# REPORT ON A GEOTECHNICAL INVESTIGATION FOR THE PROPOSED NEW ESKOM GENERAL LANDFILL AND HAZARDOUS WASTE FACILITY, LEPHALALE

## 1. TERMS OF REFERENCE

This report has been prepared at the request of Envirolution Consulting (Pty) Ltd and forms part of the specialist studies required for an Environmental Impact Assessment (EIA). It presents the results of a geotechnical investigation carried out within an area identified for the development of a general landfill and hazardous waste storage facility for Eskom near the town of Lephalale in the Limpopo Province.

Since the investigation forms a part of the specialist studies for the EIA for the facility, it has been undertaken in order to meet with the requirements of Chapter 6 of the document *Minimum Requirements for Waste Disposal by Landfill*[(1)]. This chapter deals with the *Site Investigation*, however, two sections of the chapter, namely 6.3.3 *Geohydrology* and 6.4 *The Geohydrological Report,* are not included in this report as they form part of a Geohydrology study conducted by Blue Rock Consulting (Pty) Ltd, the results of which are contained in a separate report prepared by them.

## 2. SCOPE OF WORK

Based on the requirements of the aforementioned document, the investigative work has entailed establishing the physical geography of the area including the:

- Topography;
- Infrastructure;
- Climate and
- Vegetation.

In addition to the above, the report discusses the geological and geotechnical conditions underlying the area and provides information on the:

- Geology;
- Soil conditions and;
- Miscellaneous issues.

## 3. INFORMATION SUPPLIED

Information supplied during the course of carrying out the investigation included:

- A report titled *Groundwater Pollution Risk Assessment and Compliance Monitoring at Matimba Power Station*, prepared by Frank Hodgson and dated February 1995.

- A report titled *Groundwater Quality and Pollution Plume Modelling at Matimba Power Station* by Riaan Grobbelaar, Lore-Mari Cruywagen, Elna de Necker and Frank Hodgson, dated October 2000.

- A report dated 2006 and titled *Interpretation of Groundwater Results Matimba Power Station Compiled* by Danie Vermeulen of the Institute for Groundwater Studies at the University of the Free State.

- Various maps, plans and photographs of the area.

## 4. BACKGROUND TO THE DEVELOPMENT

Whilst the Medupi Power Station is being constructed waste will be generated until it has been completed in about 2014. Approximately half of this will be hazardous waste and the other half general waste. It is expected that existing Matimba Power Station will also generate the same amount of waste and in similar proportions. Also, the proposed two Waterberg Coal Fired Power Stations are likely to generate waste volumes slightly higher than that for the Medupi Power Station. All of these power plants have a design life of 50 years and the total waste generated from them over this period is expected to be of the order of 1 200 000 $m^3$, split equally between general and hazardous waste.

The approximate waste volumes generated from the power stations will require that Eskom provides a suitable waste management strategy and appropriate facilities. The latter will include access roads, weigh bridges, water pipelines for potable water, the waste disposal site, distribution lines for providing electricity to the facility and buildings such as offices.

## 5. THE SITE

As alluded to above, the proposed landfill will be designed so as to accommodate both general and hazardous waste from the Medupi and Matimba Power Stations, as well as the proposed future Waterberg Power Stations and the nearby Eskom construction village in Maropong.

The site selection process identified the following potentially suitable candidate sites:

- Four sites on the farm Grootvallei 515 LQ located adjacent to the Medupi Power Station, which is currently under construction.

- The farm Grootestryd located within the Matimba Power Station.

Following a site selection process during the scoping exercise, the site located within the Matimba Power Station on the farm Grootestryd was considered the most favourable. Within this preferred area a further three sub-sites were identified as being suitable.

