- *Site Investigations*
- *Slope Stability*
- *Rock Mechanics*
- *Soil Mechanics*
- *Foundations*
- *Borrow Pits and Materials*
- *Roads*
- *Groundwater*
- *NHBRC*
- *Mine Stability*

The Results of a Geotechnical Investigation for the Proposed Panorama Business Park, Pietermaritzburg, KwaZulu-Natal

Client: Devrog Family Trust

Reference: 11-031

Dated: 6th October 2011

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The Results of a Geotechnical Investigation for the Proposed Panorama Business Park, Pietermaritzburg, KwaZulu-Natal

Reference: 11-031-R01 **Dated: 6th October 2011**

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Figure 1. Layout of the Site, showing location of Test Pits and DCP tests

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The Results of a Geotechnical Investigation for the Proposed Panorama Business Park, Pietermaritzburg, KwaZulu-Natal

Reference: 11-031R01 **Date: 6th October 2011**

1. INTRODUCTION & TERMS OF REFERENCE

The site, known as the Rem of Erf 783, Panorama Gardens, is located immediately to the northeast of the industrial area of Willowton, Pietermaritzburg. It is understood that the development will comprise a business park with both commercial and general business components as well as a 12000 m^2 commercial centre in the southern portion of the property. Geotechnically it is important to establish the subsurface conditions which prevail on the site and to this end, Mr P Govender ASG Properties, on behalf of the Devrog Family Trust, requested GeoZone GeoServices to provide a cost estimate for carrying out the geotechnical investigation for the above development. This cost estimate, referenced 072-11, was submitted on the 30th August 2011 and was accepted by the client, following which GeoZone GeoServices was verbally appointed to carry out the work as proposed.

This report presents the findings of the geotechnical investigations for the site and discusses the results of the fieldwork, geology, laboratory testing and sub-surface conditions. Based on the fieldwork and laboratory data, recommendations are provided for foundations, earthworks, roads and site drainage.

2. AVAILABLE INFORMATION

The following information was drawn upon for the purposes of the investigation:

- A site plan by Mark Puttick & Associates titled *"Panorama Business Park Revised Alt 5A: with Piped Channel"*, dated August 2011.
- The 1:250 000 Geological Map titled *"Durban"* as compiled by the South African Geological Survey, 1988,
- Google Earth imagery of the site, and
- Garmaps Africa 2010.

3. SITE DESCRIPTION

The site is located immediately northeast of the Willowton industrial area in Pietermaritzburg. It is understood that the development will comprise a business park with both commercial and general business components as well as a 12000 m^2 commercial centre in the southern portion of the property. The property is irregularly shaped and elongated along a north-south axis and in total occupies an area of 117 561 m^2. It is bounded on its northern extremity by the Afrisam quarry, to the northwest by an informal settlement, to the east by residential properties and a school, to the west by Birmingham Road and to the south by Naven Boulevard. Access to the site is via a gravel road which traverses the northwestern margin of the property. Topographically the land slopes towards the west at gradients ranging from 1 vertical in 7.5 horizontal on the upper slopes to less than 1 vertical in 30 horizontal on the lower, northwestern portions of the site. A small westward flowing stream approximately bisects the site with associated vlei areas in the lower lying portion of the property. At the time of the investigation the site was undeveloped with the southern and western margins of the site covered in invasive vegetation while the eastern, higher portion was under eucalyptus trees and scrub. The layout of the site is shown in Figure 1.

4. FIELDWORK

The fieldwork was carried out on the 23rd September 2011 and comprised the excavation, logging and sampling of test pits to determine the underlying soil conditions. In addition, DCP tests were carried out adjacent to the pits to determine the *in situ* shear strength of the underlying soils and the depth to a suitable founding horizon and excavation depths. The positions of the tests pits and DCP tests are shown in Figure 1.

