TCTA

MOKOLO AND CROCODILE WATER AUGMENTATION PROJECT (MCWAP)

CONTRACT № TCTA 07-041

CONSULTING SERVICES FOR MCWAP

PHASE 2: GEOTECHNICAL INVESTIGATIONS
STAGE 3: Operational Reservoir - Steenbokpan

VOLUME 3: GEOTECHNICAL INTERPRETIVE REPORT

June 2012

MOKOLO CROCODILE CONSULTANTS

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MOKOLO AND CROCODILE WATER AUGMENTATION PROJECT

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[1] Finalisation pending outstanding Phase 2 Geotechnical Results

MOKOLO AND CROCODILE WATER AUGMENTATION PROJECT

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PHASE 2: GEOTECHNICAL INVESTIGATIONS

STAGE 3: Operational Reservoir - Steenbokpan

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PHASE 2: GEOTECHNICAL INVESTIGATIONS STAGE 3: Operational Reservoir - Steenbokpan

VOLUME 3: GEOTECHNICAL INTERPRETIVE REPORT

EXECUTIVE SUMMARY

1 REPORT STRUCTURE

Report “Phase 2: Geotechnical Investigations: Stage 3: Operational Reservoir - Steenbokpan” comprises three volumes, of which this is Volume 3:

- Volume 1: Geotechnical Report (Factual report);
- Volume 2: Annexures supporting Volume 1; and
- Volume 3: Geotechnical Interpretive Report (**This Volume**).

This Volume interprets the data contained in Volumes 1 and 2 and should be read in conjunction with them.

It must be appreciated that the diameter of the pipe is not known at the time of reporting. This fact must be borne in mind whenever any interpretation is given. Such interpretation may have to be revisited once the pipe diameter is known.

2 SITE DESCRIPTION

The investigations carried out covered two main aspects, namely the centreline and borrow pit investigations. Geotechnical conditions are consistent over the whole site. The area is flat with the only features occasional pans (some dry but some with water).

The underlying geology is Waterberg sediments (sandstone, conglomerate). These sediments were only observed in test pits and no outcrops were observed, as the bedrock is covered by sands of Quaternary Age. Ferricrete or calcrete frequently occurs below the sand and refusal often occurs on them.

Sand suitable for use as bedding or selected backfill material is in good supply and borrow sources are well distributed over the site. Much of the soft material from the trench may be re-used as backfill. On a few occasions slight seepage was recorded in test pits.

The nearest sources of concrete aggregate are located near Lephalale (about 50 km east of Steenbokpan). The demand for concrete aggregate will be low as there are no structures of any significance.

3 INTERPRETATION OF PIPE CENTRELINE INVESTIGATIONS

3.1 Seismic Hazard

Peak Ground Acceleration values are predicted at between 0,08 and 0,10 g, with a 10% probability of being exceeded in a 50 year period. This would equate to a moderate to low level of seismic risk.

3.2 Excavatability and Definition of Rock-line

Refusal occurred on a variety of materials, ranging from ferricrete, ferricrete gravels and boulders, calcrete and Waterberg sandstone. The rock-line used in calculations has been based on the refusal depth recorded in test pits dug with a TLB.

It is expected that where pedogenic materials (ferricrete, calcrete) and also gravels are encountered, an excavator may be able to penetrate these, provided that the bucket used has sharp teeth as this is probably more relevant than break-out force for “ripping” through these materials. Sandstone was encountered in relatively few test pits and the TLB employed was only able to excavate between 0 and 500 mm into bedrock.

It must be pointed out that any assessment of excavatability based on individual test pits is often conservative, as it is constrained by the fact that:

- the trench bottom is of limited extent; and
- the sides of the trench have a confining effect on the material encountered in the base.

For the purpose of this report (and will be used in subsequent tender documentation), the rock-line was defined using the observed rock level in the test pits, spaced at 200 m intervals. It should be noted that a more accurately defined rock-line between the test pits should be established before any blasting is done, particularly because of the undulating rockhead. It is recommended that, at the start of construction, rock levels are confirmed by closely spaced holes for the accurate design and execution of the drilling and blasting process. The actual rock level should also be confirmed at each of the blastholes drilled.

The blast design will define the excavatability of the blast rock. The excavatability of all other material is generally regarded as non-problematic. However the impact of roots should be taken into account.

3.3 Blasting

As the pipe diameter is not known at this stage, meaningful comment on blasting operations and technique is not possible.

3.4 Corrosivity

Based upon the total DIN 50929-2 assessment, the route is generally deemed to be non corrosive, but specific "hot-spot" corrosion has been identified, predominantly in the vicinity of the existing Eskom ash dump, where leachate may traverse along the permeable selected backfill to the existing pipeline and permeate into the low points, thereby increasing the aggressiveness of the adjacent soil. The primary corrosion protection system, namely the pipe coating, needs to be managed with care during construction and the secondary corrosion protection system, namely the Cathodic Protection, needs to be introduced at the "hot-spots".

3.5 Stability of Trench Excavation

Slight seepage was encountered in only three test pits. No instability of the sides was observed in these, but caving of the sides did occur in two other (dry) test pits where loose sands were present. In general, it is expected that, where no groundwater is present and roots are carefully cut, and not pulled, the trench side slopes in earth material can be as follows:

- Vertical side slopes up to 1.5 m above the trench bed is expected to have a stand-up time of approximately 10 days; and
- Above the 1.5 m level, the trench side slopes should be battered to 1:1 (V:H).

The exception to this is when water is encountered in sandy materials. This condition can give rise to running sands which will only be stable at a slope of about 1:3 (V:H). It is possible that groundwater may be encountered in the vicinity of pans, following periods of heavy rain that fills the pans.

3.6 Material Compactability

In general the sands identified for use as bedding and selected backfill to the pipe from either trench excavation or borrow pits are fairly fine grained and their compactability factor (CF) is in the range of 0.20 to 0.46 but with the upper limit usually about 0.40. Sand with a CF between 0.10 and 0.40 is considered to be suitable, provided precautions (such as ensuring that backfill is maintained at the same level on both sides of the pipe and a minimum thickness of sand is placed over the top of the pipe before compaction of the main fill commences) are taken around flexible pipes and where the sand may become saturated. Those with a CF >0.40 are generally considered unsuitable, but may be considered for use in the event that no other material is available.

The sands from borrow pits for use as bedding and selected backfill contain a fairly small percentage of oversize material and it will be necessary to screen the sand at the borrow pit to scalp off this oversize and roots.

The general interpretation of the compactability test results is that the sands will potentially require above average compaction effort and supervision of the process to ensure compliance with the specified compaction requirements. It is recommended that the layer thickness used for compaction should not exceed 150 mm (uncompacted) and the material should be watered to optimum moisture content.

3.7 In-situ Soil Fertility

The results of the soil fertility tests are included in Volume 2. These results were, however, received at a late stage in the process and have not been interpreted and such interpretation will have to be conducted in the future.