## 6. INVESTIGATION

The geotechnical investigation commenced with a desk study, which entailed obtaining as much information as possible of the site that may provide an indication of the most likely subsoil and associated geotechnical conditions prevailing within the area. By determining the underlying geological setting together with the prevailing topographical and climatic conditions, the weathering characteristics of the host rock can be estimated and an indication of the most likely geotechnical conditions underlying the site established. The information obtained from the desk study is discussed in Sections 7 and 8 of this report.

The desk study was followed by a site reconnaissance which was carried out on Wednesday 17$^{th}$ December 2008 and entailed visiting the site and walking over the entire area whilst noting and recording information from visible surface features. Information from this phase of the investigation, together with the desk study, provided a preliminary assessment of the geotechnical conditions underlying the site and identified areas necessary for further investigation.

Following the desk study and reconnaissance survey described, the field investigation was carried out on 18 to 20 March 2009 and entailed setting out and excavating twenty trial holes employing a CAT 416 Tractor-Loader-Backhoe (TLB) excavator. The holes, referenced TH1 to TH20, were excavated at the positions shown on the attached site plan, Figure 1, and were positioned so as to cover the footprint of the entire site.

The test pits were excavated to an average depth of 2,0 m ranging from 0,7 to 3,8 m below the present ground surface, and each of the holes was profiled in accordance with the standard methods prescribed in the document *Guidelines for Soil and Rock Logging in South Africa (1990)*[(2)] prepared by the Geotechnical Division of the South African Institute of Civil Engineers and the Association of Engineering Geologist of South Africa.

The soil profiles are attached as Appendix A with this report and the subsoil conditions discussed in Section 8.

## 7. GEOGRAPHY

Based on the information acquired from the Desk Study phase of the investigation, described in Section 5 of this report, the information discussed below was obtained.

### 7.1 Topography

The site investigated covers an area of some 41 hectares, which includes the existing landfill facility, and is located in the western corner of the premises of the Matimba Power Station. According to the 1:50 000 topographical map referenced DA Ellisras, the ground surface was originally fairly flat. However, it is presently undulating being covered with mounds of fill.

A disused waste facility occupies the central south western sector of the site. It covers an area of some 11 hectares and comprises a perimeter earth wall within which ash, rubble and other waste has been discarded.

Figure 7.1 Existing waste facility on the site

**7.2 Infrastructure**

Infrastructure in the area is limited to a buried high voltage cable which runs alongside about two thirds of the length of the boundary fence on the south western perimeter to the site. A mini substation is located in the western corner.

A disused water pipe installed by ISCOR runs alongside and over the full length of the north western boundary to the area, and a gravel surfaced road follows the electric fence which forms the south western and north western limits to the site.

**7.3 Climate**

Summer in the region extends from mid-October to mid-February and is characterised by hot, sunny weather often with afternoon thunderstorms. Winter occurs from May to July and is typified by dry, sunny, days and cold nights. The monthly distribution of average daily maximum temperatures range from 22,3℃ in June to 31,9℃ in January. The region is the coldest during July when temperatures drop to a minimum of 3,7℃ at night.

Lephalale receives some 400 mm of rain per year, with most of it occurring in mid summer. It receives the lowest rainfall of 0 mm in June and the highest averaging 81 mm in January.

Based on Weinert's N map[3], the area falls within the region where N is about 4,5 which is at the border between mechanical and chemical weathering of rock and suggests that a deep soil profile in the area is unlikely.

### 7.4 Vegetation

The site has been extensively disturbed and as a result in many places covered by alien vegetation and weeds. Dense groves of mature acacia and other trees are, however, present over parts of the site and illustrated below.

Figure 7.2: Typical vegetation covering the site

## 8. SUBSURFACE CONDITIONS

### 8.1 Geology

Based on the 1:250 000 geological map, an extract of which is attached below, the site located on the farm Grootestryd is underlain by rocks of the Karroo sequence comprising sandstones, gritstone, mudstones and coal.

Frequently the rocks mentioned, or their weathered derivatives, are overlain by transported Quaternary deposits including windblown Kalahari sand and pedogenics, mainly as calcrete.