4.1 Test Pits

Fourteen test pits, designated TP1 to TP14, were machine-excavated using a mechanical backhoe to depths ranging from 1.9 m to 2.7 m below existing ground level, with an average depth of 2.2 m. These test pits were profiled, sampled and backfilled on completion and selected soil samples submitted to a soils laboratory for testing.

The detailed soil profiles from the test pits are presented in Appendix A.

4.2 DCP Tests

Thirteen dynamic cone penetrometer (DCP) tests, designated DC1 to DC13 were carried out adjacent to the test pits and the results of the DCP tests, plotted as blow count versus depth, are presented in Appendix B.

5. SUBSURFACE SOIL CONDITIONS

The site is underlain by mantle of fill, pedogenic, colluvial and residual soils which overlie weathered dolerite of Post Karoo age. The Afrisam Quarry, located adjacent to the northern margin of the property, extracts dolerite aggregate from what is a significantly sized dolerite intrusion, the limits of which are not confined to those of the quarry boundaries. As such dolerite is expected to underlie much of the surrounding areas. In addition the mineralogy of the dolerite has had an effect on the underlying soil profiles, about which will be discussed in greater detail below.

5.1 Fill

Fill was encountered in TP’s 1, 2, 3, 8, 13 and 14 and extended to depths ranging from 0.2 m (TP 13 and TP14) to 1.9 m (TP3) with an average depth of 0.7 m. By its nature fill is always variable and ranged from gravel road base to accumulations of garden potting bags and garden refuse.

5.2 Colluvium

The colluvial soils comprise in general slightly moist, dark greyish brown, soft, silty Clay which extends to depths ranging from 0.2 m to 1.1 m below existing ground level, with an average depth of 0.5 m. In a number of instances the colluvium is not present due to earthworks which have been undertaken on the site during its development history.

5.3 Pedogenic Horizons

Pedogenic material, comprising ferricrete Gravel and hardpan horizons, were encountered most significantly in TP 10 and TP 12. Iron oxides, derived from the high iron content contained within the mineralogy of the large dolerite intrusion to the north, have been carried by groundwater and deposited within the soil horizons. This has lead to accumulations of ferricrete gravel and ultimately to a hard, impenetrable horizon known as hard pan. Hard pan was encountered in TP 10 and proved impossible to penetrate with the TLB backhoe while a 0.8 m thick accumulation of ferricrete Gravel was encountered in TP 12.

5.4 Alluvium

Alluvium was encountered in TP’s 3, 8 and 9 and was seen to comprise very moist to wet, greyish brown/yellowish, soft, very silty Clay which extends to depths in excess of 3.0 m below existing ground level.

5.5 Residual Soils

The residual soils extend to depths extending to in excess of 3.0 m and typically comprise reddish brown, soft to firm, silty Clay with variable amounts of ferricrete gravel scattered throughout the horizon which may be indicative of a fluctuating water table.

5.6 Bedrock

Bedrock was not encountered in any of the test pits excavated on the site.

6. GROUNDWATER & SURFACE WATER

Groundwater seepage was encountered in the following test pits as follows:

TP3 – Slight flows at 2.5 m
TP8 – Moderate flows at 2.0 m
TP9 – Strong flows at 1.5 m

All of the above pits were excavated within the influence of the stream which drains the site with the stronger flows occurring in the western, wetter areas. Surface seepage and standing water was also encountered along the valley invert and in the western corner of the site.

7. RESULTS OF THE INVESTIGATION

7.1 Laboratory Testing

In order to evaluate the engineering properties of the soils and their suitability as subgrade and fill material, selected indicator and bulk samples were taken from the test pits and submitted to a soils laboratory for testing. The results of the laboratory tests are given in Appendix C and summarised in Table 1 below.

