AUGUST 2017

GEO-TECHNICAL INVESTIGATION FOR THE FOUNDATION DESIGN OF A PROPOSED PUBLIC TRANSPORT FACILITY ON ERF RE/7/239-IQ IN EMDENI, SOWETO, GAUTENG PROVINCE.

FINAL REPORT

Report No. JOB MS201714

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EXECUTIVE SUMMARY

Client	Johannesburg Development Agency (JDA)
Consultant	Mshandukani Holdings (Pty) Ltd.
Site location	Erf RE/7/239-IQ in Emdeni, Soweto, Gauteng.
Purpose of investigation	The purpose of the investigation is to recommend the foundation design for the proposed public transport facility at Erf RE/7/239-IQ in Emdeni, Soweto.
Regional geology	According to the available official geological map covering the site area 1: 250 000 scale geological sheet numbered 2626 West Rand Geological Series. The site is underlined by Basaltic lava, Agglomerate and Tuff of the Klipriviersberg Group of the Ventersdorp Super-group. (Geology map is shown in Appendix C, Fig.2)
Excavation conditions	Intermediate excavation conditions (as per SANS 1200 D) generally prevail on site. Excavation of material was achieved using a TLB.
DCP test results	The in-situ CBR values of the soil layers are **34%** at an average depth of **0.4 m** for all the test pits. DCP results deeper than 1.5m cannot be relied on due to increased frictional resistance of the testing rods hence the tests were limited to 1.0m.
Laborotaty Results	Based on the laboratory results, the overall site maximum allowable bearing pressure of soil was calculated at **26.16 kPa** on the residual Basaltic lava. Laboratory results also indicate that the material has a minimum bulk density of **1577kg/m^3** and a maximum bulk density of **1646kg/m^3** with an average optimum moisture content of **19.1%.** The maximum soaked CBR value is **1%** at 95% MODAASHTO.
Foundation depth	The top layer of colluvium cannot be used as layer works material during construction. Therefore the foundation designs should be founded at an average depth of **0.4m** on the residual material of Basaltic lava.

NHBRC classification	Based on NHBRC classification can be classified as **R-H1; P (Imported fill)** with a maximum of 7.5-15mm shrinkage is expected.
TRH14 classification	Based on TRH 14 of Material Classification this material can be classified as **G6 at 0.4m.**
Soil bearing pressure	Allowable soil bearing pressure of **271 kPa** for in-situ CBR values and **26.16 kPa** for soaked CBR values (taken at TP 1 on residual Basaltic lava).
Recommendations	**For Commercial Facility purposes:** **Normal Reinforced Strip Footing foundation** it is therefore recommended for the ablution block building where In-situ reconstruction or ripping is done at an average depth of **0.4m** below surface on residual material of Basaltic lava. Proper compaction should be adhered to while back filling the trenches and foundations with G5 material and should be compacted to 95% MODAASHTO at 150mm intervals. **For Transport Purposes:** 1. **Pad or Spread Footing foundation** is therefore recommended (for the steel columns that will carry the roof structure) at an average depth of **0.4m** on Residual material of Basaltic lava. 2. **Pavement recommendations:** In-situ ripping should be at an average depth of at **0.4m** on residual material Basaltic lava. The estimated traffic or vehicle per day is <75 vehicles and <5 heavy vehicles per day with a total loading traffic of <0.3x106E80s per lane (Guideline for human settlement planning and design Vol.2). ➢ **Surface:** Pavement bricks can be placed on the surface for walking lanes, and asphalt concrete can be used for a smooth finish along the taxi parking lanes. The binder content present with asphalt acts

	as a lubricant when hot and as an adhesive and water proofing when cold. ➢ **Base (~150mm):** G5 material should be compacted to 95% MOD AASHTO density at 150mm interval. ➢ **Sub-base (~250mm):** G3 material should be compacted to 98% MOD AASHTO density at 150mm interval. Based on the seepage encountered on site, Exterior drainage and foundation wall damp-proof coatings; Capillary breaks at footings and at the top of the foundation wall, Insulation, air barrier and water vapor control, air barrier and thermal insulation systems should be installed to prevent groundwater water from infiltrating; such an impermeable layer can assist as a sealing layer.

