, however, remain the same.

In addition to the borrow sources located, information was obtained of commercial sources of construction materials (crushed stone and sand for use in concrete).

The results of the investigation are given in **Appendix H**.

2.6 Packer (Lugeon) Testing and Measurement of the Water Table

Water pressure or packer / Lugeon tests were required to be conducted in the exploratory boreholes drilled at the abstraction weir site.

Tests were conducted in accordance with the methodology described by Houlsby (1976).

Test results are included on the borehole logs which are presented in Appendix B.

2.7 Standard Penetration Tests (SPTs)

Standard Penetration Testing (SPTs) was generally conducted in the boreholes drilled at the proposed site of the Vlieëpoort weir.

Tests were conducted at 1.5 m intervals within the upper, unconsolidated soil horizons. The number of blows was recorded for each 75 mm penetration of the total 450 mm. The total number of blows for 300 mm penetration, which follow the initial 150 mm 'bedding in', constitute the N-value.

Test results are included on the borehole logs which are presented in Appendix B.

2.8 Borrow sources

No borrow pits were identified during the investigations. In order to provide an indication of the likely location of bedding and backfill material, possible borrow areas have been indicated on Figure 1-1.

2.9 Laboratory testing

No laboratory testing was carried out.



Figure 2-1: Location of Test Pits, Rock Outcrops and Possible Burrow Pits

3. GENERAL GEOLOGICAL SETTING

3.1 Lithology and Stratigraphy

The large geographical area of the scheme, which extends from the proposed Vlieëpoort weir site in the south, to the delivery area near Steenbokpan in the north, also has, as a further consequence, an extensive geological coverage.

The oldest lithologies are found in the southern portion of the study area, becoming progressively younger towards the north.

The oldest lithologies encountered belong to the late Archaean to early Protozeroic (i.e. approximately 2 650 to 2 050 million years) Transvaal Supergroup, and comprise the dolomitic rocks and ironstone formations of the Chuniespoort Group, and the slightly younger shales, quartzites and lavas of the Pretoria Group.

The central portion of the study area is underlain by the sandstones of the Waterberg Group which are considered to be between 1 700 and 2 000 million years in age (Johnson *et. al.*, 2006).

The northern portion of the study area is underlain by rocks of the Karoo Supergroup which comprises a succession of sandstone, siltstone, shale and mudstone and are approximately 150 to 270 million years in age.

Extensive diabase intrusions are particularly prevalent with the central portion of the study area where they are seen to have intruded the sandstones of the Waterberg Group.

Extensive areas, particularly in the north, are covered by Quaternary Age sands which are younger than 1.8 million years.

3.2 Structural Geology

The structural geology of the study area is similarly highly complex and a single paragraph cannot do these complexities justice.

The older, Transvaal Supergroup rocks in the south of the study area have a moderate to shallow dip of 15° to 30° towards the south-east; reflecting the intrusion of the younger Bushveld Complex, which depressed these underlying strata. These Transvaal Supergroup rocks are extensively faulted. Although faults are generally of limited extent, some major faults, which can be traced for distances in excess of 50 km, can be identified.

In the south, the sandstones of the Waterberg Group dip at shallow angles in a northerly direction, but become almost horizontal towards the north. Prominent NE- and NW-striking lineaments are recognized and likely represent intrusive diabase dykes.

The Karoo sedimentary strata are essentially sub-horizontally bedded, but are extensively faulted. Some of the faults may be traced for significant distances.

3.3 Economic Geology

The right bank of the Crocodile River at the Vlieëpoort Abstraction Weir site forms the boundary of the Ben Alberts Nature Reserve within which Kumba Resources is mining iron ore.

The envisaged abstraction weir is not intended as a storage facility and the elevated water level will largely be confined within the current river bank. No new inundation of any mineable mineral reserves will therefore occur as a result of impounding.

In the north, beyond the Eenzaamheid Fault, coal is extracted on a large scale from the Ecca sediments. The pipeline routes are generally located south of this fault, except for limited transgressions in the vicinity of the Medupi Power Station.

3.4 Seismic Hazard

Published seismic hazard maps of southern Africa (Kijko, *et al*, 2003) indicate Peak Ground Accelerations (PGAs) in the order of 0.1 g to 0.04 g within the study area, becoming progressively lower towards the north. These accelerations might be considered to represent a moderate to low level of seismic hazard.

3.5 Climate and Weathering

The study area straddles the climatic N = 5 line (Weinert, 1980) which indicates that neither chemical decomposition nor mechanical disintegration are dominant modes of weathering, and that both modes of weathering are likely to have an influence.

4. GEOTECHNICAL INVESTIGATION

4.1 Vlieëpoort Abstraction Weir

4.1.1 Introduction

The Vlieëpoort site is located on the farm Hanover 341KQ. The proposed site is located at a narrowing of the valley where the Crocodile River cuts through the Vlieëpoortberge (Figures 4-1 and 4-2). Approximate coordinates for the site are Lo 27 Y -31 979 X 2 725 486.

The Vlieëpoortberge rise to elevations in excess of 1400 masl on either side of the river (Plate 1), where the elevation of the river bed is less than 900 masl. The site is characterised by a relatively wide river section, estimated in the order of 350 m.



Figure 4-1: Plate 1: A General View of the Vlieëpoort Abstraction Weir site, viewed from high up on the left flank, showing the general site topography. The Crocodile River can be discerned in the Valley



Figure 4-2: Geotechnical Investigations: Vlieëpoort Abstraction Weir – Site Layout Plan



Figure 4-3: Geotechnical Investigations: Vlieëpoort Abstraction Weir – Longitudinal Section



Figure 4-4: Geotechnical Investigations: Vlieëpoort Abstraction Weir, Balancing Dam – Layout Plan with Borehole Positions

At the time of the investigation the envisaged structure was likely to comprise a low diversion weir and including the low-lift pumping station located within the narrow valley, with appurtenant works comprising the desilting works (silt trap), and balancing dam as well as the high-lift pumping station located approximately 1500 m to 2500 m to the north of the weir site (Figure 4-4).