5 INTERPRETATION OF BORROW PIT AND COMMERCIAL SOURCE INVESTIGATIONS

5.1 Material Availability

The distribution of borrow sources of granular material suitable for use as bedding and backfill is shown in Table 1.

Table 1: Borrow pit summary

BP no.	Location (WGS84 Lo27)		Chainage (m)	Offset to pipeline (m)	Est. volume bedding & soft backfill (m^3)	Compactability Factor (range)
	Y	X				
43#	-041 362	2 658 260	-2,000	on Stage 1	≈100,000	0.32 – 0.39
53+	-040 487	2 641 428	2 300	50 L	50,000	0.30 – 0.34
52	-037 097	2 640 453	6 000	50 L	≈100,000	0.32 – 0.41
50	-035 600	2 633 400	12,800	100 R	≈100,000	0.36 – 0.41
48	-032 678	2 632 164	16,500	200 R	≈100,000	0.31 – 0.46
49	-029 600	2 629 600	20,700	50 R	≈100,000	0.20 – 0.38
15$	-028 890	2 622 230	25,500	500 R (on Stage 4)	≈100,000	0.34 – 0.39

Closest BP to south, on Stage 1 $ BP on Stage 4 + BP only partly investigated

L = Left/south or west of pipeline R= Right/east of pipeline

5.1.1 Distribution of Borrow Sources

As may be seen from Table 1, potential borrow sources are well distributed along the route and the spacing between them is generally less than 4 km, except for the 6.8 km between BP50 and BP52. The distribution shown assumes that BP53 (which was not fully investigated) will prove to be suitable.

It must be pointed out that, where sources have been identified, the volume of material that has been proven is double that required (assuming a 2,000 mm diameter pipe). This implies that all borrow pits will not necessarily be opened, nor will the whole borrow pit necessarily be utilised. The likelihood of this is further enhanced when it is accepted that the volume of bedding material required from borrow ignores any bedding material that is available from the pipe trench.

5.1.2 Quality of Borrow Materials

Prospecting for borrow pits was, in certain instances, hampered by constraints on access to some properties to prospect for granular material. This is particularly the case for BP53 and, once the access issues have been resolved, detailed test pitting (at 30 m centres) and laboratory testing will be necessary to prove this source.

The sources identified are discussed separately.

BP53. This source has not been investigated in detail and only limited laboratory testing has been carried out on it. Based on these limited results, it appears that there should be about 50,000 m^3 of sand present. The sand is of good quality (A2-4, with a single A4 encountered) and mixing and stockpiling in the borrow pit should yield an A2-4 material which should be suitable for use in all bedding material types (SC2 = bed, bedding cradle and selected fill blanket).

Should it, for any reason, not be possible to utilise this source, material from BP52 and BP43 (the nearest source to the south on Stage 1) would have to be hauled in to fill this 8 km gap. The quantity of material available from these two sources should be sufficient to service this greater spacing between them.

BP52. This source contains more than the targeted 100,000 m^3 and is of good quality, generally an A2-4 but with a single A3 tested. Stockpiling (and mixing) in the borrow pit should yield an A2-4 material which is suitable for all bedding material types.

BP50. This source is of good quality (A2-4), should provide more than 100,000 m^3, and is located about 6,800 m north of BP52. The material is suitable for use in all bedding material types.

BP49. The material in this source is similar to that in BP52 and, after mixing and stockpiling in the borrow pit should yield an estimated 100,000 m^3 of A2-4 material suitable for use as all material bedding types.

BP15. This source is located on Stage 4 and is about 500 m right/east of the pipeline route (opposite Ch. 25,500 m). It will have to be utilised to fill the 6,7 km gap between BP49 (Ch. 20,900 m) and the end of Stage 4 in Steenbokpan (about Ch. 27,600). The source should yield about 100,000 m^3, so should be able to provide sufficient material to meet the requirements of Stage 4 and to address the “gap” to the south of Steenbokpan. Details of the source are given in the Stage 4 report.

It is anticipated that the project will not have sufficient volume of topsoil to rehabilitate all affected areas. It is recommended that the trees and roots that are removed during construction be used to supplement the organic content of potential fertile soil.

5.2 Corrosivity

Based upon the total DIN 50929-2 assessment, the borrow pit material is generally deemed to be non corrosive. With the exception of BP7A, BP8, BP23 and BP24, the borrow pits are ostensibly free from Sulphate Reducing Bacteria (SRB), as the sulphide levels are low and within acceptable values. BP7 and BP8 show elevated levels of sulphide, indicating that SRB are active and present in these soils. However, the material from these sources can be utilised as the impact of SRBs can be sufficiently managed by means of coating selection and cathodic protection.

5.3 Material Compactability

The general interpretation of the borrow pit compactability test results is that the sands contain a high percentage of fines and will require above average compaction effort and supervision of the process to ensure compliance with the specified compaction requirements. It is recommended that the layer thickness used for compaction should not exceed 150 mm (uncompacted) and the material should be watered to optimum moisture content.

5.4 Haulage of Borrow and Spoil

The pipe diameter and its associated trench dimensions has not yet been fixed. Accordingly, no accurate calculation of volumes has been possible and no mass haul diagram has been prepared. It is estimated that approximately 350,000m^3 of material will be excavated from the pipe trench.

In due course a mass haul diagram must be developed to plan the optimal use of the borrow sources and to balance that with the required spoil volume. This mass haul diagram should also highlight the necessity for, and optimal location of, any additional borrow pits to be identified in the future.

5.5 Commercial Sources

Two commercial sources of fine and coarse aggregate have been identified in the vicinity of Thabazimbi. These are about 50 km east of Steenbokpan and the Operational Reservoir. Details of these sources are given in the Stage 1 report.

No structures of any significance are anticipated on Stage 3 and the volume of concrete will be minimal.

5.6 Availability of Laboratory Test Results

At time of writing, not all laboratory test results have necessarily been supplied by the testing laboratory and the following cut-off dates apply:

- Received by 30 July 2011, bound into Annexures (Volume 2) and have been interpreted in Volume 3; and
- Received after 1 August 2011, are not bound into Volume 2, not interpreted in Volume 3 and are stored electronically in the Project Files.

On the first page to each Annexure in Volume 2 a summary is included detailing the status of any outstanding test results.