The comments and recommendations contained within this report are based on a limited number of test pits, DCP's and laboratory results. It is therefore recommended that all excavation and foundation trenches be inspected by a geotechnical engineer or engineering geologist during construction to verify that the founding condition are not at variance with those described herein.

1. INTRODUCTION AND TERMS OF REFERENCE

Mshandukani Holdings was appointed by Johannesburg Development Agency (JDA) to conduct a Geotechnical investigation for foundation design for the proposed construction of a public transport facility at Erf Re/7/239/-IQ Emdeni, Soweto, Gauteng Province.

The investigation was aimed at assessing materials and establishing the specific site geotechnical conditions and recommendation.

The following are some of the objectives of geotechnical investigations:

- Specific Geology of the Site.
- Potential Geotechnical Restraining Factors.
- Excavation Conditions.
- Presence of Groundwater.
- Classification of the Site Material According To the TRH14 Classification System.

A 1:250 000 geological map of the area was studied prior to the investigation taking place.

2. THE SITE

2.1 Site Location

The proposed site for development is located at Erf Re/7/239-IQ in Emdeni, Soweto, Gauteng. The central coordinates of the site are Latitude 26.242387°S and Longitude 27.839793°E at an average elevation of 1640 m above mean sea level.

2.2 Climate

The average high during the summers is 25 °C although it can go as high as 37 °C. A record high of 40 °C has been recorded. Average high temperatures are around 16.7 °C and the average lows are 5 °C and – 5 °C during winter months. Temperatures can drop to – 10 °C and a record low of –10 °C has been recorded. The average rainfall in springs is 450 mm per annum.

2.3 Land-use

During the geotechnical investigation, the site was vacant and used as taxi rank. The site is located in a township area, surrounded by a residential area.

3. METHOD OF INVESTIGATION

The field investigation was conducted on the 28th of June 2017 comprising of a site visit, excavation of three test pits and three DCP tests in areas reflecting different soil horizons. All sites were profiled along the vertical excavation face by the author using the MCCSSO soil profiling technique advocated by Jennings, 1963 (see Appendix A). The rock faces were profiled according to current methods and procedures, (Brink ABA and Bruin RMH); "Guidelines for Soil and Rock Logging" (SAIEG, AEG & SAICE, 1994). Representative samples were submitted for testing to a commercial soils laboratory for the determination of the CBR, MOD and foundation indicator tests.

PIC A: The TLB used to excavate test pits

PIC B: DCP test underway

4. GEOLOGY

4.1 Regional Geology

According to the published 1:250,000 regional geological map (Bulletin Map No. 2626 WES-RAND), the most extensive part of the area is underlain by Basaltic lava, Agglomerate and Tuff of the Klipriviersberg Group of the Ventersdorp Super-group, with a layer of fault bound occurrence of dolomite and chert of the Malmani Subgroup of the Chuniespoort Group of the Transvaal Sequence occurring some 3km from of the study area.

4.2 Site Geology

The site geology was determined by means of excavating three test pits. Table 1 below show generalized soil profiles across the site with detailed descriptions included in Appendix A.

Table 1. Generalised soil profiles

Test Pit no.	Colluvium	Residual Basaltic lava	Basalt	Test pit depth (m)
TP1	0.0-0.2 m	0.2-3.0m	-	3.0m
TP2	0.0-0.1 m	0.1-1.5m	1.5-2.3 m	2.3 m
TP3	0.0-0.4 m	0.4-3.0m	-	3.0 m

5. HYDROGEOLOGY

5.1 Drainage Patterns

General run-off from the most part of the site will be towards the south west, following the slope or gradient of the terrain in the area.

There is a **wetland** some 156 meters away from the site; its trail is towards the south west. Evaluation should be conducted prior to construction by an Environmental specialist who has knowledge about wetlands and will assess the environmental impacts that may be posed on the existing wetland. This person shall field stake the wetland boundary and this line shall be surveyed by a professional land surveyor if the delineation is required for a land division pursuant. Please see the attached map (**Appendix B, Figure 3**) where wetland is shown.

1:50 and 1:100 Year Flood Lines

A 1:50 year flood line implies that an area below that line has a high probability of being flooded at least once in every fifty year period. Similar contextual definition applies for the 1:100 year flood line.