A public gravel road is located on the left flank of the river, while on the right bank access to the weir site is gained via the Ben Alberts Nature Reserve belonging to Kumba Resources.

A total of eight boreholes were drilled at the weir site; three on the left bank of the river (numbered VBH 01 to VBH 03) and five on the right bank (numbered VBH 05 to VBH 08 and VBH 9A). Borehole positions are indicated on Figure 2.3. Borehole VBH 04 on the left bank was cancelled

4.1.2 General Geology

The published 1:250 000 geological map (Thabazimbi Sheet 2426, Council for Geoscience) indicates the Vlieëpoort Mountains are aligned with the banded ironstone formations of the Penge Formation, and the dolomites, cherts and subordinate shales of the Malmani Subgroup; all of the Chuniespoort Group, Transvaal Supergroup. The strata strike roughly in a north-easterly direction and dip at angles between 20° and 30° in a south-easterly, i.e. upstream, direction. The drilling results indicate, however, that the bedding generally varies between 40° and 60°, although a single borehole (VBH 1) reveals a sub-horizontal attitude.

At the indicated weir position, the structure is located near the contact between the older dolomitic rocks and the younger banded ironstone formations which are slightly upstream, in the area of the bend in the road.

Upstream of the mountain range, the wider valley area is underlain by the sedimentary strata of the Timeball Hill Formation of the Pretoria Group, comprising shales and sandstones.

The geological map indicates a minor fault a short distance downstream of the indicated weir site, aligned roughly parallel to the proposed centre-line. The geological map gives no indication of faulting at the actual centre-line, nor is there field evidence to indicate the possibility of major faulting. The relatively straight section of the Crocodile River downstream of the weir site suggests a prominent lineament which might be a fault, although there is no evidence of visible dislocation of strata to support this.

4.1.3 Site Geology

4.1.3.1. Introduction

Because of the limited size of the proposed structure the entire structure is essentially located between the steep slopes of the Vlieëpoortberge, i.e. within the river section. For sake of simplifying, this site description, however, the respective river banks and the river channel will be described as separate entities.

The left bank extends for an approximate distance of 140 m from the river bank. The river channel is roughly 30 to 40 m wide and the right bank is in the order of 130 m in length.

The respective river banks are characterised by dense riverine vegetation. This 'greater river section' constitutes the floodplain and is therefore relatively flat with localised undulations which would be re-sculpted in the event of major flooding.

4.1.3.2. Left Flank

4.1.3.2.a.1 Overburden

The left flank is covered by alluvial deposits which are between 13 and 39.5 m in thickness; thinning in the direction towards the left flank and away from the river channel.

The alluvium comprises clays, as well as horizons of sub-rounded to sub-angular, medium to coarse gravel and cobbles (diameter 10 to 60 mm, occasionally up to 200 mm) in a matrix of sandy clay or clay. It is to be expected that these alluvial deposits are highly variable in terms of composition, both laterally and vertically, and that lenses or channels of clays, silts, sands and gravels / cobbles are to be expected.

4.1.3.2.a.2 Bedrock

Bedrock comprises banded ironstone, and is intersected at depths varying between 13 and 39.5 m.

The bedrock can be described as comprising moderately or highly weathered, very closely to closely jointed, occasionally medium jointed (spacing generally <60 mm up to 200 mm and occasionally 400 mm), orange- or grey-brown to purple, very hard rock occasionally medium hard rock banded ironstone.

On a regional scale, these banded ironstones are known to possess a dip between 20° and 30° in a south-easterly direction, i.e. upstream direction. The drilling results indicate a steeper dip of approximately 40° (from horizontal). Towards the left flank, however, borehole VBH 1 reveals the bedding to be sub-horizontal (dipping between 0° and 10°). At least two sets of discontinuities are recognized, comprising prominent joints via the bedding planes (either dipping at 0° to 10°, or approximately 40°), and a set of very steeply dipping or sub-vertical joints (70° to 90°).

The rock mass is further characterised by variable, irregular quartz veining reflecting the re-cementation of brecciated rock and suggesting the proximity of a possible fault. The variable attitude of the bedding might further suggest disruptions due to faulting.

4.1.3.3. River Channel

The geological conditions within the river channel were investigated by the drilling of a borehole angled from the right bank (VBH 5). Although a vertical borehole (VBH 4) was set out on the opposite bank, which would have allowed extrapolation of the geological horizons, this borehole was later cancelled. Borehole VBH 3, however, indicates the maximum thickness of the alluvial deposits is to the left of the current river channel.

No bedrock outcrop occurs within the river channel which is defined by sandy banks. Downstream of the envisaged centre-line the river channel becomes wider and is characterised by extensive sandbanks.

4.1.3.3.a.1 Overburden

The alluvial deposits within the river channel comprise a succession of clays, sands and horizons / lenses of sub-rounded to sub-angular gravels and cobbles (diameter generally between 10 and 100 mm, and occasionally up to 150 mm) within an expected matrix of clay or sand. Extrapolated borehole information indicates the thickness of alluvium varies between 28 and 39.5 m.

Borehole VBH 5 intersects a horizon immediately above the underlying bedrock which may represent partly recemented palaeo-alluvium. The recovered materials comprise angular to sub-rounded gravels and cobbles (diameter between 20 and 120 mm), comprising moderately to highly weathered, medium hard rock to extremely hard rock banded ironstone, within a matrix of ferruginized, sandy clay. The borehole suggests a horizon thickness of almost 5 m.

4.1.3.3.a.2 Bedrock

Bedrock comprises banded ironstone and is intersected at extrapolated depths varying between 28 and 39.5 m.