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GLOSSARY

ACV	Aggregate Crushed Value
ARC	Agricultural Research Council of South Africa
BH	Borehole
BP	Borrow Pit
CF	Compaction Factor
CP	Cathodic protection
DCI	Ductile Cast Iron
DCP	Dynamic Cone Penetrometer
DPL	Dynamic Probe – Light
DWA	Department of Water Affairs
Erosion	The group of processes whereby soil or rock material is loosened or dissolved and removed from any part of the earth's surface
Fertiliser	An organic or inorganic material, natural or synthetic, which can supply one or more of the nutrient elements essential for the growth and reproduction of plants
Land capability	The ability of land to meet the needs of one or more uses under defined conditions of management
MIC	Micro biologically induced corrosion
MCC	Mokolo Crocodile Consultants
MCWAP	Mokolo Crocodile Water Augmentation Project
PI	Plasticity Index
PPD	Peak particle displacement
PPI	Probable Performance Index
PPV	Peak particle velocity
RL	Reduced level in metres above mean sea level (masl)

SANAS	South African National Accreditation System
SAR	Sodium Absorption Ratio
SPT	Standard Penetration Test
SRB	Sulphate reducing bacteria
TAM	Total available moisture
TCTA	Trans-Caledon Tunnel Authority
TLB	Tractor-loader-backhoe
WGS84	World Geodetic System (1984)

1 INTRODUCTION

Report "Phase 2 Geotechnical Investigations: Stage 3: Operational reservoir - Steenbokpan" comprises three volumes, of which this is Volume 3:

- Volume 1: Geotechnical Report (Factual report);
- Volume 2: Annexures supporting Volume 1; and
- Volume 3: Geotechnical Interpretive Report (**This Volume**).

This Volume interprets the data contained in Volumes 1 and 2 and should be read in conjunction with them. Where Figures have appeared in the first two Volumes, these are not repeated in this Volume.

It must be appreciated that the diameter of the pipe is not known at the time of reporting. This fact must be borne in mind whenever any interpretation is given. Such interpretation may have to be revisited once the pipe diameter is known.

2 INTERPRETATION

2.1 Background

2.1.1 Earlier Investigations

As part of the Feasibility Stage investigations during an earlier stage of the project, geotechnical investigations were performed for Phase 2 of the MCWAP.

The pipeline route investigation carried out during the Feasibility Stage comprised test pitting (using a TLB[2]) along the centreline of the pipeline route at a nominal spacing of 5 km. The pits were dug to a depth of 4 m (or to refusal of the TLB), were profiled in accordance with standard procedures and logs of each test pit compiled. The soils encountered were visually evaluated to provide a preliminary assessment of their suitability for use as bedding and selected backfill to the pipe. No borrow sources were identified, nor was any laboratory testing carried out on any samples.

Dynamic Probes – Light (commonly incorrectly referred to as DCPs) were conducted adjacent to and in selected test pits in order to provide a quantitative assessment of the consistency of the soils encountered. These soundings were reduced to equivalent Standard Penetration Tests (SPT) N-values (blows per 300 mm penetrated) and are presented graphically (as SPT N-values versus depth) on the soil profiles.

The fieldwork was carried out under competitive tender by the soils testing laboratory, Civilab.

Applicable data from these investigations has been extracted from the reports on this work and is integrated into the current report.

[2] Minimum characteristics: a) Backhoe depth not less than 4 m; gross power not less than 70 kW; and bucket breakout force not less than 60 kN.

2.1.2 Current Investigations

The investigations carried out covered two main aspects, namely the centreline and borrow pit investigations.

2.2 Availability of Laboratory Test Results

At time of writing, not all laboratory test results have necessarily been supplied by the testing laboratory and the following cut-off dates apply:

- Received by 30 July 2011, bound into Annexures (Volume 2) and have been interpreted in Volume 3; and
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On the first page to each Annexure in Volume 2 a summary is included detailing the status of any outstanding test results.

2.3 Centreline Investigation

The centreline investigation comprised excavation of test pits (with a TLB) at a nominal 200 m spacing along the pipeline route. Between Chainage 3,100 and 5,850 m (where toe route follows the boundary between Rooipan 357LQ and Rooipan 355LQ) access to the centreline was not permitted by the landowners. This gap must be filled in later investigations once access has been arranged.

In order to provide information over this section of the route test pits were dug along Road D175 from the Operational Reservoir to where it reconnects with the preferred route on Road D175 (Ch. 0 to 8,603 m). This Alternative Alignment is up to 2,500 m south of the preferred route.

Where the route continues northwards towards Steenbokpan, test pits were dug within the Road D175 road reserve and are thus offset by about 20 m from the pipeline centreline. They are, nevertheless, considered to be representative of the conditions to be expected on the centreline.

During an earlier phase of this project (Pre-Feasibility Stage), test pits were dug at a nominal spacing of 5 km along the centreline. The profiles recorded and laboratory test results from this investigation have been extracted form that study and are included in this report.

2.3.1 Excavatability

A summary of the conditions encountered at each test pit is given on the spreadsheet bound into Volume 2 (Annexure A1) and a summary of the refusal depths encountered in test pits is shown graphically on Figure 1 and on Figure 2 for the Alternative Alignment.