By law, residential developments below the 1:50 year flood lines areas are prohibited. This is due to the risk of flooding leading to property damage, health and life hazards, inconveniences etc. A hydrological study should be commissioned to determine the 1 in 50 year flood line. Proper flood line should be available from the Local Municipality Town Planning Department

5.2 Water Trajectory

Infiltrated water will flow through the upper permeable soil horizons (Colluvium) and accumulate on the medium hard rock (basalt). The investigation was conducted during the dry season.

6. GEOHAZARDS

6.1 Seismic Activities

Seismic-hazard can be described as being the physical effects of an earthquake or earth tremor. Examples of such phenomenon include surface faulting, ground shaking and liquefaction (Kijko A et al, 2004).

According to the published (Council for Geosciences) Seismic Hazard Identification Maps of South Africa, Site falls under an area with a 10 % probability of 0.16g (peak ground acceleration) being exceeded in a 50 year period. The peak ground acceleration is the maximum acceleration of the ground shaking during an earthquake.

For masonry and concrete structures, a 4 to 5 Hz Spectral Acceleration is assumed. This natural frequency of the structure can give an indication of the spectral part of the earthquake motion time history that has the capacity to introduce energy into the structure. Spectral Acceleration (ARS – acceleration-response spectra) is the movement experienced by the structure during an earthquake / seismic event.

This phenomenon is known as resonance. Resonance is where the frequency of the applied harmonic force is consistent with the natural frequency of a vibrating body. At resonance, the vibrating body will exhibit the maximum amplitude of response displacement leading to extremely high structural distress similar to popular example of the Tacoma Narrows Bridge that was situated in Washington State, near Puget Sound. Therefore, frequencies far away - either lower or higher - from the natural frequency of the structure have little capability of damaging the structure.

This area is a low seismic hazard area and the construction materials to be used (gravel) are in harmony with the naturally occurring site conditions. As a result, no major problems are foreseen in this regard.

6.2 Ground Subsidence, Sinkholes and Landslides

Subsidence and sinkholes occurs in areas with large underground cavities typically resulting from large scale shallow to very shallow mining and also from dolomite/limestone dissolution. It can also appear where high thickness of unconsolidated material exists.

This site showed no signs of previous subsidence or sinkhole occurrences. Furthermore, there is no evidence or record of active mining in the immediate vicinities that might cause drop in the ground water level thus triggering ground subsidence. The site is not a dolomitic land, so it cannot be subject to risk of all sizes of sinkhole and doline formation.

The probability of landslides and mudslides occurring at this area are also rare. This is primarily due to the low climatic conditions and composition of residual and transported materials in this area.

7. GEOTECHNICAL EVALUATION

7.1 Construction Material Suitability

The aim of this geotechnical site investigation report was to determine the different engineering geological properties of the surface and subsurface soils in accordance with the GFSH–2 guidelines, NHBRC. The intention is to be able to recommend for foundation design for the construction of a public transport facility in Emdeni, Soweto. The investigation shows that the structures can be founded on:

- **Normal Reinforced Strip Footing foundation** it is therefore recommended for the ablution block building where In-situ reconstruction or ripping is done at an average depth of **0.4m** below surface on residual material of Basaltic lava. Proper compaction should be adhered to while back filling the trenches and foundations with G5 material and should be compacted to 95% MODAASHTO at 150mm intervals.

- **Pad or Spread Footing foundation** is therefore recommended (for the steel columns that will carry the roof structure) at an average depth of **0.4m** on Residual material of Basaltic lava.
- **Pavement recommendations:** In-situ ripping should be at an average depth of at **0.4m** on residual material Basaltic lava. The estimated traffic or vehicle per day is <75 vehicles and <5 heavy vehicles per day with a total loading traffic of <0.3×10^6 E80s per lane (Guideline for human settlement planning and design Vol.2).
    - **Surface:** Pavement bricks can be placed on the surface for walking lanes, and asphalt concrete can be used for a smooth finish along the taxi parking lanes. The binder content present with asphalt acts as a lubricant when hot and as an adhesive and water proofing when cold.
    - **Base (~150mm):** G5 material should be compacted to 95% MOD AASHTO density.
    - **Sub-base (~250mm):** G3 material should be compacted to 98% MOD AASHTO density at 150mm interval.

The material on site (material of residual basaltic lava) can be classified as **G6** at 0.4m on TRH 14 of Material Classification. According to NHBRC soil classification, the soil on site is classified as **R-H1; P (Imported fill)**, with up to an estimated 7.5-15 mm total settlement.