An upper, more-weathered horizon is recognised in places where the rock is described as highly to completely weathered, very closely to closely jointed (joint spacing generally less than 60 mm, occasionally up to 100 mm), orange-brown, hard rock to soft rock banded ironstone. In places, the recovered material comprises ferruginized sand which is taken to represent the completely weathered banded ironstone. This upper horizon is roughly 4 m in thickness, and is underlain by moderately weathered, closely occasionally very closely jointed, grey- and purple banded, extremely hard rock banded ironstone.

There is some evidence of re-cementation of the previously brecciated rock mass, as well as slippage via steep 60° to 70° joints which is considered to indicate a possible palaeo-fault. Within the upper, weathered bedrock horizon (which is largely recovered as gravel) there is abundant iron oxide or manganese oxide which is present as a cementing agent / matrix and it is possible that this material represents the weathered fault zone.

At least two sets of discontinuities are recognized. The dominant discontinuities are joints formed along the bedding, i.e. dipping at angles of approximately 40° to 50°, but other cross-cutting joints are also recognised.

4.1.3.4. Right Bank

A total of five boreholes were drilled on the right bank, including one located slightly downstream of the proposed centre-line (VBH 9A). Boreholes were drilled in a staggered pattern so as to maximise the borehole coverage and investigate a potential structure footprint, rather than investigating a single centre-line. One borehole (VBH 8) was effectively drilled on the lower slopes of the right flank.

4.1.3.4.a.1 Unconsolidated Overburden

The drilling shows variable but substantial thickness of alluvial deposits. These materials vary in thickness between 11.36 m (BH VBH 9A) and 28 to 29 m (VBH 5 and VBH 8, respectively). Drilling results suggest the thicker deposits are found near the current river channel and at the foot of the right flank, with less substantial deposits in the centre, suggesting possible palaeo- or sub-surface channels. The shallowest alluvium thickness is in tersected by borehole VBH 9A, but it should be noted that this borehole is some distance downstream of the centreline.

The alluvium comprises a succession of clay and clay sand as well as medium to coarse gravels and cobbles in a matrix of silty to clayey sand. It is to be expected that these materials show both lateral and vertical variation and occur in the form of variable horizons and lenses or pockets. It might further be noted that the recovered samples are not likely representative, particularly the non-cohesive horizons and the matrix which were typically not recovered at all.

4.1.3.4.a.2 Bedrock

Bedrock comprises banded ironstone.

The rock mass is generally moderately weathered, but in places (boreholes VBH 6 and VBH 8) an upper horizon of highly to completely weathered rock is present.

Where present, this upper horizon comprises very closely to closely jointed (completely fractured), grey to brown, medium hard to soft rock banded ironstone. The underlying rock mass may be described as moderately weathered (or occasionally moderately to highly weathered), closely jointed occasionally very closely or medium jointed, grey and purple banded, extremely hard rock banded ironstone.

Up to three sets of discontinuities are recognised. The dominant joints are the "bedding joints" which have developed via the banding within the rock material. These "bedding joints" are generally very closely or closely spaced and typically have smooth surfaces and are characterised by staining. The attitude of these "bedding joints" generally varies between 40° and 60°. In addition to these joints, at least two or three other joint sets occur; namely sub-vertical joints, sub-horizontal joints as well as a second set of joints which dip at angles between 40° to 50°. Typically, joint surfaces are smooth, or slightly rough, and stained.

In many instances, there was evidence of minor slippage or dislocations via these bedding joints, as indicated by dislocation of the banding. These 'micro-faults' have generally been re-cemented; typically by quartz infill but also (borehole VBH 8) by a green-coloured mineralisation. In places, these re-cemented joints appear randomly orientated and suggest an earlier brecciation. This brecciation, as well as the common slip via the "bedding joints", suggest the proximity of a fault or fault zone.

4.1.4 Geotechnical Considerations

4.1.4.1. Weir Founding Conditions

The envisaged, low structure will, of necessity, comprise a mass concrete gravity structure in order to safely pass the expected floods. At the very least the structure would incorporate a central spillway section to allow the design floods to be safely passed. Structure details were, however, not confirmed at the time of writing.

Such a structure would typically require founding on a rock mass of sufficient quality that it would possess a Deformation Modulus in the order of 8 GPa to 10 GPa. Typical rock mass characteristics which would be expected would comprise a degree of weathering of moderate or better, a rock material strength of approximately 100 MPa, joints which are at least closely spaced, or wider, and joint conditions which would comprise slightly rough surfaces and medium hard to hard joint wall rock.

The depths at which such conditions are encountered in the respective boreholes are summarised below in Table 4-1.

BH no	Founding depth for mass concrete structure (m)	Thickness of unconsolidated overburden (m)	Thickness of rock horizon to be excavated (m)	Remarks
VBH 1	13,1	13,1		
VBH 2	27,0	27,0		
VBH 3	>39,5	39,5		
VBH 4	?	?	?	Borehole not drilled
VBH 5	34,3	28,1	6,2	Borehole angled; reference here is made to vertical depths / thickness
VBH 6	28,5	27,3	1,2	
VBH 7	18,5	18,5		
VBH 8	34,0	29,35	4,65	
VBH 9A	14,0	11,4	2,6	

Table 4-1: Founding Depths for a Mass Concrete Structure

It is evident from the summary presented above in Table 4-1 that suitable founding conditions for a mass concrete structure are only encountered at prohibitive depths.

4.1.4.2. Site Suitability / Founding Options

Ordinarily, such depths which vary between 13 m and more than 39 m would be considered too great to justify the excavation of the unsuitable founding materials. If construction of an abstraction structure at this site is to be considered, then it would either require that the founding criteria are revised, or that a means of foundation improvement is employed so that the structure might be founded at shallower, more-economic depths.

One of the ways to move forward would be to improve the foundation to the extent that economic founding of the abstraction weir could be achieved at significantly shallower depths. A possible option in this regard would be to conduct jet grouting along the length of the centre-line; and in this manner a cut-off would be created in the form of overlapping columns on which the abstraction weir might be constructed.