Table 1: Summary of Laboratory Test Results

TP No.	Depth (m)	Description	Particle Size Percent retained			Atterberg Limits (%)			GM	OMC (%)	MDD (kg/m³)	CBR Values Compaction MDD %					Swell (%)	Heave	Class & Group Index
			Clay & Silt	Sand	Gravel	LL	PI	LS				90	93	95	98	100			
TP5	1.0 - 1.5	Reddish brown CLAY – Residual Dolerite	99	1	0	68	27	13	-	-	-	-	-	-	-	-	-	Low	A-7-5(19) -
TP11	0.5 - 1.0	Orange brown, very silty CLAY – Residual Dolerite	69	24	7	50	28	12	0.45	20.6	1670	3	4	5	9	12	0.86	High	A-7-6(16) G10
TP13	0.6	Reddish brown silty CLAY – Residual Dolerite	83	16	1	57	31	14	-	-	-	-	-	-	-	-	-	High	A-7-6(19) -

Key

LL	-	Liquid Limit	OMC	-	Optimum Moisture Content			
PI	-	Plasticity Index	MDD	-	Maximum Dry Density	CBR	-	California Bearing Ratio
LS	-	Linear Shrinkage	G8	-	Classification in Terms of TRH14 (1985)	NP	-	Non Plastic
SP	-	Slightly Plastic	CBD	-	Cannot be Determined	GM	-	Grading Modulus

8. DEVELOPMENT RECOMMENDATIONS

8.1 Proposed Development

It is understood that the site will be subdivided into 37 general sites, a single 41 645 m^2 general/commercial site, a religious site and public open space. An access ring road is to be constructed within the bounds of the property, allowing access to all of the sites.

8.2 Rippability & Trenchability

Soft excavation in terms of SABS 1200 is anticipated to at least a depth of 3.0 m below existing ground level using light earthmoving equipment due to the soft nature of the underlying soils. Some difficult excavation may locally occur below 1.2 m in the vicinity of TP 10 where the hardpan ferricrete occurs.

8.3 Earthworks

It is recommended that all earthworks be carried out in accordance with SABS 1200 (current version).

In terms of construction recommendations, all topsoil and fill should be cleared from the areas that will be subject to earthworks and the topsoil stockpiled for later site rehabilitation.

It is anticipated that the cuts carried out on the site will be predominantly in colluvial and residual soils. These materials should to be battered back at gradients of 1 vertical in 2 horizontal (slope angles of 27 degrees) to promote their long-term stability. Any slopes greater than 2.0 m in height will need to be analysed in terms of their global stability and should be discussed with GeoZone GeoServices prior to any construction taking place.

During embankment and fill construction, the fills should be placed in layers not exceeding 200 mm loose thickness, and compacted to a minimum of 93% Modified AASHTO dry density. Any boulders, rubble, or material larger than two-thirds of the layer thickness must not be included in the fill material. In addition the fill material should be worked within ± 2% of the optimum moisture content to reduce the threat of heave of the material during compaction. Saturated, heaving soils will be impossible to compact to the specified 93% Modified AASHTO dry density. Visual assessment of the *in situ* soils, and the laboratory test results show that the materials which underlie the site are less than or equal to G10 in quality, with the G10 material being suitable only for the construction of general fills. The laboratory maximum dry density and optimum moisture content are indicative and it is highly recommended that additional testing be carried out as soon as construction starts to determine as closely as possible a working maximum dry density for the earthworks operation.

8.4 Drainage

One of the most important factors in the promotion of a stable site is the control and removal of both surface and groundwater from the property. It is important that the design of the stormwater management system allow for the drainage of accumulated surface water and that it is collected and disposed of in a responsible manner.

Both during and after construction, the platforms should be well graded to permit water to readily drain from the site, and to prevent ponding of water anywhere on the surface. All terraces and earthworks in general should be graded to prevent ponding and ingress of water into the subsurface soils.

8.5 Surface Drainage

Surface water collected on any roads, hardened areas or parking areas should be directed to and collected in open, lined drains or piped off the various sites. Run-off from roofs should be piped from gutters through downpipes and similarly discharged into the stormwater system. Care must be taken not to affect area downslope of the development as volumes of run-off can be significant and lead to damage and erosion to adjacent properties.