7.2 Stability of Excavations

All of the borrow pits were observed as having stable sidewalls and this is a good indication for the behaviour of the materials when excavated and allowed to stand vertically, unsupported.

For safety reasons, sidewalls of excavations deeper than 1.5 m should be battered back to 1:1 in dry conditions. Should oblique jointing or any seepage be noted, then the sidewalls may need to be battered at a much flatter gradient. The groundwater table level should be assessed to determine if the sidewall batter will be subjected to potential pore pressure build-up. Subject to assessment, the batter design may include a groundwater relief system beneath the proposed liner system. This is only acceptable for excavation depths restricted to less than 3.0 m. All safety precautions should be adhered to. Should battering be deemed unpractical due to some site conditions, sidewalls should be supported by suitably designed shoring technique.

7.3 Site Classifications

Based on the results of the investigation, the site can be classified into two site designation zones **(class 'R' and 'H'),** as set out in the NHBRC (1999) guideline document of which the appropriate tables have been included in Appendix A.

7.4 DCP Tests

Three DCP tests were conducted (as shown in figure 1. below) to obtain an indication of the densities and CBR values for the subsoil. Measurements were taken at depths varying from surface to refusal. The results for the DCP tests are included in Appendix A.

In summary, the site average CBR values of the soil layers are **34%** at an average depth of **0.4m,** with an allowable soil bearing average pressure range of **271 kPa.** DCP results deeper than 1.5m cannot be relied on due to increased frictional resistance of the testing rods hence the tests were limited to 1.0m.

Table 2. Summarized DCP Results

TEST PIT NO	SUBGRADE DEPTH(m)	%CBR VALUE	TRH 14 CLASSFICATION	CONSISTENCY/ HARDNESS	MATERIAL AT SPECIFIED DEPTH
TP1	0.1	0	G6	Very dense	Imported fill
TP2	0.4	34	G6	Firm	Residual Basaltic lava
TP3	0.2	82	G2	Firm	Imported fill

7.5 Analysis of Laboratory Results

This investigation is based on 14 data points (3 DCP and 3 Test pits) extended below the pavement design level and representative samples were obtained for laboratory testing according to TRH 14 (1985) standard. Parameters obtained from laboratory testing are specified below:

- Moisture-Density Compaction
- Foundation Indicators
- Road indicators (Atterberg limit, grading modulus and particle size)
- CBR

Samples were obtained for road indicators, CBR and density tests, with the results included in this report. Analysis of laboratory result indicates Medium to high Potential Expansiveness and a maximum swell of 8.3% (TP1).

Compaction properties for samples obtained from one of the excavated test pits (TP 1) showing the site overall minimum and maximum dry density (MDD) and optimum moisture contents are shown below on Table 3 with a very low CBR value and low to medium swell percentage.

Table 3. Compaction Properties

	MDD (kg/m³)	OMC	CBR@ 93%	CBR@ 95%	CBR@ 98%
Minimum	1471	17.4	1	1	1
Maximum	1646	19.1	1	2	3

Table 4. Summary of road indicator test

Test Position	Depth (m)	Atterberg limit			Grading			PERCENTAGE FINER THAN (mm)				GM	CLASSIFICATION		POTENTIAL EXPANSIVENESS
		LL	PI	LS	Silt & clay %	Fine Sand %	Coarse Sand %	0.075	0.425	2.00			HRB	UNIFIED	
TP 1	2.5-3.0	58	24	12	11	69	0	89	93	97		0.21	A-2-5 (17)	-	LOW-MEDIUM

Summary of road indicator test results are tabulated on Table 4 above. The laboratory results shows clayey residual soil with low to medium heave potential expansiveness and an expected swell 8.3%, on samples obtained having minimum grading modulus of 0.21 and a maximum PI of 24. The minimum obtained CBR @ 95% is 1. Parameters obtained show excellent to good subgrade properties.

7.6 Geotechnical Constraints

The impact of the geotechnical constraints on developments may be evaluated according to Table 5 below, which is a summary of the general geotechnical constraints relevant to developments (Partridge, Wood and Brink, 1993). The Class column indicates the severity of the specific constraints for this site.