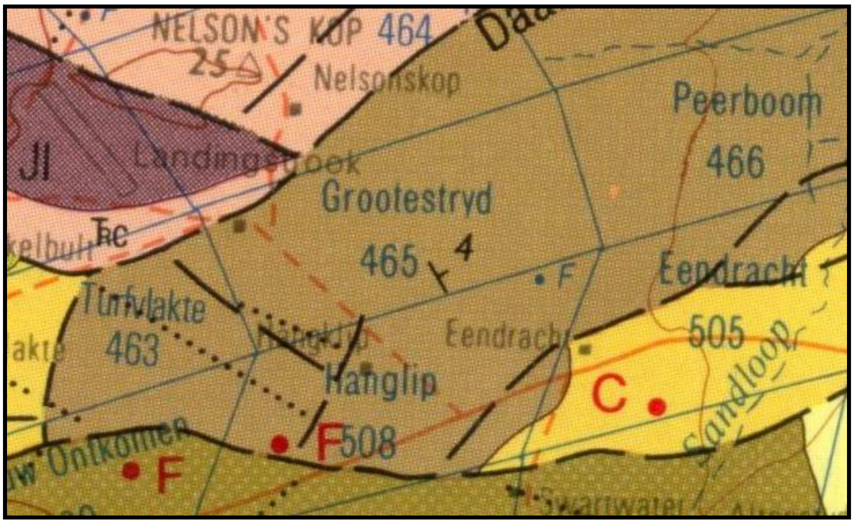


**Figure 8.1**
Geological map of the farm Grootestryd underlain by rocks of the Karroo Supergroup: sandstone/gritstone/mudstone/coal

### 8.2 Subsoils

The subsoils encountered within the trial holes include the following:

8.2.1 Fill: occurs over most of the site and comprises largely of very loose to loose sand with sandstone gravels, cobbles and numerous boulders. The fill extends to depths ranging from 0,2 to 1,8 m below the present ground surface averaging 0,9 m. In the vicinity of trial hole TH14, put down on top of the existing waste dump indicates that ash is also present within the fill material.

8.2.2 Topsoil: covers the site to an average depth of 0,2 m and comprises a dark brown and brown, very loose to loose, fine sand containing many roots.

8.2.3 Aeolian sand: underlies the fill or topsoil consists of brown and orange, mainly very loose to loose although occasionally medium dense to dense, fine sand with occasional roots. In places scattered calcrete concretions occur with depth. It extends to an average depth of 2,0 m in the range 0,35 to 3,5 m below the present ground surface.

8.2.4 Ferricrete: underlies the aeolian sand in places and where encountered is predominantly nodular and of medium dense consistency, however, hardpan ferricrete of soft rock consistency is also present in the vicinity of trial hole TH8. The ferricrete is of an average thickness of 0,2 m in the range 0,1 to 0,4 m.

8.2.5 Calcrete: often underlies the aeolian sand at an average depth of 1,8 in the range 1,0 to 2,8 m and is predominantly dense and nodular. Hardpan calcrete is also present in the soil profile in the vicinity of trial holes TH6 and TH9 as very soft to soft rock.

8.2.6 Residual sandstone: was encountered in hole TH11 only where it underlies the aeolian sand at a depth of 0,35 m below surface. It is slightly moist, comprising fine sand of medium dense to dense consistency.

8.2.7 Shale: occurs in the vicinity of hole TH13 only where it is a grey, yellow and orange, highly weathered, closely jointed, soft rock. It breaks into flaky gravels with a little sand and its apparent dip is towards the east.

The trial holes refused at depths ranging from 0,7 to 3,8 m, below the present ground surface mostly on dense to soft rock calcrete or ferricrete, or large boulders in a sand fill matrix.

8.2.8 Ground water. No water seepage was encountered in any of the trial holes, however very moist conditions were observed in hole TH10.