8.6 Sub-Surface Drainage

There are significant accumulations of surface water in western corner of the site where the stream exits the property. It is understood that the stream is to be contained within a concrete culvert over a portion of this area. However it may prove that subsoil drains need to be installed to draw down the water table in this locality, and if so, it is recommended that these are designed according to the filter criteria of the *in situ* soils to prevent piping. The design of subsoil drains should be discussed in detail with GeoZone GeoServices should the need arise.

8.7 Evaluation of Founding Conditions & Foundation Recommendations

8.7.1 Overview

The geology of the site is characterised by accumulations of localised fill, colluvial, alluvial and residual soils, which are inferred to be underlain at depth by dolerite of Post Karoo age. The laboratory test results show that the residual soils have a high heave potential in most instances when assessed according to the van der Merwe criteria, with cumulative heaves in excess of 26 mm being anticipated. In addition, some localised pinholing was noted within the soil structure, indicative of a voided soil profile and a collapse potential of the soils. Finally in the wet, alluvial areas on the northwestern and western portions of the property have led to soft founding conditions with an associated threat of settlement.

8.7.2 Site Classification

Based on the above assessment, the site has been classified according to the NHBRC criteria. Strictly speaking the NHBRC regulations are for single-storey dwellings but have found widespread use throughout the industry when assessing founding conditions for relatively lightly loaded structures in general. As such they provide tried and tested methods for reducing the effects of soil heave or settlement.

The area to the northwest and west has been classified as **S2** due to the thick accumulation of saturated, soft clays in this area. Much of the remainder of the site has been classified as **H2** due to the high heave potential of the underlying clays, and the thickness thereof. A localised area in the vicinity of TP 10 has been classified as **H** due to the presence of a hardpan Ferricrete horizon, although the limits of this horizon still need to be defined. As such it may be prudent to treat the whole site, apart from the **S2** area, as **H2**. Table 3 below is a summary of the NHBRC Classes and anticipated soil movements.

Table 3: Summary of NHBRC Site Classes and Anticipated Range of Soil Movement.

TYPICAL FOUNDING MATERIAL	CHARACTER OF FOUNDING MATERIAL	EXPECTED RANGE OF TOTAL SOIL MOVEMENTS (mm)	ASSUMED DIFFERENTIAL MOVEMENT (% OF TOTAL)	SITE CLASS
Fine grained soils with moderate to very high plasticity (clays, silty clays, clayey silts and sandy clays)	EXPANSIVE SOILS	<7,5 7.5-15 15 - 30 >30	50% 50% 50% 50%	H H1 H2 H3
Fine grained soils (clayey silts and clayey sands of low plasticity), sands, sandy and gravelly soils	COMPRESSIBLE SOILS	<10 10-20 >20	50% 50% 50%	S S1 S2

8.7.3 Founding Solutions

Armed with the geotechnical data gathered from the test pits, DCP tests and laboratory test results the most appropriate founding solutions can be selected for each site classification.

8.7.3.1 H Areas

H areas are very limited and care should be exercised when deciding on a founding solution for sites in this vicinity as the limits of the hardpan Ferricrete which allow this area to be classified as **H** have not yet been defined. For H areas, normal founding solutions may be adopted, with foundation loads not exceeding 75 kPa.

8.7.3.2 H2 Areas

H2 areas predominate over the site due to a combination of high heave potential and thick accumulations of clay. As such a number of founding solutions are available as per the NHBRC requirements, and these are summarised in detail in Appendix D. However, from a practical point of view it is recommended that a stiffened or cellular raft foundation solution be adopted for founding the structures, with the necessary site drainage and service and plumbing precautions. For the larger structures split construction with full movement joints within the masonry are recommended.

8.7.3.3 S2 Areas

For the **S2** areas in the northwest, it is recommended that stiffened strip footings or stiffened or cellular rafts with lightly reinforced or articulated masonry be adopted.

Bearing pressures should not to exceed 50 kPa and the slabs should be constructed with mesh reinforcement. Site drainage and service/plumbing precautions need to be taken.