These broad feasibility-level investigations are not sufficient for the detailed design of such a jet-grouted cut-off, and further studies would be required in this regard. It is unlikely that the size of the coarse fraction (gravels and cobbles) would present a constraint on the technical feasibility of this option.

4.1.4.3. Foundation Permeability and Grouting

The Packer or Lugeon tests were intended to be conducted within the bedrock horizons of all boreholes. However, in many instances the degree of jointing was such that a sealing of the packer could not be achieved and the tests could not be conducted satisfactorily as the desired pressures could not be attained. In other situations, the very broken bedrock prevented successful insertion of the packer, and testing could not be successfully conducted.

Test results are summarised below in Table 4-2.

BH no	Lugeon values (depths of testing in brackets)	Depth of water table (m)	Remarks
VBH 1		7,35 m	Total water loss at 13,25 m
VBH 2		7,84 m	Total water losses at depths of 27,15 m and 30,5 m. Could not attain desired pressures
VBH 3		7,19 m	No packer test – packer cannot be inserted
VBH 5	8 (35,7 – 38,3 m)	5,96 m	Loss of drilling water at 5,96 m. Decreasing Lugeon values indicate void filling. Test unable to attain higher pressures.
VBH 6	? (28,2 – 31,32 m)	8,05 m	Loss of drilling water recorded at 8,05 m. Lugeon test unable to reach desired pressure.
VBH 7		?	Drilling water loss of 30%, becoming 70% from depth of 18,40 m
VBH 8	? (31,5 – 34,69 m)	18,05 m	Unable to reach desired pressure. Increasing Lugeon values possibly indicates wash-out (or dilation)
VBH 9A		5 80 m	

Table 4-2: Summarised Results from Packer (Lugeon) Testing

Note: Measurements shown for VBH 5 (angled borehole) refer to vertical depths

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VBH 5	8 (35,7 – 38,3 m)	5,96 m	Loss of drilling water at 5,96 m. Decreasing Lugeon values indicate void filling. Test unable to attain higher pressures.
VBH 6	? (28,2 – 31,32 m)	8,05 m	Loss of drilling water recorded at 8,05 m. Lugeon test unable to reach desired pressure.
VBH 7		?	Drilling water loss of 30%, becoming 70% from depth of 18,40 m
VBH 8	? (31,5 – 34,69 m)	18,05 m	Unable to reach desired pressure. Increasing Lugeon values possibly indicates wash-out (or dilation)
VBH 9A		5,80 m	

Table 4-3: Summarised Results from Packer (Lugeon) Testing

Note: Measurements shown for VBH 5 (angled borehole) refer to vertical depths

Only limited packer (Lugeon) tests were possible and the results are insufficient to draw firm conclusions from. Indications are, however, that the founding conditions are relatively permeable. Comments from the driller pertaining to losses of drilling water further indicate the rock mass is likely permeable.

As a rule, permeable founding conditions such as encountered here would require a programme of curtain grouting to reduce the water losses and contribute to the general integrity of the founding rock mass. Because of the small size of the envisaged weir, the usual concern relating to elevated uplift pressures due to the increased head is not an issue and foundation grouting would therefore not be required for that reason.

4.1.5 Recommendations

These preliminary feasibility-level investigations have identified a number of areas of uncertainty which require clarification for detail design and construction purposes. Follow-up geotechnical investigations must give attention to the following:

- Geophysical surveys must be conducted at the proposed site, with the aim of confirming whether or not linear geological features traverse the proposed centreline. Such features would typically represent lineaments or fault- or shear-zones. A further output of the geophysical surveys would be to extend the ability to extrapolate the points of information gathered from the boreholes. In addition, the geophysical surveys would provide information on the depths to bedrock and, therefore provide an indication of anomalies which might have some influence on final site layout.
- Additional drilling must be conducted in order to investigate any geophysical anomalies which might have been detected. This is a necessary step in order to evaluate the geotechnical conditions associated with such anomalies, and allow evaluation of the implications for design of the structure. Additional drilling will also

be driven by design changes to the structure and the need to confirm founding conditions for these changes.

- Limited test pits are required to provide opportunity to investigate the shallow soil horizons, and enable representative samples to be obtained for laboratory testing. An intensive test pitting exercise is not warranted as the alluvial deposits are very thick, and the ability to investigate the deeper profile is severely limited. Excavation within these alluvial deposits would also be associated with some safety risk.
- A laboratory testing programme is necessary in order to characterise the geological materials. Of particular importance is the need to obtain representative samples of the alluvial materials from depths up to 40 m, in order to do grading analyses and assess the geotechnical characteristics. This information would be the key in assessing likely material behaviour (possible susceptibility to liquefaction, for example) and possible foundation solutions, including jet-grouting, for example. The usefulness of the laboratory test data will be dependent on the ability to recover representative samples. While SPT and thin-walled tube samplers are routinely used to obtain samples, the presence of gravels and cobbles would prevent the success of these sampling methods.

4.2 Vlieëpoort Balancing Dam, Silt Trap and High Lift Pump Station

4.2.1 Introduction

Initial layouts provided for the silt trap, balancing dam and high lift pump station to be located in close proximity to the Vlieëpoort Abstraction Weir. Following subsequent optimisation of the layout, these components were re-located to positions roughly 1 500 to 2500 m to the north of the weir site (Figure 4-8).

Because of constraints on site access due to the thick natural bush, and the difficulty in obtaining landowner permission for the investigations, only limited site investigations were ultimately conducted for the balancing dam and high lift pump station; these investigations comprised the drilling of four rotary core boreholes (boreholes VBH DAM1 to VBH DAM4) at positions along the proposed perimeter of the balancing dam. One of these boreholes (VBH DAM1) also investigates conditions at the proposed position of the high lift pump station. No boreholes were drilled inside the proposed footprint area of the balancing dam.