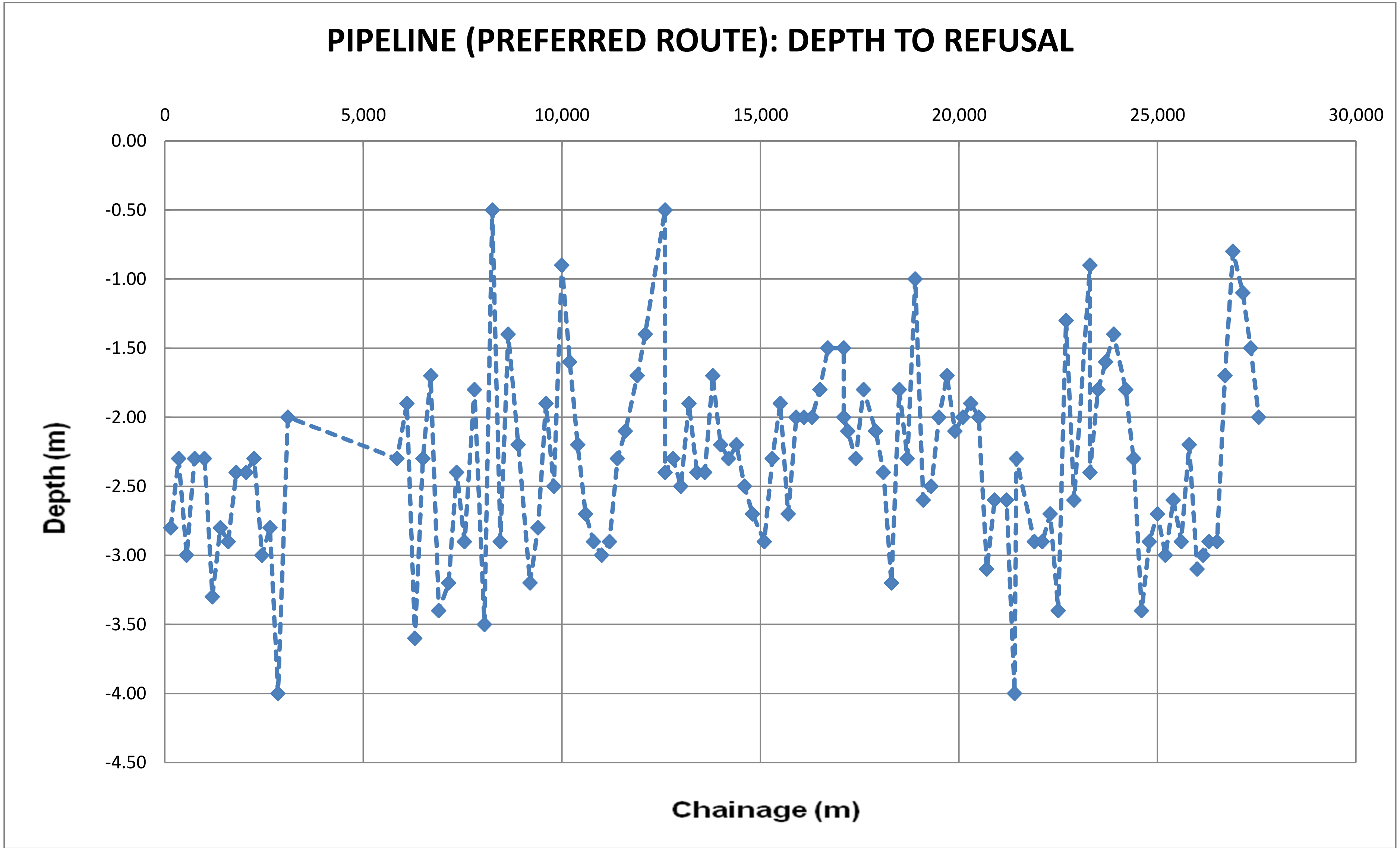

Figure 1: Summary of refusal depths: Preferred Route: Chainage 0 to 27,600 m

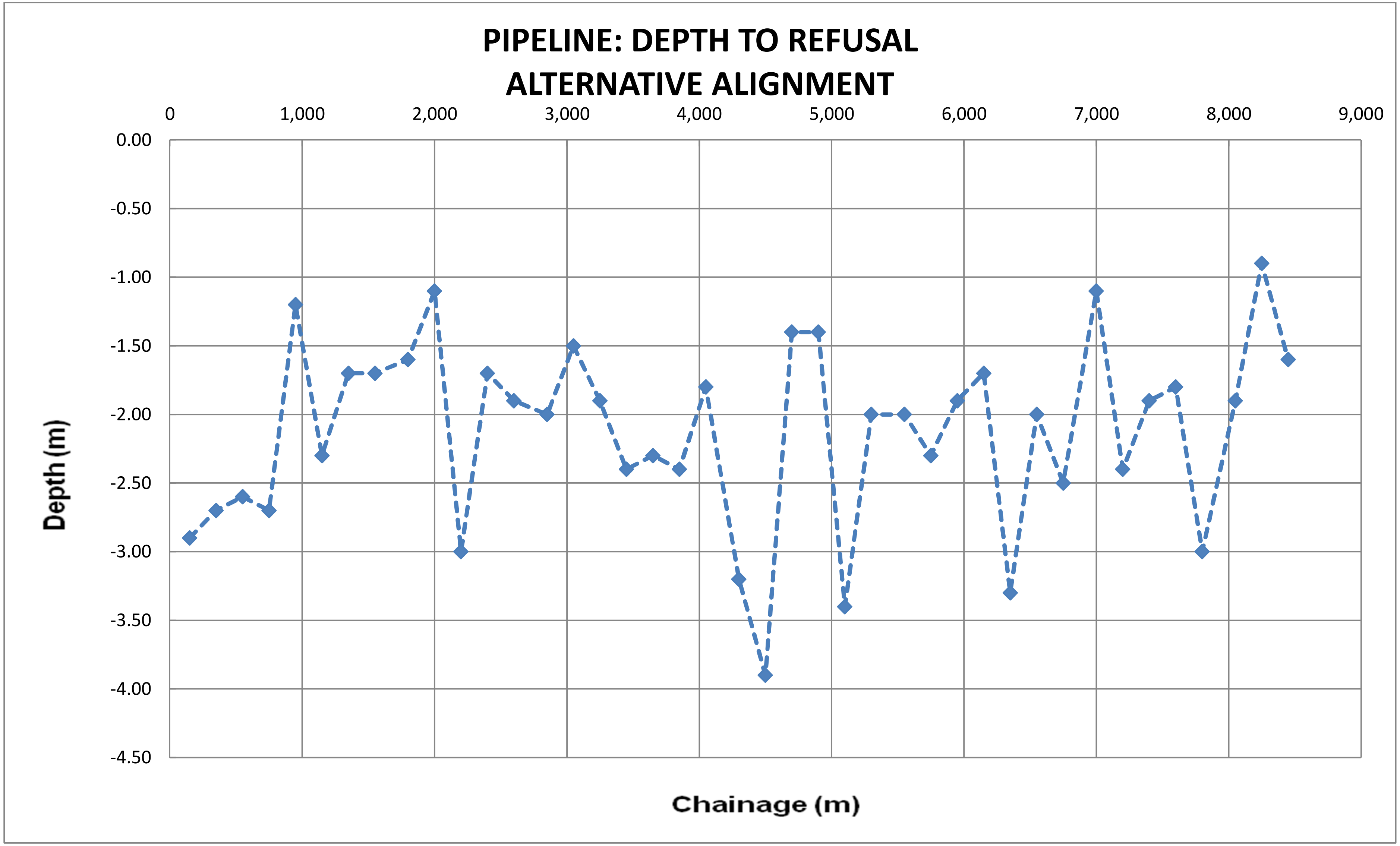

Figure 2: Summary of refusal depths: Alternative Alignment: Chainage 0 to 8,600

Based on a visual appraisal of the graphs presented in Figures 1 and 2, the average refusal depths along the route are summarised in Table 1.

Table 1: Summary of refusal depths (Preferred Route)

Chainage (m)	Average depth to refusal (mm)	Trench Depth
0 – 3,100	2,300	4,000 (Assumed 2,000 diameter pipe)
3,100 – 5,850	? (no access)	
5,850 - 10000	1,800	
10,000 – 13,000	500 – 3,000	
13,000 – 18,000	2,200	
18,000 – 21,000	2,000	
21,000 – 22,000	3,000	
22,000 – 24,000	1,700	
24,000 – 26,500	2,700	
26,500 – 27,600	1,400	

Table 2: Summary of refusal depths (Alternative Alignment)

Chainage (m)	Average depth to refusal (mm)	Trench Depth
0 – 0,800	2,700	4,000 (assumed 2,000 diameter pipe)
0,800 – 4,000	2,000	
4,000 – 4,250	3,500	
4,250 – 5,000	1,400	
5,000 – 7,900	2,400	
7,900 – 8,600	1,500	

Where Waterberg Sandstone was encountered, refusal of the TLB occurred very close to the rockhead. Based on data from elsewhere on the project, the sandstone is near-horizontally bedded, generally with medium spaced bedding joints (200 – 600 mm apart). Refusal usually occurs on a bedding plane and excavation is limited by the inability of the equipment to grip and prise out blocks. Where the sandstone is sufficiently closely jointed, excavation can proceed by prising out individual blocks. The blocky nature of the sandstone usually results in an irregularly sided excavation as blocks are pulled out of the side.