**8.3 Laboratory Tests**

The results of laboratory tests carried out on selected soil samples recovered from the trial holes are summarised below and presented in Appendix B of this report.

8.3.1 Indicator Tests

For more accurate identification and for classification purposes, particle size distribution analysis and Atterberg limit determinations were carried out on samples of the aeolian sand and the calcrete nodules, results of which are summarized in Table 8.1 below.

In terms of the Unified Soil Classification system the aeolian sand classifies mainly as a "SM" soil type, these being silty sands and poorly graded sand-silt mixtures. Its grading modulus averages 0,93 in the range 0,89 and 0,99 which reflects its fairly fine nature. It is also no-plastic or slightly plastic which indicates it has little plastic fines.

Table 1: Summary of results of indicator tests

| Hole No | Depth (m) | LL | PI | $PI_{ws}$ | LS | GM | MIT Size Fraction - % | | | | Classification | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | Gravel | Sand | Silt | Clay | USC | AASHTO |
| TH1 | 0.1 – 1.0 | SP | SP | SP | 1 | 0.89 | 1 | 76 | 11 | 12 | SM | - (0) |
| TH4 | 0.4 – 1.6 | 43 | 20 | 6 | 8 | 2.18 | 61 | 27 | 9 | 3 | GC | A-2-7 (0) |
| TH5 | 0.7 – 0.9 | NP | NP | NP | 0 | 0.93 | 0 | 84 | 8 | 8 | SM | A-2-4 (0) |
| TH6 | 0.3 – 0.5 | NP | NP | NP | 0 | 0.99 | 0 | 84 | 8 | 8 | SM | A-2-4 (0) |
| TH10 | 0.7 – 3.0 | NP | NP | NP | 0 | 0.89 | 1 | 79 | 11 | 10 | SM | A-2-4 (0) |
| TH13 | 1.8 – 3.2 | SP | SP | SP | 1 | 2.29 | 64 | 28 | 7 | 1 | GP-GM | A-1-a (0) |
| TH20 | 0.1 – 1.5 | NP | NP | NP | 0 | 0.95 | 1 | 85 | 9 | 5 | SM | A-2-4 (0) |

LL = liquid limit; PI = plasticity index; $PI_{ws}$ = plasticity index of whole sample; LS = linear shrinkage; GM = grading modulus, USC = unified soil classification; AASHTO = American Association of State Highway and Transportation Officials

The calcrete gravels classifies as "GC" and "GP-GM" which are clayey gravels or gravel-sand-clay mixtures and poorly graded silty gravels or gravel-sand mixtures. Grading moduli range from 2,18 to 2,29 indicating the relatively coarse nature of the material. Plasticity indices range widely from slightly plastic to 20, which is typical of calcretes.

Based on the indicator tests, the sand is considered to be of fair workability as a cover material, semi pervious and highly erosive.

8.3.2 Consolidometer Tests

Consolidometer tests were carried out on undisturbed specimens of the aeolian sand and results are summarized in Table 8.2 below.

Table 8.2: Summary of results of consolidometer tests

| Hole No | Depth (m) | nmc (%) | $\rho_d$ (kg/m$^3$) | $S_r$ (%) | $C_c$ | $C_s$ | Collapse - % @ 100 kPa |
|---|---|---|---|---|---|---|---|
| TH5 | 0,7 – 0,9 | 3,3 | 1672 | 14,9 | 0,017 | 0,010 | 5,2 |
| TH6 | 0,3 – 0,5 | 4,7 | 1725 | 23,0 | 0,007 | 0,007 | 0,9 |

nmc = natural moisture content; $\rho_d$ = dry density; $C_s$ = Swell index; $C_c$ = Virgin Compression Index.

It is evident from the tests that the dry density of the material is moderate. Interpolated collapse strains at 200 kPa pressure are estimated to be of the order of 1,6 % and 6,7 % which, together with its pinholed structure, indicates it is of “moderate problem” to “trouble” according to the classification for collapse problems proposed by Jennings and Knight[(4)].