Table 5. Geotechnical Classification: Urban Development

	Constraint	Site condition	Class
A	Collapsible soil	Collapsible grain structure absent	1
B	Seepage	Perched water at an average depth of 1.5m below surface is expected	1-2
C	Active soil	Low to medium soil heave anticipated	1-2
D	High compressibility soil	Low soil compressibility expected	1
E	Erodibility of soil	Low erodibility of soil anticipated	1
F	Difficulty of excavation to 1.5 m	Intermediate difficulty in excavation	2
G	Undermined ground	No known undermined areas	1
H	Instability in areas of soluble rock	None encountered and is 3km away from site	1
I	Steep slopes	A gentle slope exists on site	1
J	Areas of unstable natural slopes	No unstable slopes	1
K	Areas subject to seismic activity	The area has no zone of known seismic activity	1
L	Areas subject to flooding	There is a wetland next to site	1-2
Geotechnical classes: Most favourable (1); Intermediate (2); Least favourable (3) Reference: (Partridge, Wood & Brink, 1993)			

8. RECOMMENDATIONS

8.1 Foundation Design

The geotechnical investigation was carried for the proposed construction of a public transport facility on Erf Re/7/239-IQ in Emdeni, Soweto, Gauteng. It should be borne in mind that the geotechnical boundaries are inferred. So, some variations to the reported conditions should be expected.

The site predominantly falls within NHBRC Site Soil **Class R-H1; P (Imported Fill)** (7.5-15 mm estimated total settlements) and the proposed structure should be founded on:

- **Normal Reinforced Strip Footing foundation** it is therefore recommended for the ablution block building where In-situ reconstruction or ripping is done at an average depth of **0.4m** below surface on residual material of Basaltic lava. Proper compaction should be adhered to while back filling the trenches and foundations with G5 material and should be compacted to 95% MODAASHTO at 150mm intervals.

- **Pad or Spread Footing foundation** is therefore recommended (for the steel columns that will carry the roof structure) at an average depth of **0.4m** on Residual material of Basaltic lava.
- **Pavement recommendations:** In-situ ripping should be at an average depth of at **0.4m** on residual material Basaltic lava. The estimated traffic or vehicle per day is <75 vehicles and <5 heavy vehicles per day with a total loading traffic of $<0.3 \times 10^{6}$ E80s per lane (Guideline for human settlement planning and design Vol.2).
    - **Surface:** Pavement bricks can be placed on the surface for walking lanes, and asphalt concrete can be used for a smooth finish along the taxi parking lanes. The binder content present with asphalt acts as a lubricant when hot and as an adhesive and water proofing when cold.
    - **Base (~150mm):** G5 material should be compacted to 95% MOD AASHTO density.
    - **Sub-base (~250mm):** G3 material should be compacted to 98% MOD AASHTO density at 150mm interval.

Proper compaction should be adhered to while back filling the trenches and foundations with G5 material and should be compacted to 95% MODAASHTO at 150mm interval. Damp proof membrane / course should be able to inhibit the ingress of moisture. Dewatering holes should be commissioned to relieve pore pressure at foundation level.

Based on the seepage encountered on site this is however recommended: A detailed geohydrological analysis should be carried out to gain a good understanding on the transmissivity and porosity of the bedrock (aquifer testing). Monitoring holes on the upstream and downstream of the structure should be constructed in order to monitor ground water levels in all seasons. Exterior drainage and foundation wall damp-proof coatings; Capillary breaks at footings and at the top of the foundation wall, Insulation, air barrier and water vapor control, air barrier and thermal insulation systems should be installed to prevent groundwater water from infiltrating; such an impermeable layer can assist as a sealing layer.

Strip footing supports a load bearing wall and transfers the load of a structure directly to the underlying soil (Knappett and Craig, 2012). The two main objectives (limit states) that foundations need to satisfy are:

- The capacity or resistance of the foundation should be adequate enough to support the applied loads and;
- Foundations should prevent excessive deformation under the applied loads.

9. GENERAL

Regular checks on the quality and compaction of the backfill to the terraces should be made during construction. It is assumed that the development will be serviced by the usual municipal services and no recommendations are made for on-site sanitation, waste disposal, cemetery, and storm water reticulation services.

Site drainage should be design in such a way that water is channelled from Buildings into a suitable storm water drainage system to avoid structural distress over a period of time. Conditions prevailing at the site suggest that no problems are foreseen for the development of the proposed structures, provided the recommendations outlined in the report are adhered to.