During MCWAP Phase 1 investigations, elsewhere on the project, excavatability tests were carried out on the Waterberg Sandstone using a 35t excavator (CAT 330C) fitted with a 3-tine rock bucket (as will probably be used during construction to define the boundary between “hard” and “soft” excavation). The investigation showed that it was only possible to excavate 100 to 400 mm into the Waterberg Sandstone, after about 15 minutes of effort. It must be pointed out that the test pits dug were of limited length and width. It may be that, during construction where a longer and wider trench is excavated, it may be possible to grip and prise out blocks. Once a block is loosened, it may then be possible to dig up successive adjacent blocks.

Refusal of the TLB frequently occurred on the ferricrete or calcrete. It is anticipated that a large excavator, particularly if fitted with sharp teeth, may be able to penetrate the pedocrete. Where the pedocrete was penetrated, it was found to be up to 1,400 mm thick, but is generally less than 500 mm thick. In places the ferricrete was found to be underlain by a material identified as calcrete. It was only possible to excavate a shallow depth into this material and it may be that the “calcrete” is in fact weathered sandstone. This interpretation will have to be confirmed in due course.

2.3.2 Blasting

It is anticipated that blasting will be required to trench in the Waterberg (and also in diabase, if this is intersected).

Blast design is dependent on the diameter of the pipe*. As this diameter is not known at this stage, this matter will have to be addressed in the future.*

2.3.3 Corrosivity

The rate of corrosion of buried ferrous (steel and Ductile Cast Iron (DCI)) pipelines will depend largely upon the conditions prevailing in that environment. These conditions are largely controlled by the type of soil, dissolved substances present in the soil, the biological activity of the soil, stray currents from both transit or adjacent cathodic protection (CP) stations and long line corrosion currents, etc.

The corrosive nature of the soil is typically subdivided into “broad”, “general” or “typical” assessments such as the soil resistivity and the comprehensive or detailed studies involving the “full” or global assessment of the (prevailing) soil, such as the chemistry, geomorphology, etc.

The soil resistivity is a broad and general approximation of the corrosivity of a soil along a pipeline route. Most soils are neither homogeneous nor uniform and therefore these resistivity (conductivity) measurements are purely a basis, and an indication regarding the corrosion risks which may be broadly or generally described along a given route, but the measurement is not absolute. It gives an indication of the apparent resistivity (corrosion risk) and will indicate if sulphate reducing bacteria (SRB) are to be expected, whether cathodic protection will be required along most of the route or predominantly in hot spots or in isolated areas. It also gives a typical indication as to the protective coatings that may be used as the primary corrosion protection system of the ferrous pipeline.

It must also be appreciated that the soil resistivity (the inverse of the soil conductivity) is dependent upon the soil moisture content, amount of dissolved salts, etc. Therefore, the measurements are susceptible to seasonal variations in the moisture content of the soil and fluctuations in the level of ground water, rainfall patterns, etc.

In order to understand the true corrosive natures of the soil, a number of measurements need to be assessed, of which the soil resistivity is but one of these measurements.

Soils are generally characterised by their texture, colour and drainage properties. Soils that are coarse in texture, such as sands and gravels, tend to permit free circulation of air to a reasonable depth and are free draining. Corrosion rates often approach those experienced in atmospheric conditions. Fine textured soils, such as clays and silts have a high water retaining capacity and also tend to restrict the circulation of oxygen. These soils can promote anaerobic conditions leading to very corrosive conditions. Often “mixed” or non homogeneous soils are encountered along a pipeline route which can result in the two extremes. In a similar way in which the water composition affects the corrosion rate inside the pipelines, the groundwater composition and soil chemistry contributes towards the external corrosivity.

The majority of soils tend to be neutral in nature, in other words, they possess a pH which ranges from 6 to 7.5. Soils in which pH exceeds 8, are generally classified as alkaline. Acidic soils, whose pH is less than 6 are generally aggressive towards buried ferrous materials.

The pH is important, as both alkaline and acidic soils affect the performance of coatings and the efficacy of cathodic protection.

The contribution of micro biologically induced corrosion (MIC) in soils has been known for many years. In essence, all soils are biologically active, however, it is generally accepted that the anaerobic conditions that occur in clay-containing soils promote MIC, as do de-aerated soils located some 1.5 m below grade. The presence of sulphate reducing bacteria (SRB) in soils causes significant and serious damage to ferrous pipes. This is due to the fact that SRB act as very efficient depolarising agents.

The most detailed and comprehensive specification and/or code of practice relating to the “Global Corrosive Nature” of soils, for ferrous materials (steel and DCI) is covered under the German Specification DIN 50 929-3.

The DIN 50 929-3 essentially investigates twelve (12) main items, which may be briefly summaris

ed below:

1. Type of soil
   a) Cohesion, clay, mud, peat, fen, etc.;
2. Soil Resistivity or alternatively soil conductivity;
3. Water Content;
4. pH; Buffer Capacity
   a) Acidity Value;
   b) Alkalinity Value;

5. Sulphide content;
6. Neutral Salts (water soluble Chlorides and Sulphates);
7. Sulphate content (acid extraction);
8. Ground water;
9. Soil Homogeneity (Horizontal);
10. Soil Homogeneity (Vertical); and
11. Structure Potential.

Having ensured the relevant parameters, the soil corrosivity or soil aggressiveness may be adequately determined.

The below Table is extracted from DIN 50 929-3 Table 2 (Page 5).

Table 3: Classification of soils according to soil aggressiveness and probability of free corrosion of unalloyed and low-alloy ferrous materials

B_0 or B_1 Value	Soil Category	Soil Aggressiveness*	Probability of Corrosion	
			Wide or Deep Pitting Corrosion	General Corrosion
≥0	Ia	Virtually non aggressive	Very Low	Very Low
-1 to -4	Ib	Weakly aggressive	Low	Very Low
-5 to -10	II	Aggressive	Medium	Low
< -10	III	Strongly Aggressive	High	Medium

B_0 = Free Corrosion in the absence of extensive concentration cells
B_1 = Free Corrosion in the presence of extensive concentration cells
* The soil aggressiveness corresponds to the probability of free corrosion in the absence of extensive concentration cells (see DIN 50 929-3 Sub clause 5.1.1)

The data relating to the soil analysed along the pipeline route in accordance with the requirements of DIN 50929-3 are detailed in Annexures A3.3 and B2.4 for centreline and borrow pits respectively. The data is summarised below.