8.3.3 Compaction Tests

Modified AASHTO compaction tests and California Bearing Ratio (CBR) tests were carried out on the aeolian sand and calcrete nodules, the results of which are summarised in Table 8.3 below.

Table 8.3: Summary of results of compaction and CBR tests

| Hole No | Depth (m) | Description | $\rho_d$ max (kg/m$^3$) | omc (%) | CBR @ % compaction | | | | | Max Swell (%) |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 90 | 93 | 95 | 98 | 100 | |
| TH4 | 0,4 – 0,6 | Nodular calcrete | 1972 | 11,9 | 27,8 | 34,3 | 39,5 | 51,2 | 61,3 | 0,2 |
| TP10 | 0,7 – 3,0 | Aeolian sand | 2122 | 7,3 | 5,2 | 10,1 | 15,7 | 31,5 | 50,5 | 0,0 |

$\rho_d$ max = maximum dry density; omc = optimum moisture content; CBR = California Bearing Ratio

The aeolian sand and nodular calcrete have maximum dry densities of 2 122 and 1 972 kg/m$^3$ respectively at optimum moisture contents of 7,3 and 11,9. CBR tests carried out on these soils indicate that they classify as G8 and G6 materials respectively in accordance with the TRH 14 materials specifications [(5)].

8.3.4 Permeability Tests

Permeability tests were carried out in the laboratory and field soakaways were undertaken in the trial holes to arrive at an order of magnitude of permeability coefficients. The results are summarised in Table 8.4 below and show that for the field tests the aeolian sand and the calcrete gravel generally have permeability coefficients of the order of $10^{-5}$ and $10^{-6}$ metres per second respectively.

Table 8.4: Summary of results of permeability tests

| Material | Test Type | Depth (m) | Moisture Content (%) | | Dry Density (kg/m$^3$) | Coefficient of Permeability (m/s) | | |
|---|---|---|---|---|---|---|---|---|
| | | | Before Test | After Test | | Range | | Average |
| | | | | | | Minimum | Maximum | |
| Sand | Field Test | 0.0 – 0.7 | - | - | | 5.2E-06 | 1.3E-05 | 9.7E-06 |
| Calcrete | Field Test | 0.0 – 0.3 | - | - | | 1.1E-06 | 7.0E-06 | 3.8E-06 |
| Calcrete | Lab Test | 0.4 – 0.6 | 14.3 | 21.6 | 1812 | 1.2E-07 | 1.5E-07 | 1.3E-07 |
| Sand | Lab Test | | | | | | | |

## 8.4 Miscellaneous Subsurface Issues

### 8.4.1 Undermined Areas

The site is not undermined and coal mining extends some distance west of the boundary to the site.

### 8.4.2 Earth Tremors

According to Fernandez and Guzman(6) the area investigated is classified as having a seismic intensity of about VI on the modified Mercalli scale (MMS) with a 90% probability of not being exceeded during a 100-year recurrence period.

### 8.4.3 Rehabilitated Open-Cast Mines

There are no open-cast mines in the vicinity of the site.

### 8.4.4 Potential for Future Mining

No coal or other mineral reserves underlie the site and so the potential for future mining in this area is remote.

### 8.4.5 Sinkholes and Surface Subsidences

The geological conditions underlying the site do not lend themselves to the formation of sinkholes or surface subsidences such as dolines.

## 8.5 Potential for Landfill Gas and Air Quality Problems

Since most of the landfill will comprise rubble and general builders waste the potential for the significant development of gasses is unlikely as is that of air pollution.

# 9. DISCUSSION

## 9.1 Subsoil Conditions

The subsoil conditions underlying each of the sub-sites A, B and C as reflected in the test pits excavated within these areas and illustrated in Figure 2, is summarised in Table 9.1 below. The calcrete and ferricrete horizons were frequently not penetrated and hence these layers have not been shown.