The proposed balancing dam site is characterised by gentle slopes, becoming slightly steeper towards the south, including in the area of the proposed silt trap. The proposed site is covered by dense bush, with cultivated lands located directly on the northern boundary and a short distance from the western boundary (Figures 4-4 and 4-5).



Figure 4-5: Plate 2: A General View of the Proposed Site of the Balancing Dam. The Proposed Footprint just encroaches on the Cultivated Land. Borehole VBH DAM2 is visible in the foreground

4.2.2 General Site Geology

According to the published 1:250 000 geological map (Thabazimbi Sheet 2426, Council for Geoscience), the proposed site is underlain by dolomitic rocks, cherts and subordinate shales of the Malmani Subgroup, Chuniespoort Group, Transvaal Supergroup.

The proposed site is located on the gentle topography to the north-west of the Vlieëpoort Mountains which are aligned with the banded ironstone formations of the Penge Formation. The strata strike roughly in a north-easterly direction, and dip at angles between 20° and 30° in a south-easterly direction.

The proposed site is covered by colluvial materials and no geological structure is indicated on the geological map. It is likely, however, that the dolomitic strata reflect a similar attitude, i.e. dipping at moderate angles in a south-easterly direction.

The geological map indicates a minor fault aligned roughly parallel to the Vlieëpoort Mountains, a short distance downstream of the indicated weir site. The geological map gives no indication of faulting at the proposed site, although this may be masked by the cover of colluvial materials.

4.2.3 Geological Profile

The rotary core boreholes revealed the following horizons:

- Colluvium
- Dolomite residuum; and
- Dolomite bedrock.

These respective horizons are discussed briefly below.

4.2.3.1. Colluvium

The entire site is covered by colluvium comprising red-brown clay or sandy clay (matrix) with varying proportions of angular to sub-angular, medium to coarse gravels (diameter 10 to 90 mm) predominantly of banded ironstone, but also with occasional chert or dolomite. In places the gravel component is absent, and an upper horizon of red-brown, sandy clay is present (VBH DAM1). Typically, the finer matrix was not recovered during the drilling process.

The thickness of this horizon varies between 1.5 and 3.5 m.

4.2.3.2. Dolomite Residuum

The term 'dolomite residuum' is used here to collectively refer to the soils developed by the weathering of the dolomitic rocks. This horizon comprises angular to sub-angular, gravels and boulders (diameter up to 800 mm) of whitish, leached chert and red-brown, clayey sand matrix.

The horizon of dolomite residuum is intersected on the eastern boundary area of the balancing dam, and is absent from boreholes drilled on the western boundary, suggesting this horizon pinches out in a westerly direction. The dolomite residuum is intersected at depths between 1.54 and 2.67 m (VBH DAM2) and between 3.19 and 4.69 m (VBH DAM4); i.e. the thickness of this dolomite residuum varies between 1.13 and 1.5 m.

4.2.3.3. Dolomite Bedrock

The entire footprint of the balancing dam is underlain by dolomite.

Bedrock was intersected at depths varying between 1.5 and 4.69 m. No trend of increasing bedrock depths is recognised and it is assumed that the bedrock profile is randomly irregular.

The dolomite bedrock is generally described as slightly weathered (slightly leached); closely to medium jointed, grey to light grey, very hard rock dolomite. The dolomite intersected in borehole VBH DAM1 comprises moderately to highly weathered (leached), very closely to medium jointed, light grey to blue grey, very hard rock, brecciated dolomite.

Zones of prominently stained joints are recognised in borehole VBH DAM3 associated with infill material comprising brown, sand clay (with some wad), as well as in borehole VBH DAM1 where the infill comprises red-brown, gravelly clay. In other areas (VBH DAM4), minor cavities are recorded within the dolomite bedrock, while prominent weathering is also evident via certain joints.

Brecciated rock or recemented breccia is intersected in boreholes VBH DAM2, VBH DAM4, as well as in VBH DAM1.

Up to four joint sets are recognized; including sub-horizontal joints $(0^{\circ} - 10^{\circ})$, moderately steep $(30^{\circ} - 40^{\circ})$ and steep joints (70°) as well as sub-vertical joints (90°) . As a rule, the joint surfaces are stained and contain clay infill which varies in thickness from a thin film up to 5 mm. The possibility that some of the infill material has been lost in the drilling process cannot be discounted.

4.2.4 Groundwater Conditions

No water tables were recorded in any of the four boreholes drilled on the footprint and it may be assumed that the water table occurs at depths greater than 10 m.

With the exception of borehole VBH DAM3, all the remaining boreholes recorded losses of drilling water varying between 20% and 40%, indicating permeable conditions.

4.2.5 Geotechnical Considerations

4.2.5.1. Dolomite Stability

The entire footprint of the balancing dam is underlain at shallow depth by dolomitic rocks. As soluble rocks, dolomites are known to be associated with sinkholes and dolines. Construction of any structure on dolomitic rocks therefore comes with a certain level of risk of instability and construction of a water-bearing structure and associated services will increase the level of risk due to the possibility of water leakages.

The limited drilling results reveal prominently stained joints with sandy clay infill which suggest possible seepage paths. In addition, boreholes reveal minor cavities. The presence of cavities and seepage paths for water infiltration would be major contributory factors for future dolomite instability.

Another factor to consider is the presence of a large central-pivot irrigation system directly on the northern boundary of the balancing dam site. Accelerated drawdown of the groundwater level in dolomitic areas is known to trigger sinkholes and other subsidence, and a detailed hydro-geological study is necessary.

To date, no specific investigations into the dolomite stability have been conducted.