The pH of the soils along the centreline is in the range of 5 to 8 and conductivity in the range of 0.006 to 0.111 S/m. The pH is typically slightly acidic along the route with an average value of 5.43, based upon the 19 test pits analysed in terms of the Probable Performance Index (PPI). The soil resistivity (inverse of conductivity) indicates that most of the soil is generally non corrosive, with the exception of TP C2/45A, C3/24A, C7/08A and C8/03A..pHs between 6 and 8 and the conductivity between 1,274 and 2,547 µS/cmThe sands from borrow sources have a pH ranging from 4.1 to 6.3 (with a single value of 8.15) and conductivity ranging between 6 and 94 µS/cm (with a value of 845 related to the high pH of 8.15).

Based upon the total DIN 50929-3 assessment, the route is generally deemed to be non corrosive, but specific “hot-spot” corrosion has been identified, predominantly in the vicinity of the ash dump, where leachate may traverse along the permeable selected backfill to the existing pipeline and permeate into the low points, thereby increasing the aggressiveness of the adjacent soil.

2.3.4 Stability of Trench Excavation

Caving of the sides of test pits occurred in only 2 test pits and none where seepage was encountered. Test pits were excavated with vertical sides and stood up for the time that it took to profile and sample them – usually between 10 and 30 minutes – before being backfilled. In extreme cases, when water is encountered, it may give rise to running sands which will only be stable at a slope of about 1:3 (V:H). In general, it is expected that, where no groundwater is present, sideslopes of 1:1 should be stable in the soils overlying bedrock. Bedrock should be stable if cut vertical. However, where over-break and disturbance of the sandstone in blocky and jointed sandstone results from blasting, collapse of blocks from the trench sides may occur. Where well cemented pedocrete is present, this may break out in slabs and care must be taken not to undercut the overlying sand.

Shear box tests were carried out in order to quantify the stability of the test pit walls. See discussion under Section 2.3.2 a) below.

2.3.5 Compactability

In general the sands identified for use as bedding/backfill to the pipe are fairly fine grained and their compactability factor (CF) is in the range of 0.27 to 0.44, but with the upper limit usually below 0.40. Sand with a CF between 0.1 and 0.4 is considered to be suitable, provided precautions (such as ensuring that backfill is maintained at the same level on both sides of the pipe and a minimum thickness of sand is placed over the top of the pipe before compaction commences) are taken around flexible pipes and where the sand may become saturated. Those with a CF >0.4 are considered unsuitable. However, there are no other sands in the area that are of better quality and it is recommended that even those which fall outside the limit of 0.4 be used, and proper control of the compaction process maintained.

Should it be required that only material with a CF <0.1 be employed, it will be necessary to import washed sands from commercial sources (which will be prohibitively expensive) or to wash and screen material from the borrow pits (which will also be expensive).

2.3.6 Granular Backfill Material

At the time of writing only limited geotechnical test results were available in the form of Shearbox tests. Triaxial, Hydrostatic Compression, Constrained Soil Modulus tests have been scheduled, but no results have yet been received. These must be analysed and interpreted in the future during further stages of the project.

2.3.7 Roots

The presence of roots, both in the soils along the centreline and in borrow pits, will require that these soils have to be screened before being used.

2.3.8 Geotechnical Testing

The following geotechnical tests were conducted:

- Shear box tests;
- Triaxial tests; and
- Constrained soil modulus tests.

None of the test results received to date has been analysed and their properties interpreted. This will have to be undertaken in the future. M_s testing is still in progress and these results will have to be included in the report and analysed and interpreted.

2.3.9 Supplementary Laboratory Testing

In addition to the conventional testing (grading and compactability), the following additional tests were conducted:

a) Sulphate Reducing Bacteria tests;
b) Fertility tests; and
c) Shear box, Triaxial and Constrained Soil Modulus tests.

a) Sulphate Reducing Bacteria (SRB)

All soils are biologically active, however, it is generally accepted that the anaerobic conditions that occur in clay soils promote Microbially Influenced Corrosion (MIC). MIC has become one of the most important forms of corrosion. In high resistivity soils where water may become entrapped in the bottom of the pipe trench, particularly where the pipe is installed in rocky areas, the entrapped water stagnates and becomes anaerobic (no oxygen) which permits micro-organisms to proliferate and attack the exposed steel at coating defects. The latter is also of importance in anaerobic clay soils, where MIC occurs. The latter often results in severe damage to buried pipes.

With the exception of BP7A and BP8 and BP24-TP23, the borrow pits are ostensibly free from SRB, as the sulphide levels are low and within acceptable values. BP7 and BP8 show elevated levels of sulphide, indicating that SRB are active and present in these soils. However soils from these borrow pits can be used as the pipeline will be protected with a cathodic protection installation.

b) Fertility

The results of the soil fertility testing have been included in the Annexures in Volume 2. They have however not been analysed and interpreted and these will have to be carried out in the future.

c) Shear box, Triaxial and Contrained Soil Modulus Test

To date Triaxial, Constrained Soil Modulus and Hydrostatic Compression tests have been done. The test results have been included in the Annexures in Volume 2. To date Shear Box tests have not been carried out.

2.4 Borrow Pit Investigation

2.4.1 Identification

In addition to sand sources for bedding material, gravel for use on haul roads and on existing gravel roads has been identified and are discussed in the following sections.

2.4.2 Bedding Material Borrow Sources

The location of bedding material borrow pits is shown on the Locality Plan included in Annexure C in Volume 2.4, and on Figure 2 in Volume 1. A volume of 100,000 m^3 and a spacing of 5 km for bedding material sources were targeted. This provides a redundancy of 100% of the bedding and backfill required for 5 km of pipeline (assuming a 2,000 mm diameter pipe and a 4 m deep trench) and ignores that suitable material may be excavated from the pipe trench. Thus, not all the borrow pits identified will necessarily be developed to their full extent, or used at all.

a) Distribution of Borrow Sources

There are adequate sources of bedding material available and they are well distributed along the route.

The exception to this are:

- A 6,000 m gap at the start of the Stage if BP53 does not prove to be usable (either from a suitability or accessibility point of view). BP53 is located at Ch. 3,300 m. Should it be possible to develop this source, this gap will disappear.
- A 7,200 m gap between BP49 (Ch. 20,400 m) and Steenbokpan (Ch. 27,600 m). Material from BP15 (opposite Ch, 25,500 m on Stage 4) will have to be utilised to minimise haul over this section.

The compactability factor (CF) for the various borrow sources identified are summarised on Table 4.

Table 4: Summary of compactability factors

Borrow pit no.	Compactability Factor (range)
53	0.30 - 0.34
52	0.32 – 0.41
50	0.36 – 0.41
48	0.31 – 0.44

Borrow pit no.	Compactability Factor (range)
49	0.20 – 0.36
15	0.34 – 0.39

From Table 4 it may be seen that in 50% of the cases the upper range of the CF is >0.40. Nevertheless, with proper control of compaction, these sources may be employed.

b) Quality of Borrow Sources

The sources identified are discussed separately.

BP53. This source has not been investigated in detail and only limited laboratory testing has been carried out on it. Based on these limited results, it appears that there should be about 50,000 m^3 of sand present. The sand is of good quality (A2-4, with a single A4 encountered) and mixing and stockpiling in the borrow pit should yield an A2-4 material which should be suitable for use in all bedding material types (SC2 = bed, bedding cradle and selected fill blanket).