10. REFERENCES

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APPENDIX A

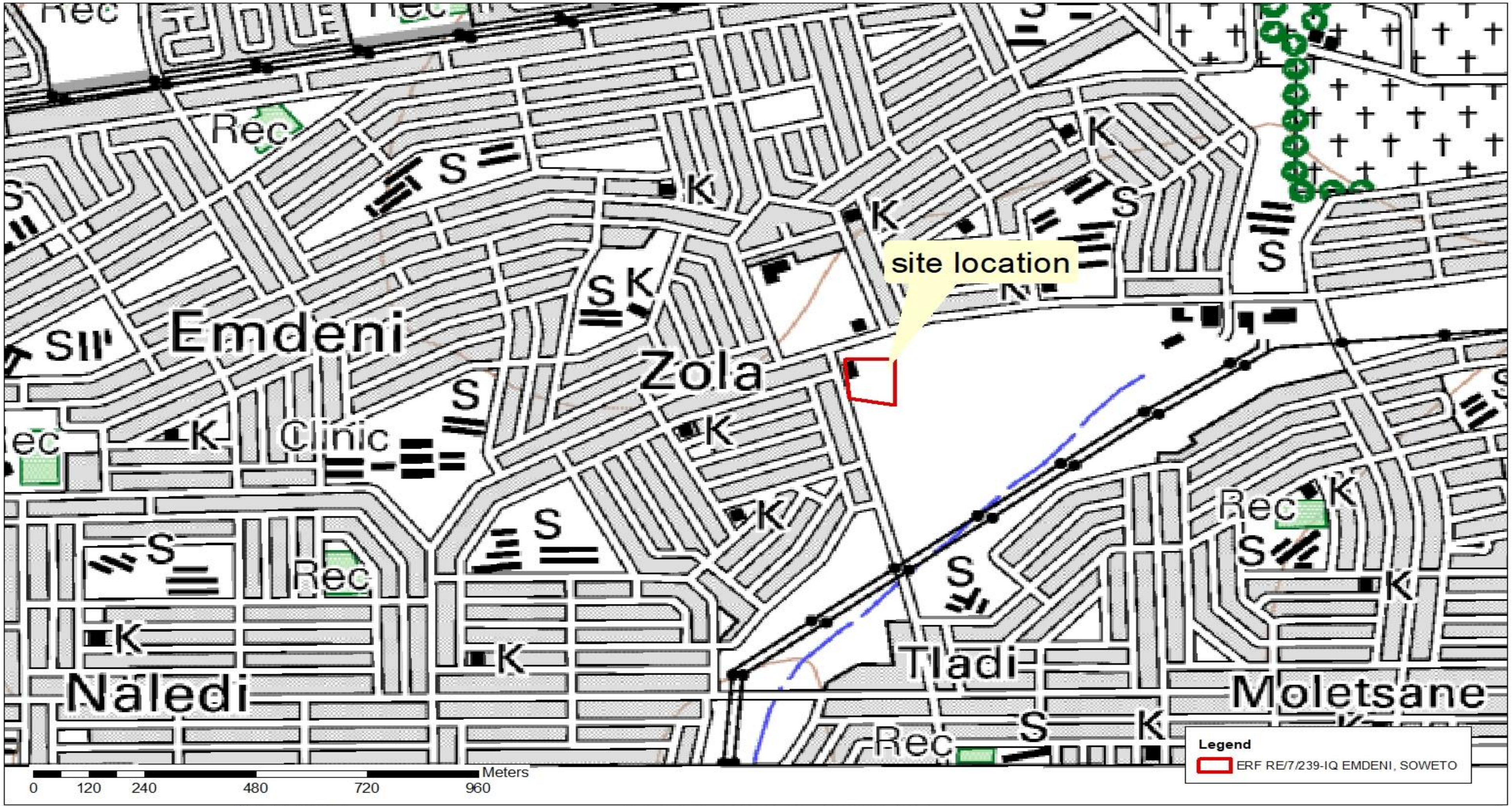
SOIL PROFILES

DCP DATA

LABORATORY RESULTS

APPENDIX B

MAPS



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Scale Factor = 1

Datum = WGS 84

MSHANDUKANI HOLDINGS

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FIGURE. 1: LOCALITY MAP
REF MAP: 2627BB ROODEPOORT 1:250 000