There is likely limited scope for relocation of the balancing dam to ensure founding on non-dolomitic rocks. Should this site still be considered, then it is essential that dolomite stability investigations be conducted to quantify and define the level of risk. Such investigations would initially comprise a gravity survey, followed by a programme of percussion drilling and evaluation of the findings. Depending on the level of inherent risk it may be possible to construct the reservoir at this site, provided appropriate designs are adopted and strict water precautionary measures are adopted.

Should limited areas of high risk for sinkhole and doline development be identified, then it might further be possible to optimise the site layout in order to minimise exposure to this risk.

A certain level of risk will always be present, however, and this would have to be acknowledged in the design and construction of the structure, as well as in the operational requirements.

4.2.5.2. Founding

Founding conditions at the proposed site of the High Lift Pumping Station were investigated by borehole VBH DAM1.

The founding depth for the pumping station is not only dependent on the geological profile; operational requirements and the level at which the pumps must be installed are also important considerations.

The borehole indicates that bedrock occurs at a relatively shallow depth in this area (3.5 m). Structure-specific foundation requirements might dictate founding levels within the bedrock, or at shallow depths within the colluvial horizon comprising colluvial gravels and cobbles in a clay matrix.

A single SPT was conducted at a depth of 1.5 m in borehole VBH DAM1, yielding an N-value of 69. While the recovered borehole sample comprised sandy clay, the underlying material contains gravel and cobbles. It is therefore possible that the tested horizon also contained a coarser fraction which was not recovered, and that the N-value might not be representative.

No investigations were conducted at the proposed site of the Silt Trap. At this stage, foundation conditions might be assumed to approximate conditions at the balancing dam.

4.2.5.3. Construction Materials

To date, no investigations of likely embankment construction materials have been conducted.

4.2.5.4. Recommendations

Follow-up geotechnical investigations are required for design purposes for the Balancing Dam / Pump Station and Desilting Works. At the time of writing, it was not known whether an alternative site would be selected, or whether the site considered at feasibility level would be favoured. While the follow-up geotechnical investigations would obviously be dependent on decisions in this regard, in essence these investigations would comprise the following;

- Geophysical investigations must be conducted at proposed alternative sites. These geophysical surveys would have two desired aims; firstly the identification of major geological features which might represent major weakness zones and represent potential problematic founding conditions, and secondly to provide information on the geological profile which would then enable extrapolation between the boreholes and test pits.
- In addition to the above, further geophysical work in the form of a gravity survey would be essential at any site which is underlain by dolomite. Such a gravity survey is an important part of any dolomite stability investigation and a precursor to follow-up percussion drilling (discussed below).
- Test pitting on the proposed sites must be conducted in order to investigate the shallow founding conditions and confirm the geological profile, as well as to allow representative samples to be obtained for further laboratory testing. Test pitting will also be conducted at potential borrow areas, should investigations indicate that materials would have to be sourced elsewhere.
- Boreholes must be drilled. The purposes of the drilling would include verification of the geological conditions at depths greater than the maximum depth of the test pits, investigation of any identified geophysical anomalies, the recovery of samples in order to assist in the characterisation of these geological materials and for measurement of geotechnical parameters by means of penetration testing, and Lugeon testing, for

example. Drilling will comprise both percussion drilling, specifically as part of the dolomite stability investigation where gravity anomalies would be targeted, as well as rotary core drilling to investigate the founding conditions for the various structures (dam / desilting works, pump station) and allow SPTs to be conducted.

A laboratory testing programme is required in order to characterise the geomaterials on the alternative footprints and provide information on their suitability for their use in construction of foundation platforms, etc. Tests will include foundation indicators (grading using both sieves and hydrometer, Atterberg Limits), as well as compaction characteristics (CBRs and Mods).

4.3 Pipeline

4.3.1 Introduction

The geology along the pipeline route varies considerably as shown in Table 4-4 and on Figure 1-2.

Rock Types	Formation	Group	
Sand, ferricrete, calcrete		Quaternary	
Diabase		Post-Waterberg intrusive	
Sandstone	Mogalakwena Formation	Waterberg Group	
Sandstone, mudstone	Makgabeng Formation		
Dolomite, chert	Malmani Subgroup		
Quartzite, shale	Black Reef Formation	Transvaal Supergroup	
Arkose, greywacke, andesite	Buffelsfontein Group		
Granite		Lebowa Granite Suite	
Granite, gneiss		Archaean Complex	

Table 4-4: Geology of Phase 2A

The pipeline route commences in the south at Vlieëpoort, where it is underlain by rocks of the Transvaal Supergroup (mostly dolomite, chert, arkoses and andesite), before crossing onto Archaean Granite. After crossing back onto Transval Supergroup rocks, it then traverses mainly Waterberg sediments (sandstone and some mudstone), with patches of granite and diabase. In the north (from about 10 km south of the point where the pipeline splits from the Spoornet line), the pipeline is on Quaternary sands (with calcrete and ferricrete), which overlie Waterberg Group sandstone.

The various geological units encountered along the centreline of the pipeline are given sequentially (from south to north on Table 4-5) and their extent is shown.

km	Anticipated Geology
0 - 5	Dolomite
5 - 7.7	Pretoria Group and dolomite
7.7 - 15.7	Granite
15.7 - 34.2	Alluvium
34.2 - 37.2	Dolomite
37.2 - 42.6	Waterberg sandstone
42.6 - 57.6	Alluvium
57.6 - 86.6	Waterberg sandstone, diabase
86.6 - 98.8	Alluvium
98.8 - 125	Quaternary sand, diabase

Table 4-5: Geology Summary for Phase 2A Route

4.3.2 Pipeline Route

Test pits were excavated at a nominal spacing of 5 km along the route of the pipeline. In order to facilitate access to the route, test pits were dug within the reserve of various proclaimed roads and within the Spoornet rail reserve. The test pit profiles are given in **Appendix D** and photographs are given in **Appendix E**.