Should it, for any reason, not be possible to utilise this source, material from BP52 and BP43 (the nearest source to the south on Stage 1) would have to be hauled in to fill this 8 km gap. The quantity of material available from these two sources should be sufficient to service this longer spacing between them.

BP52. This source contains more than the targeted 100,000 m^3 and is of good quality, generally an A2-4 but with a single A3 tested. Stockpiling (and mixing) in the borrow pit should yield an A2-4 material which is suitable for all bedding material types.

BP50. This source is of good quality (A2-4), should provide more than 100,000 m^3 and is located about 6,800 m north of BP52. The material is suitable for use as all bedding material types.

BP49. The material in this source is similar to that in BP52 and, after mixing and stockpiling in the borrow pit should yield an estimated 100,000 m^3 of A2-4 material suitable for use as all material bedding types.

BP15. This source is located on Stage 4 and is about 500m right/east of the pipeline route (opposite Ch. 25,500 m). It will have to be utilised to address the 6,7 km gap between BP49 (Ch. 20,900 m) and the end of Stage 4 in Steenbokpan (about Ch. 27,600). The source should yield about 100,000 m^3, so should have sufficient to provide material to meet the requirements of Stage 4 and to address the affective 4.2 km “gap” to the south of Steenbokpan, particularly when it is apparent that the thickness of the sand in this area ranges from 1 to 3 m thick and should be suitable for bedding sand. Details of BP15 are given in the Stage 4 report.

2.4.3 Laboratory Testing

a) Indicator, pH, Conductivity and Compactibility

The results of the laboratory testing are given in Annexure B, included in Volume 2.2.

The gradings of the bedding material sources show that some of the sources contain oversize material. The gradings of the various borrow sources and an indication of the proportion of oversize material are given in Table 5.

Table 5: Oversize material

BP no.	Ch.	Maximum size (mm)	Percentage to be scalped (% >9.5mm)	No. of TP with oversize / total TP tested
53+	2,300	13.2 – 9.5	3	1/7
52	6,000	26.5 – 19.0	5	1/21
50	12,800	53.0 – 37.5	1 to 10	2/19
48	16,500	53.0 - 37.5	2 to 10	3/28
49	20,400	<9.5	0	0/17
15	25,50+	37.5 – 26.5	19	1/19

+Only partly investigated

Note that in certain test pits, the gravel underlying the bedding material has been tested as a gravel source. These samples were not considered when evaluating the oversize.

In addition to the oversize material present, roots occur frequently in the test pits, often to the full depth of the test pits. It must be noted that the test pits were positioned to avoid trees and, even deeper and thicker roots may be present. Screening will thus be necessary to remove roots.

b) Geotechnical Testing

At the time of writing, the results of the geotechnical testing (Triaxial, Constrained Soil Modulus (Ms) and Hydrostatic Compression) was still in progress. Only Triaxial and Hydrostatic Compression test results for a single test pit on BP49 have been received to date and are bound into Volume 2. These have not bee analysed or interpreted. Subsequent test results will be filed electronically at MCC and will have to processed in the future.

2.4.4 Gravel Borrow Sources

Material for use in gravelling access or haul roads is required. The quantity of gravel required has been estimated at 81,000 m^3, based on the following assumptions:

- Public roads: 29 km, 8 m wide, 150 mm thick, gravelled twice (69,000 m^3);
- Private roads: 5 km, 5 m wide, 150 mm thick, gravelled once (4,000 m^3);

- Haul roads to BPs: 0.5 km, 6 m wide, 150 mm thick, gravelled once (500 m^3); and
- Haul roads parallel to pipeline: 27 km, 6 m wide, 150mm thick, gravelled once, 30% require gravelling (7,500 m^3).

No specific gravel borrow pits have been identified. Gravel is present below all of the granular backfill borrow pits and it is assumed that this gravel will be utilised for gravelling of roads. The gravel is fairly thin, but can be extracted once the sand has been removed. Where gravel is extracted from sand borrow pits, the sequence of extraction will have to be managed as the sand has to be removed to expose the gravel. The process will be further complicated if the borrow pit is to be used as a spoil dump.

Much of the route is on sandy materials and it is assumed that not all of these areas will require gravelling (30% has been allowed for above).

The estimated volume of gravel available from bedding material borrow pits is shown on Table 6.

Table 6: Gravel from bedding borrow pits

BP no.	Chainage	Estimated volume (m^3)	Gravel type
53	2,300	18,000	Ferricrete, sandstone
52	6,000	12,000	Ferricrete
50	12,800	8,000	Ferricrete
48	16,500	45,000	Ferricrete, sandstone
49	20,400	5,000	Ferricrete, sandstone
15$^+$	25,500$^+$	15,000	Ferricrete

$^+$ On Stage 4

Note: Estimated volumes probably conservative.

From Table 6 it appears that there is sufficient gravel available for gravelling purposes.

In general, the sands identified for use as bedding/selected backfill contain a fairly small percentage of oversize material and it will be necessary to screen the sand at the borrow pit to scalp off this oversize. The fine gravel scalped off can probably be used for gravelling of haul and access roads. A list of the borrow pits at which oversize material (and an indication of the likely extent thereof) appears as Table 7.

The distribution of the sources is shown on the Locality Plan included in Annexure C – Borrow Pit Plans in Volume 2.4 and on Figure 2: Regional Geology in Volume 1.

The suitability of the gravel for use as a wearing course on gravel roads has been assessed according to Draft TRH20: “The structural design, construction and maintenance of unpaved roads” (1990), and the results are shown on the following Table.

Table 7: Suitability of gravel as wearing course

BP no.	Ch.	Max. size (mm)	Oversize index	Shrinkage Product#	Grading Coeff.	TRH20 Class	Comments
53	2,300	26.5-19.0	0 (0)	193 –254	30	E	Good (but may be dusty)
52	6,000	53.0 - 37.5	0 - 4	0 - 42	22 – 42	B	Ravels & corrugates
50	12,800	37.5 - 26.5 53	0	0 - 92	21 -32	B	Ravels & corrugates
48	16,500	53.0 37.5	0 - 25	0 - 293	3 – 36	B/E	Good to ravels & corrugates
49	20,400	Nil tested					
15	25,500	37.5 – 26.5	0 15	0	5- -17	B	Ravels & corrugates
Recommended Standard		37.5	<5	100 - 365 (<240 pref.)	16 - 34		

Average value in brackets
Oversize index = % retained on 37.5 mm sieve
Shrinkage Product = Linear shrinkage x % passing 0.425 mm
Grading Coefficient = (% passing 26.5 mm - % passing 2 mm) x % passing 4.75 mm/100
Recommended Standard is for unpaved rural road.

