- *Site Investigations*
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- *Groundwater*
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- *Mine Stability*

The Results of a Geotechnical Investigation on Erf 1359, Huntley Road, Queensburgh.

Client: Yethusodwa (Pty) Ltd

Reference: 22-032

Dated: 15th November 2022

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Reference: 22-032 **Date: 15th November 2022**

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1. INTRODUCTION & TERMS OF REFERENCE

A number of seven storey high blocks of flats are proposed on a site known as Erf 1359, located on Huntley Road, Queensburgh. Due to the high foundation pressures imposed by such structures it is important to understand the prevailing geotechnical conditions underlying the site from a founding and slope stability point of view, bearing in mind that the slopes will be modified by bulk earthworks and imposed loads. As such, GeoZone GeoServices was asked by Mr M Mondli, of Mondli Consulting Services on behalf of his client, Yethusodwa (Pty) Ltd to provide a cost estimate for carrying out the geotechnical investigation as per the details set out in Table 1 below.

Table 1: Terms of Reference

Client	Yethusodwa (Pty) Ltd
Contact	Mr M Mthembu
Proposal Reference	055-22 dated 29th September 2022
Appointment Date	11th October 2022

The results of the investigation are presented below.

2. AVAILABLE INFORMATION

The information drawn upon for the purposes of the investigation is listed in Table 2 below:

Table 2: Information used in this Investigation

Description	**Source**
Satellite Imagery	Google Earth 2022
1:250 000 Series Geological Map titled *2830 Durban*	Council for Geoscience
Engineering Geology of Southern Africa Volume 3 The Karoo Sequence.	A.B.A. Brink, Building Publications, 1983

3. SITE DESCRIPTION

The pertinent details of the site are summarised in Table 3 below:

Table 3: Details of Site

Coordinates	29°52'26.40"S, 30°55'54.98"E (approximate centre of the site)
Topography	Slopes steeply to the northeast at gradients of 1:3, vertical to horizontal. A drainage gully bisects the lower portion of the site, flowing towards the northeast.
Area (m^2)	Approximately 1.64 Ha.
Boundaries	Huntley Road to the south, developed properties to the east and west, thick invasive bush and vegetation to the north.
Existing Infrastructure	None
Access	Via Huntley Road to the south of the site
Vegetation	Abundant invasive vegetation

Figure 1 shows the location of the site and its immediate environs.

4. FIELDWORK

The fieldwork was carried out between the 1st of November 2022. A second day of fieldwork was required due to the steepness of the site and the almost impenetrable bush that had to be cleared to provide access to the test positions. Seven machine dug test pits, designated TP1 to TP5, and TP11 and TP12, and five hand dug pits, designated TP6 to TP10, were dug across the site to determine the nature of the underlying soils, the depth to bedrock rock, and the presence of a water table. In addition, twelve Dynamic Cone Penetrometer Light tests, designated DC1 to DC12, were carried out adjacent to each test pit to determine the shear strength of the underlying soils.

The positions of these test pits and DPL tests are shown in Figure 1. The logs of the test pits are included in Appendix A and the results of the DPL tests, plotted as blow count versus depth, are included in Appendix B.

5. GEOLOGY

The site is underlain by residual soils which are underlain by weathered tillite of the Dwyka Group.

5.1 Colluvium

Colluvium in the usual sense is absent from the site except in a broad sense.

5.2 Residual Soils

The upper residual soil profile comprises dark greyish brown mottled yellowish brown, subangular to angular, highly weathered, medium to coarse tillite Gravel in a dense intact matrix of sandy clay. This horizon extends to depths ranging from 0.2 m (TP2 and TP3) to 0.45 m (TP11). The thickness of this horizon increases towards the lower portions of the site.

5.3 Bedrock

Rockhead comprises highly weathered, light brown becoming greyish brown with depth, closely jointed very soft to soft material, with increasing hardness with depth. Hard, bluish grey Tillite boulders up to 1.2 m in diameter were noted on the surface in the upper portion of the site.

6. GROUNDWATER

No groundwater was encountered in any of the test pits excavated on the site.

7. LABORATORY TESTING

In order to assess the engineering properties of the *in situ* soils, which will assist in determining the heave potential of the soils, their suitability for fill construction and as subgrade materials for road and pavement construction, bulk samples were taken from the test pits and submitted to a soils laboratory for testing. The results of this test are summarised in Table 4 and the raw data are presented in Appendix C.