Pits were excavated using a TLB (Hydromek 102B 60kW) and profiled by a geospecialist in accordance with accepted South African standards (Brink and Bruin, 2002). The terms used are defined in **Appendix A**. Dynamic Penetrometer Light (DPL) soundings were undertaken immediately adjacent to the test pits in order to provide a quantitative assessment of the consistency of the in-situ materials. These soundings are shown graphically as equivalent SPT values (blows per 300 mm penetrated) on the relevant soil profiles.

No laboratory testing was undertaken on samples recovered from the Phase 2A alignment, nor were potential borrow areas investigated.

The Phase 2A investigation was intended to give a general impression of likely geotechnical conditions over the pipeline route, but due to the wide spacing of the pits, detailed conclusions and summaries cannot be made regarding the ground conditions. More frequent spacing will be necessary to confirm that the profiles encountered are indicative of prevailing conditions and not localised deposits.

Frequent exposures occur in cuttings along the rail line, and this data has been included on the profile summary in **Appendix G**.

4.3.3 Summary and Conclusions

More varied geology is present on the Phase 2A route. Probably not so much within the Waterberg Group to the north, which are similar to the soils encountered along the western half of the Phase 1A MS pipeline route. Along the southern half of the Phase 2A route, extensive outcropping of the rock was observed within the rail cuttings. Significant alluvium deposits are also anticipated along the Phase 2A route, which may include expansive soils. At the southern end of the pipeline route Transvaal Supergroup rocks will be encountered.

The test pits and documented geology indicate that borrow pits are likely to be easily identified over the northern half of the route. The thinner, more gravelly soils over the southern half will make the location of borrow pits more difficult, and significant haulage distances may be necessary. Transportation of the borrow material will not be significantly hampered by adverse topography, as the majority of the pipeline lies alongside existing roads and along the Spoornet rail alignment. Where the pipeline route lies within farm boundaries, the land is also generally flat.

The geology along the route is variable and this reflects in the geotechnical conditions anticipated. In view of the wide spacing of test pits, reliance is made on observations made on site. The numerous cuttings and borrow pits along the Spoornet line provided a valuable insight into conditions along the route. The findings are summarised in **Appendix G** and on Figure 2-2.

The geotechnical conditions anticipated along the pipeline are summarised in the following sections.

4.3.3.1. Excavatability

The predicted average depth to refusal is as follows:

km	Average refusal depth (m)
0-37	2.2
37-54	2.5
54-56	0.0
56-59	2.0
59-71	1.5
71-75	0.0
75-78	2.5
78-92	3.0
92-98	2.0
98-106	3.0
106-115	1.0
115-120	2.0

Table 4-6: Predicted Average Depth to Refusal

4.3.3.2. Suitability for Bedding/Soft Backfill

The predicted thickness of material suitable for use as bedding or soft backfill is as follows:

km	Average thickness of suitable material (m)
0-6	0.0
6-15	1.5
15-37	0.0
37-50	1.5
50-56	0.0
56-58	2.0
58-61	1.0
61-75	0.0
75-82	2.0
82-88	3.0
88-101	1.5
101-106	2.0
106-120	1.0

Table 4-7:	Predicted	Thickness for	r Bedding/Soft B	ackfill
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4.3.3.3. Groundwater

No seepage was encountered in any of the test pits dug along the route and it appears that this is unlikely, except in the vicinity of streams (and particularly on the south bank of the Matlabas River).

4.3.3.4. Stability of Excavations

An average side slope to trenches of 1:1 in soft may be assumed (i.e. to an estimated average overall depth of 1.5 m). In the Waterberg sandstones, which underlie most of the route, excavations into rock may be cut vertically.

4.3.3.5. Borrow Sources

No investigations for bedding and soft backfill were carried out.

Notwithstanding this, bedding material and soft backfill should be freely available north of about km 55 and an average haul distance of 5 km seems to be indicated. In this area, it is probable that much of the soil excavated from the pipe trench will be re-usable for bedding and soft backfill. Between km 0 and 55, borrow sources would seem to be difficult to locate and haul distances of at least 10 km are likely. Possible locations of borrow sources are indicated on Figure 2-2.

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APPENDIX A:

SUMMARY OF SOIL AND ROCK PROFILE DESCRIPTION TERMINOLOGY

1. MOISTURE CONDITION			2. COLOUR		
Term Description					
Drv			The Predominant colours or colour combinations		
Slightly	Requires ac	dition of water to reach optimum	are described including secondary coloration		
moist	moisture co	ntent for compaction	described as banded, streaked, blotched,		
Moist	Near optimu	im content	mottled, speckled or stained.		
Very Moist	Requires dr	ying to attain optimum content			
Wet	Fully satura	ted and generally below water table			
-		3. CONS	SISTENCY		
	3.1 1	Non-Cohesive Soils		3.2 Cohesive Soils	
Term		Description	Term	Description	
Very Loose	Crumbles ve geological p	ery easily when scraped with ick	Very soft	Easily penetrated by thumb. Sharp end of pick can be pushed in 30 - 40mm. Easily moulded by fingers.	
Loose	Small resist geological p	ance to penetration by sharp end of ick	Soft	Pick head can easily be pushed into the shaft of handle. Moulded by fingers with some pressure.	
Medium Dense	Considerabl end of geolo	le resistance to penetration by sharp ogical pick	Firm	Indented by thumb with effort. Sharp end of pick can be pushed in up to 10mm. Can just be penetrated with an ordinary spade.	
Dense	Very high resistance to penetration to sharp end of geological pick. Requires many blows of hand pick for excavation.		Stiff	Penetrated by thumbnail. Slight indentation produced by pushing pick point into soil. Cannot be moulded by fingers. Requires hand pick for excavation.	
Very Dense	High resista pick. Requi	ligh resistance to repeated blows of geological ick. Requires power tools for excavation		Indented by thumbnail. Slight indentation produced by blow of pick point. Requires power tools for excavation.	
4. STRUCTURE		5. SOIL TYPE			
	-		5.1 Particle Size		
Term	Description		Term	Size (mm)	
Intact	Absence	Absence of fissures or joints		>200	
Fissured	Presence	of closed joints	Pebbles	60 – 200	
Shattered	Presence of closely spaced air filled joints giving cubical fragments		Gravel	60 – 2	
Micro- shattered	Small sca the size o	le shattering with shattered fragments f sand grains	Sand	2 - 0,06	
Slickensided	Polished p movemen	planar surfaces representing shear tin soil	Silt	0,06 - 0,002	
Bedded Folitated	Many resi rock.	dual soils show structures of parent	Clay	<0,002	
		6. ORIGIN		5.2 Soil Classification	
	6.1	Transported Soils			
Term	n	Agency of Transportation			
Collusi		Gravity deposite	۹ ۸۱۰۰		
Talua	Talue Seree et acerte		10 00		
i aius S Hillwash		Fine colluvium		20 80	
Alluvial		River deposits	SAND 40 SLIGHTLY SLIGHTLY SLIGHTLY 50 CLAY		
Aeolian Wind deposits		Wind deposits			
Litoral Beach deposits		Beach deposits			
Estuarine Tidal – river deposits		60 CLAY SLIGHTLY CLAY 40 70 SANDY SANDY SLIGHTLY CLAY 80 CLAY SANDY SILTY SLITY CLAY 90 CLAY SANDY SILTY CLAY SLITY 90 SLIGHTLY CLAY SANDY SILTY CLAYEY SAND CLAYEY SAND 100 SANDY SILTY SANDY SILT SILT 10 100 SANDY SILTY CLAYEY SAND SANDY SILT SILT 0 100 SILTY SAND SANDY SILT SILT 0 100 SILTY SAND SANDY SILT SILT 0			
Lacustine Lake deposits					
6.2 Residual soils					
These are products of in-situ weathering of rocks and are described as e.g. Residual Shale					
6.3 Pedocretes					
Formed in transported and residual soils etc. calcrete, silcrete, manganocrete and ferricrete.					

SUMMARY OF DESCRIPTIONS USED IN ROCK CORE LOGGING

1. WEATHERING					
Term	Symbol		Diag	nostic Features	
Residual Soil	W5	Rock is discoloured and completely changed to a soil in which original rock fabric is completely destroyed. There is a large change in volume.			
Completely Weathered	W5	Rock is discoloured and changed to a soil but original fabric is mainly preserved. There may be occasional small corestones.			
Highly Weathered	W4	Rock is discoloured, discontinuities may be open and have discoloured surfaces, and the original fabric of the rock near the discontinuities may be altered; alternation penetrates deeply inwards, but corestones are still present.			
Moderately Weathered	W3	Rock is discoloured, d alteration starting to pe	iscontinuities may be enetrate inwards, inta	open and will have discolo act rock is noticeably weake	ured surfaces with r than the fresh rock.
Slightly Weathered	W2	Rock may be slightly discoloured, particularly adjacent to discontinuities, which may be open and will have slightly discoloured surfaces, the intact rock is not noticeably weaker than the fresh rock.			
Unweathered	W1	Parent rock showing n	o discolouration, loss	s of strength or any other we	eathering effects.
	2.	HARDNESS		3. C	OLOUR
Classification	Fi	Field Test Compressive Strength Range MPa			
Extremely Soft Rock	Easily peeled wit	h a knife	<1	The predominant colours or colour combination	
Very Soft Rock	Can be peeled with a knife. Material crumbles under firm blows with the sharp end of a geological pick.		1 to 3	are described including secondary colouration described as banded, streaked, blotched, mottled, speckled or stained.	
Soft Rock	Can be scraped with a knife, indentation of 2 to 4 mm with firm blows of the pick point.		3 to 10		
Medium Hard Rock	Cannot be scraped or peeled with a knife. Hand held specimen breaks with firm blows of the pick.		10 to 25	•	
Hard Rock	Point load tests must be carried out in order to distinguish between these classifications		25 - 70		
Very Hard Rock	These results may be verified by uniaxial compressive strength tests on selected samples.		70 - 200	•	
Extremely Hard Rock		>200			
			4. FABRIC	-	
4.1 (Grain Size		4.2	Discontinuity Spacing	
Term	Size (mm)	Description for: lami	Bedding, foliation, nations	Spacing (mm)	Descriptions for joints, faults, etc.
Very Coarse	>2,0	Very Thickly Bedded		> 2000	Very Widely
Coarse	0,6 - 2,0	Thickly	y Bedded	600 - 2000	Widely
Medium	0,2 - 0,6	Medium Bedded		200 - 600	Medium
Fine	0,06 - 0,2	Thinly Bedded		60 - 200	Closely
Very Fine	< 0,06	Lam	ninated	<60	Very closely
Thinly Laminated			<3		
5. ROCK NAME		6. STRATIGR	APHIC HORIZON		
Classified in terms of origin:					
IGNEOUS	Granite, Dio	rite, Gabbro, Syenite, D Trachyte, Andesite, Ba	iabase, Dolerite, salt.	Identification of rock type in terms of stratigraphic	
METAMORPHIC	Slate, Q	uartzite, Gneiss, Schert	t, Sandstone	hori	zons.
SEDIMENTARY	Shale, Muc Conglor	Shale, Mudstone, Siltstone, Sandstone, Dolomite, Conglomerate, Tillite, Quartzite, Linestone.			

APPENDIX B: BOREHOLE LOGS

APPENDIX C: BOREHOLE CORE PHOTOGRAPHS

(D-1)

APPENDIX D: SOIL PROFILES

APPENDIX E: TEST PIT PHOTOGRAPHS

APPENDIX F:

TEST PIT SUMMARY SPREAD SHEET

REPORT DETAILS PAGE

Project name:	Mokolo and Crocodile River (West) Water Augmentation Project (MCWAP)
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PSP

Approved for PSP by:

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