From the table it is evident that the gravels present are generally suitable for use for gravelling of haul or access roads. The Shrinkage Product is generally low, reflecting a largely cohesionless (sandy) material, which will ravel and corrugate and be dusty. This is typical of the sandy materials found in the area. Blending with clayey soils in order to increase the plasticity (and hence the Shrinkage Product) is not be a viable option. Many of the gravel roads observed in the area have been built with similar gravels and show indications of ravelling and corrugations, even where vehicle counts are low.

2.4.5 Utilisation and Suitability

Borrow sources are well distributed over the Stage and their utilisation may be based on economic considerations. Once the analysis and interpretation referred to in 2.4.2.b) above have been completed, the utilisation and suitability should be re-assessed.

2.4.6 Mass Haul

As the pipe diameter is not known and the utilisation and suitability (see 2.4.3 above) have yet to be finalised, no accurate mass haul diagram can be generated at this stage. This will have to be generated at a later stage.

2.4.7 Spoil Sites

In addition to the borrow pits identified during the current investigations, there are very few old borrow pits along Stage 3 which may be used as spoil sites. Four potential sites have been identified and are shown on Figure 1 and on Table 8 in Volume 1. These show that the sites are generally small and the only large site (Site P) is located 7 km north of the pipeline route. No negotiations have been held with landowners to ascertain if these are available for use. An estimate of the volume of each site is given, but the exact volume of material that can be spoiled in each will have to be accurately measured. As no mass haul has been generated at this stage, it is not possible to comment on whether there is sufficient volume to accommodate the likely volume of spoil, but it appears that it will be necessary to utilise some or all of the current borrow pits as spoil dumps.

The location of the potential spoil sites is shown on Figure 1: Drawing number 2A-G3-020 in Volume 1 and the approximate volumes available at each in Table 8 in Volume 1.

2.4.8 Commercial Sources of Concrete Aggregate

The nearest commercial sources of concrete aggregate (coarse and fine) are located in the vicinity of Lephalale, about 50 km to the east. These sources have been discussed in detail in the Phase 1 and Stage 2 reports. Waterberg sandstone has been crushed at a site (Afrimat) about 15 km east of Lephalale for use during construction of the Medupi Powerstation. The crushed stone from this site was tested previously but was found to be too soft to meet SABS1083 specifications. If this site is still operative at the time of design of Stage 3, it may be worth resampling and testing the aggregate. The volume of concrete aggregate required on Stage 3 is limited (estimated at less than 2,000 m^3).

2.4.9 Corrositivity

An assessment of corrositivity of the borrow pit materials has not been carried out. This must still be done.

A similar assessment was conducted on several borrow pit (BP) sites and the results thereof are also detailed in Annexure B. In essence most of the BPs contain suitable “backfill” with the sole exception of BP8-TP19 and BP24-TP23. The former BP data is not in-line with the pH, conductivity, sulphide values obtained at the other TP on BP8, suggesting that there is an outcrop of corrosive soil. Similarly results from BP24-TP23 are discarded and therefore no further comment is warranted.

2.5 Site Specific Investigations

No large structures are planned along Stage 3 and thus no site specific investigations have been carried out. In this regard it must be noted that no investigation has been carried out at the "end-user” reservoir that will be required at Steenbokpan.

3 SUMMARY AND CONCLUSIONS

3.1 Future Investigations

Various aspects will require additional investigation or evaluation in the future. These are:

- Re-interpretation of the data supplied in this report to address the finally selected pipe diameter and trench dimensions;
- Prove BP53 in detail once access has been arranged;

- Re-evaluation of the distribution and size of borrow pits, taking into account the pipe diameter (and associated trench dimensions);
- Preparation of a preliminary mass haul diagram to aid in the optimal distribution of any additional borrow pits that may be required and the identification of adequate and suitably located spoil sites;
- The feasibility of using existing borrow pits as spoil dumps and environmental constraints related to this;
- Negotiations with landowners to allow the use of existing borrow pits as spoil sites;
- Preparation of a final mass haul diagram once all borrow pits and spoil sites have been finalised, Investigation for retention dams at scour points along the pipeline. The location and size of these have not yet been finalised;
- The evaluation of likely blasting methods;
- Should the Alternative Alignment along Road D175 in the south be followed, an additional borrow source must be sought in this area to address the 8.6 km gap along this section;
- Investigation of the geotechnical conditions at any scour points that may be present along the route; and
- Interpretation of Ms and Triaxial test results.

3.2 Summary

The geotechnical conditions along the Stage are similar and may be summarised as follows:

a) The underlying geology comprises Waterberg Sandstone over the whole route. No outcrops were observed and bedrock was only encountered in test pits.

b) Refusal usually occurred on ferricrete or calcrete. The latter may in fact represent weathered sandstone.

c) Soils are generally sandy and suitable for re-use in the pipe trench.

d) The topography is subdued with scattered pans providing the only features in the area. Some pans are not perennial, but all are expected to hold water following summer rains.

e) Slight seepage was encountered in only 3 test pits.

f) In only 2 test pits was caving of the sides encountered. It must be borne in mind that, due to their limited length, test pits are likely to show better stability than long pipeline trenches.

g) The soils along the route are sandy and those from the trench are probably re-usable for backfilling.

h) The quality of granular material from borrow sources is good and is suitable for use in all backfill applications. The sources are well distributed along the route.

i) Concrete aggregate will have to be hauled in from Lephalale where it is available commercially.

j) There are no natural sources of water for construction purposes along the route and water will have to be hauled to site, or boreholes drilled to provide water. It is assumed that landowners are unlikely to agree to the extraction of water from pans. Should this be negotiated, the quality of the water will have to be determined.

k) Commercial hunting farms are present along most of the route and the effect of construction on these hunting activities should be noted when programming operations, particularly blasting.

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ANNEXURE A

DIN 50929-3 SOIL AGGRESSIVENESS (NO RESULTS)

ANNEXURE B

LAND CAPABILITY RATING (CHAMBER OF MINES)

Criteria for Wetlands

- Land with organic soils or supporting hygrophilous vegetation where soil and vegetation processes are water determined.

Criteria for Arable Land

- Land, which does not qualify as a wetland.
- The soil is readily permeable to a depth of 750 mm.
- The soil has a pH value of between 4,0 and 8,4.
- The soil has a low salinity and SAR.
- The soil has less than 10 % (by volume) rocks or pedocrete fragments larger than 100 mm in the upper 750 mm.
- Has a slope (in %) and erodibility factor ("K") such that their product is <2,0.
- Occurs under a climate of crop yields that are at least equal to the current national average for these crops.

Criteria for Grazing Land

- Land, which does not qualify as wetland or arable land.
- Has soil, or soil-like material, permeable to roots of native plants, that is more than 250 mm thick and contains less than 50 % by volume of rocks or pedocrete fragments larger than 100 mm.
- Supports, or is capable of supporting, a stand of native or introduced grass species, or other forage plants utilisable by domesticated livestock or game animals on a commercial basis.

Criteria for Wilderness land

- Land, which does not qualify as wetland, arable land or grazing land.