Table 4: Summary of Laboratory Test Results

TP No.	Depth (m)	Description	Particle Size (% retained)				Atterberg Limits (%)			GM	Heave	MDD (kg/m³)	OMC (%)	CBR Values					Swell (%)	Group Index & TRH14 Classification
			Clay	Silt	Sand	Gravel	LL	PI	LS					Compaction MDD %						
														90	93	95	98	100		
TP1	0.0-0.40	Highly weathered medium to coarse tillite GRAVEL in a dense intact matrix of sandy clay – Residual Tillite.	17		13	70	25	11	6	2.26	Low	1994	8.0	8	11	14	20	25	0.12	A-2-6-(0) G8
TP4	0.0-0.35	Dry dark brown dense intact sandy CLAY. Residual Tillite.	18		21	51	27	13	7	2.04	Low	2008	10.4	13	17	20	25	29	0.02	A-2-6-(0) G7
TP4	0.35-1.40	Light brown completely weathered very closely to closely jointed very soft rock TILLITE. Dwyka Group	26		28	46	24	10	6	1.82	Low	1999	8.6	9	12	15	21	26	0.11	A-2-4-(0) G8

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			

8. DISCUSSION OF RESULTS

8.1 Proposed Development

A number of seven-storey flats has been proposed for the site, with associated cut-to-fill platforms to accommodate the structures, access roads and parking areas.

8.2 Stability of the Site

The site is considered stable and suitable for development provided that the recommendations in this report are adhered to. However some caution does need to be exercised in the northwest corner, where the existing slope dips in a similar direction to that of the regional dip. This will be discussed in more detail under Section 9.6.

8.3 Rippability & Trenchability

Soft excavation in terms of SABS 1200 is generally anticipated to depths of approximately 1.0 m below existing ground level, below which heavy ripping and possible blasting is expected.

8.4 Site Clearance and Earthworks

Bulk earthworks drawings were not available at the time of the investigation and as such it is impossible to comment specifically on how the earthworks should be undertaken. However, the following general recommendations should be followed.

The site should be grubbed down and all the remaining vegetation removed from the site. There is almost no colluvium on the site and as such there will not be a need to stockpile this material for site rehabilitation. However, the gravelly upper residual tillite material may prove to be a useful source of subgrade and fill material and it is recommended that this upper 200 to 400 mm thick horizon is stripped and stockpiled for later use.

Cuts will be in the very soft to soft, highly weathered tillite material in the upper metre of and possibly deeper. Deep cuts may encounter W3 slightly weathered, soft to hard rock. Fills will be founded on this same material and in all likelihood be constructed with the W4, highly weathered, yellowish brown, very soft to soft rock which forms the upper horizons. Depending on the fill requirements it should be possible to use the less weathered, blocky material as pioneer layers in the base of the fill wedges. It is expected that the material will meet a G10 quality at least and may be used in general fills and as subgrade. However, this will be confirmed when the laboratory test results become available.

During the compaction process the material should be placed in layers not exceeding 200 mm loose thickness, and compacted to a minimum of 93% Modified AASHTO maximum dry density in the areas of cut platforms, parking areas and roads. Where structures are to be built this specification should be raised to a density of 95% Modified AASHTO to prepare the area for the surface beds.

Any boulders, tree stumps or material larger than two-thirds of the layer thickness must not be included in the fill material. In addition, it is imperative that the emplaced fill material should be worked within 2 percent of the optimum moisture content to ensure that the correct degree of compaction is attained. Material which deviates too far from the optimum moisture content will be difficult to compact to the specified density. Too much water will cause the layer to heave as the compactive effort is directed into trying to essentially compress a liquid. Alternatively, too little water will prevent the soil particles from binding together into a denser, tightly-locked mass. It is important therefore that due attention is given to the quality assurance aspect of the operation at the start of construction.

In that the integrity and quality of the bulk earthworks programme will affect the entire development it is important that a quality assurance plan be put in place to ensure that the correct compactions are achieved and that the earthworks contractor has done his job correctly. GeoZone GeoServices can assist in this regard to ensure that the correct compactions are being achieved.

8.5 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, particularly in view of the steepness of the site. It is important that the design of the stormwater management system allows for the collection and removal of accumulated surface water in a responsible manner. Both during and after construction, the various platforms should be well graded to permit water to readily drain from the site, and to prevent ponding of water anywhere on the surfaces. All terraces and earthworks in general should be graded to prevent ponding and ingress of water into the subsurface soils.

8.5.1 Surface Drainage

Surface water collected on the platforms, hardened areas and access roads should be collected in open, lined drains and directed off site and into the valley invert.

Run-off from roofs should be piped from gutters through downpipes and similarly discharged into the stormwater system. It is imperative that the design of the stormwater system is such that it is able to cope with the significant run off that a development of this nature can generate.

Due to the considerable quantities of surface run-off which is anticipated from the hardened areas and roofs, consideration needs to be given to reducing the energy of the flows and possibly attenuating the hydrographic curves during periods of intense rainfall. The design of these systems falls outside the scope of this report.

It is also important to ensure that stormwater is prevented from entering fill wedges and backfill behind retaining structures, in particular dry stack walls. Concrete aprons and dish drains along the crest of retaining structures should be installed to assist in this regard.