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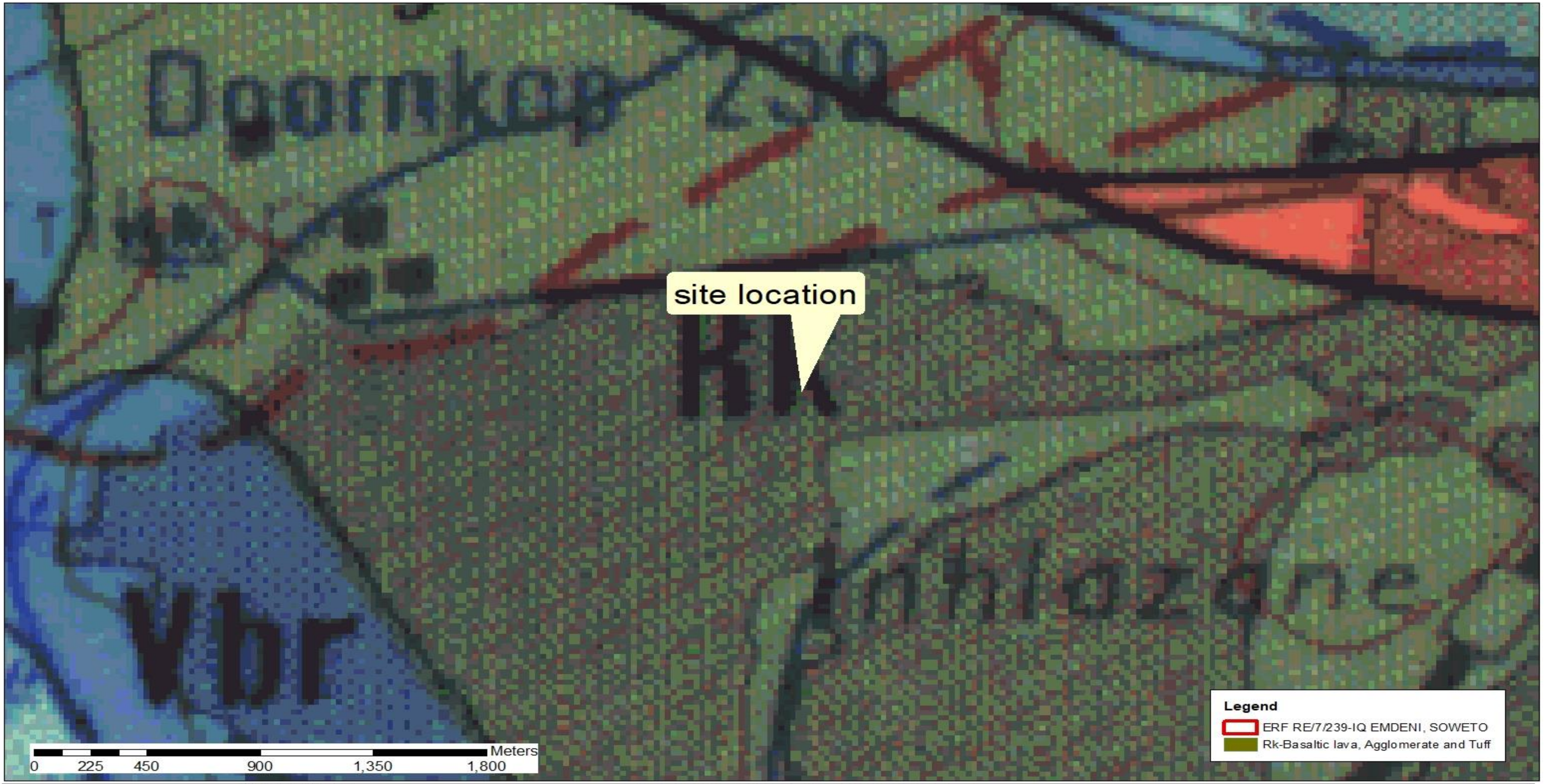

Projection Information
Transverse Mercator
Central Meridian = 29°
False Easting = 0°
False Northing = 0°
Scale Factor = 1

Datum = WGS 84

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FIGURE. 2: GEOLOGY MAP
REF MAP: 2627 WEST RAND GEOLOGICAL SERIES 1:250 000

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FIGURE. 3: TEST PITS AND DCP POSITIONS