8.7.3.4 General

In terms of preparing the surface beds for the proposed structures, these should be compacted to 95 percent Modified AASHTO Maximum Dry Density.

Under no circumstances should foundations be placed in fill unless it has been specifically engineered to support structural foundations. Should the structures bridge the cut-to-fill line then the footings need to be taken down through the newly emplaced fill and founded on the *in situ*, undisturbed soil horizons, or alternatively founded on structural rafts.

It is recommended that GeoZone GeoServices inspects and approves all foundation excavations to confirm depth of founding and bearing capacity of the underlying founding horizons.

8.8 Roads, Hardened Areas and Driveways

From the preliminary laboratory test results, the soils that underlie the site are G10 (and possibly less than G10) in quality due to very high clay and silt content. Table 3 below, derived from the Technical Recommendations for Highways (TRH14) summarises the material requirements for various pavement layers.

Table 3: TRH14 Material Code Requirements for Various Pavement Layers

Layer	**Material Code**
Subbase	G5 and G6
Selected Layer	G6, G7, G8, G9
Subgrade	G8, G9, G10

In this light then, it is recommended that any soils which are of lower quality than the those given for Subgrade in Table 3 above are undercut to a depth of 200 mm or 300 mm, depending on axle loads, below platform level and replaced with G8 or better material, compacted to 93% Modified AASHTO Dry Density.

As a general rule of thumb, the highest quality material that is economically available should always be used for fill and subgrade construction.

9. SUMMARY & CONCLUSIONS

This report presents the findings of a geotechnical investigation carried out for the proposed development of the Rem of Erf 783, Panorama Gardens, Pietermaritzburg, KwaZulu-Natal.
The site is underlain by fill, colluvial and residual soils that are inferred to overlie weathered dolerite bedrock of Post Karoo age. Slight groundwater seepage was encountered on the in the lower, northwestern portion of the site.

Soft excavation in terms of SABS 1200 is generally anticipated at depths in excess of 3.0 m below existing ground level except for areas underlain by hardpan Ferricrete where hard excavation is anticipated. It is recommended that all earthworks be carried out in accordance with SABS 1200 (current version).

One of the more important factors in the promotion of a stable site is the control and removal of surface water from the property. It is important that the design of the stormwater management system allows for the drainage of accumulated surface water from the platforms and into the stormwater system or natural drainage lines.

The soils below the site have a high heave potential and cumulative heave and settlements are expected to be greater than 20 mm based on the current soil profiles. As such the site has been classified as H2 and S2 and the necessary precautions should be taken when founding structures on the property. A localised area of H soil conditions was encountered but the extent of the hard pan ferricrete horizon which underlies this zone needs to be ascertained.

The entire site is underlain by clayey colluvial and residual soils, which are of G10 and possibly <G10 in quality. This low specification material will be unsuitable for subgrade and will need to be undercut to a depth of 200 or 300 mm and replaced with G8 material, compacted to a density of 93 percent Modified AASHTO.

It is recommended that a site specific, detailed geotechnical investigation be carried out to determine the geotechnical conditions for each site. This will have inherent cost savings as the optimum founding solution for each site can be determined without having to apply a one-size-fits-all design for the foundations.

Finally, the ground conditions described in this report refer specifically to those encountered at the test positions on the site. It is therefore possible that conditions at variance with those discussed above may be encountered elsewhere on the site. In this regard it is important that GeoZone GeoServices carry out periodic inspections of the site during construction to ensure that any variation in the anticipated ground conditions can be assessed and revised recommendations made to avoid unnecessary delays and expense. Furthermore it is important that the construction phase of the project be treated as an augmentation of the geotechnical investigation.

________________________ 6th October 2011

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Figure

Appendix A
(Test Pit Logs)

Appendix B
(DCP Test Results)

Appendix C
(Laboratory Test Results)

Appendix D
(NHBRC Site Classification Tables)