Table 9.1: Summary of soil conditions within sub-sites

| Soil Type | Soil Thickness (m) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Site A | | | Site B | | | Site C | | |
| | Mean | Min | Max | Mean | Min | Max | Mean | Min | Max |
| Fill | 0,05 | 0,25 | 0,25 | 0,75 | 0,70 | 1,60 | 0,30 | 0,20 | 0,20 |
| Topsoil | 0,17 | 0,10 | 0,35 | 0,05 | 0,20 | 0,20 | 0,18 | 0,20 | 0,30 |
| Sand | 0,92 | 0,65 | 1,95 | 0,18 | 0,70 | 0,70 | 2,28 | 1,30 | 3,30 |

The table illustrates that most of the fill underlies Site B with lesser amounts underlying Site C. Cognisance should, however, be taken of the presence of mounds of fill covering the area in general and sites A and B in particular.

A substantial thickness of sand underlies Site C with very little in the vicinity of Site B, and estimated quantities of available material from the area, listed in Table 9.2 below, suggests that cover material as the aeolian sand in available from Sites A and C.

Table 9.2: Summary of soil volumes available

| Soil Type | Volume ($m^3$) | | |
|---|---|---|---|
| | Site A | Site B | Site C |
| Fill | 2 500 | 37 500 | 1 667 |
| Topsoil | 8 500 | 2 500 | 9 167 |
| Sand | 46 000 | 8 750 | 113 750 |

## 9.2 Environmental Impacts

Typical geotechnical impacts that may affect the development are tabulated below together with mitigating measures. Again, the effect on the ground water regime is discussed in another report.

Table 9.3: Geotechnical Impacts for all sub-sites

| Impact | Mitigation measures |
|---|---|
| Leachate seepage through porous soil cover into groundwater. | Impermeable lining with leachate detection system to be provided beneath landfill. |
| Availability of cover material. | 200 000 $m^3$ aeolian sand available from within the overall site. |
| Insufficient quantity of cover material. | Kalahari sand is ubiquitous in the region and borrow pits will have to be identified if volume on site is insufficient. |
| Potentially collapsible sand blanketing the site. | Raft foundations or stiffened footings will have to be provided for all buildings. |
| Suitability of *in-situ* material for access road. | The aeolian sand when compacted provides material of G8 quality. A base course and riding surface will have to be provided. |

**9.3 Site Selection**

The selection of the most appropriate of the three sub-sites, as far as the subsoils and bedrock conditions are concerned, has been carried out utilising the Site Ranking Matrix presented in Table 9.4 overleaf. The effect of these sub-sites on the ground water regime has been addressed in another report prepared by Blue Rock Consulting (Pty) Ltd. and is not discussed here.

Non of the sub-sites exhibit fatal flaws in so far as unstable areas, steep slopes, shallow bedrock or pans and vleis are concerned, and so these are not discussed further. The environmental impacts and mitigating measures discussed in 9.2 above apply equally to the three sub-sites.

Following is an explanation of the ranking matrix presented below.

*Suitability for extension:* The possibility exists that land may be required to increase the capacity of the landfill in the future. Ideally this expansion should take place adjacent to the existing facility since the infrastructure such as roads, weigh bridges and offices will be in place. Site A can be extended to the south, land is available to the north and south of Site B for expansion, and little land is available for expansion of Site C. Site B is therefore the more preferable for future expansion.

*State of the site*: reflects the degree of disturbance to which the site has been subjected to in the past. Sites A and B have been extensively disturbed, whilst Site C appears to have been less so. Ideally, development on a disturbed site is preferable to that on an undisturbed site suggesting that Sites A and B are preferable for the development.

*Soil depth:* is the thickness of soil available for use as cover material during operations and at closure. Sites A and C have approximately 160 000 $m^3$ of sand available respectively, whilst Site B is blanketed by fill comprising sand and boulders. Ideally the landfill should not be placed within a depression or an excavation lower that the surrounding ground, since water can collect in it. This situation arises when the cover material is excavated from beneath the foot print of the landfill. The optimal sighting of the facility is therefore at ground level and sourcing some of the cover material form a nearby location such as Site A and C.

*Soil quality:* reflects the suitability of the available material for use as cover. As discussed previously, the aeolian sand blanketing Sites A and C classifies as a fine silty sand with little, if any, plastic binder. It is considered to be of fair workability as a cover material and its permeability coefficient indicates that it is semi pervious. It is however, considered to be highly erosive due to the lack of plastic fines.

No suitable cover material underlies Site B within the depth range investigated where it is blanketed by fill.

*In-situ permeability:* is the ease with which water seeps through the underlying surface soil and into the ground water. All of the sites are blanketed by fairly permeable aeolian sand or fill to depths of between 0,7 and 2,3 m. The calcrete that invariably underlies the sand is marginally less permeable but still occurs within the sub-sites. The ranking for the three sub-sites is therefore of the same order.

9.4: Landfill Site Ranking Matrix - Geotechnical

| Candidate Site | Suitability for expansion | State of Site | Soil | | | Total |
|---|---|---|---|---|---|---|
| | | | Depth | Quality | *In-situ* permeability | |
| A | 1 | 1 | 1 | 1 | 0 | 4 |
| B | 2 | 1 | 2 | 0 | 0 | 5 |
| C | 0 | 0 | 1 | 1 | 0 | 2 |

Employing the ranking used to arrive at the most favourable site, it can be concluded that sub-site B is the more suitable for the reasons discussed above.

## 10. CONCLUSIONS

Based on the profiles of twenty trial holes excavated within the area proposed for the development of the general landfill and hazardous waste facility within the Matimba Power Station, together with results of a range of laboratory tests, the following can be concluded:

- The site has been extensively disturbed and mainly blanketed by fill.
- The area is underlain by windblown sands which are potentially collapsible, semi-pervious and erodible.
- Sub-sites A and C have extensive sand deposits whilst sub-site B is mainly underlain by thick fill deposits.

In view of the above main findings the presence and nature of the windblown sand covering most of the site suggests that it should be used as cover material for the facility. Since Site B has mostly fill underlying it, it is recommended that it be selected as the preferred sub-site, however, all sites are equally suitable for the development of the facility.

The permeable nature of the sand and fill blanketing the area will require that the facility be lined to prevent seepage of leachate into the regional ground water.

The potentially collapsible nature of the sand will also require that special foundation precautions be implemented to address the possibility of settlement occurring to buildings founded within it.

## 11. REFERENCES

1) *Minimum Requirements for Waste Disposal by Landfill.* Department of Water Affairs and Forestry Republic of South Africa. 2nd edition 1998.

2) *Guidelines for Soil and Rock Logging in South Africa.* Association of Engineering Geologists, Geotechnical Division of the South African Institute of Civil Engineering and South African Institute of Engineering Geologists. Geoterminology Workshop 1990.

3) *The Natural Road Construction Materials of Southern Africa.* H. H. Weinert, H & R Academia (Pty) Ltd. 1980.

4) Jennings, J.E. and Knight, K. *A guide to construction on or with materials exhibiting additional settlement due to collapse of grain structure.* Proceedings, 6th Regional Conference for Africa SM and FE. Durban. 1975.

5) Technical Recommendations for Highways TRH 14: *Guidelines for Road Construction Materials*, Committee of State Road Authorities. 1989.

6) Fernandez L. M. and Guzman J. A. *Earthquake hazards in Southern Africa.* Seismological Series 10, Published by Geological Survey of South Africa 1979.

B A Harrison Pr Eng April 2009
for Inroads Consulting cc

# APPENDIX A

## SOIL PROFILES

# APPENDIX B

## LABORATORY TEST RESULTS