8.5.2 Sub-Surface Drainage

Subsurface soil drainage is not expected to be required on the site due to the lack of groundwater seepage that was encountered in the test pits. However, should groundwater be encountered, it is recommended that subsoil drains be installed, designed according to the filter criteria of the *in-situ* soils to prevent piping. Geofabric separation layers may also be used to keep clay from entering gravel drainage zones. GeoZone GeoServices can assist with detailed design of drainage measures.

In situ groundwater seepage aside, it is imperative that drainage is included behind all retaining structures to ensure that the moisture content of the fill wedge in these areas is kept to within acceptable levels. Saturated fill behind a wall will lead to the lowering of the shear strength of these materials and possible failure of the structure.

8.6 Evaluation of Founding Conditions and Foundation Recommendations

It is understood that seven storey structures are proposed for the site. The foundation pressures for buildings of this size are not insignificant and caution will need to be exercised in terms of deciding on the best foundation solution, and ensuring that foundation pressures are transferred down to a suitable bearing horizon. Furthermore, it is important that the global stability of the site is maintained through judicious siting of the buildings, the removal of the surface and groundwater and ensuring that any discontinuities in the rock mass do not become planes of weakness along which slope failure takes place.

The DPL results show that refusal depths are consistent across the site, ranging from 0.3 to 0.9 m maximum. The test pits also show shallow refusal, ranging from 0.65 m to 1.60 m for the machine dug pits, which is a better indication of where a suitable founding horizon lies than that of the hand dug pits.

In terms of founding single storey and double storey structures, it is expected that foundations will be taken down to the underlying bedrock which lies close to the surface, and as such soil heave and settlement is expected to be negligible. In addition, the bulk earthworks will also reduce the soil cover in the cut areas, with founding expected to be directly on weathered tillite over much of the site. Rock is expected to lie at depths ranging from 0.2 m to 0.5 m below existing ground level, although it is important to ensure that the foundations are located in very soft rock at least.

For the multistorey structures it is envisaged that live and dead loads will be transferred laterally to columns, with the vertical column loads being transferred to a bearing horizon via a selected and suitable foundation. For a seven-storey building, column loads are expected to be high. Brink (1983) has indicated that Grade 3 tillite has an average unconfined compressive strength of 74 MPa, and these are useful figures when deciding on design foundation pressures. However, this figure is assumed to apply to the rock material and not the overall rock mass and as such does not take into account the possible presence of soft, highly weathered horizons, or clay-filled bedding planes within the rock mass. An irregular weathering profile may also reduce the strength of the rock mass laterally across the site. Consideration also needs to be given to the orientation of the strata. The published regional dip is towards the east at angles of between 10 degrees and 30 degrees. The majority of the Huntley Road site slopes to the north which is favourable in terms of slope stability, in that there is no intersection of the bedding planes with the topography, – that is, there should be no daylighting of bedding planes in the natural slope or any cut slopes on the site. However the northwest corner slopes down towards the drainage gully, which is approximately the direction of the regional dip of the strata. The Dywka Tillite is a massive deposit but there may be localised bedding planes within the rock mass and it is important to ascertain if this is the case, and if so, whether they daylight in the cut faces. Daylighting bedding planes in cut slopes with imposed loads from the structures, plus the possibility of influx of stormwater from hardened areas, roofs and leaking plumbing is a recipe for disaster.

The depth to high quality, W3 grade, soft to hard rock is currently unknown.

Based on published and field data, and taking cognisance of the uncertainty associated with a shallow geotechnical investigation of this type, it is recommended that the foundations are taken down to W3, medium weathered, '2nd brown' soft to medium hard rock. If this recommendation is carried out, then the columns may be supported on suitably designed and reinforced pad foundations. Taking into account the above factors, it is recommended that foundation pressures are kept to below 750 kN/m^2 to mitigate the effects of settlement, and to reduce the possibility of movement along clayey/shaley bedding planes within the rock mass. A range of pad sizes based on various column loads has been calculated for foundation pressures of 750 kN/m^2 and these are summarised in Table 5 below. These figures are for guidance purposes only and the structural engineer will need to calculate the loads and pad sizes independently.

Table 5: Guideline Column Loads and Estimated Pad Dimensions

Column Load (kN)	**1000**	**1500**	**2000**	**2500**	**3000**	**3500**	**4000**
Pad Width (m)	1.15	1.41	1.63	1.83	2.00	2.16	2.31
Pad Length (m)	1.15	1.41	1.63	1.83	2.00	2.16	2.31
Area of Pad (m^2)	1.3	2.0	2.7	3.3	4.00	4.7	5.3
Pressure (kN/m^2)	750	750	750	750	750	750	750

The above figures are based on the results of the fieldwork and published data. However, it is imperative that the geotechnical conditions are confirmed on site during the construction phase, as seven storey structures come with their own challenges and responsibilities. In this regard it is imperative that GeoZone GeoServices visits the site during the construction phase to ensure that the foundations have been taken down to rock with an unconfined compressive strength of at least 3 MPa. During this inspection attention will also be given to any unfavourably orientated bedding planes or clay filled discontinuities that may affect the global stability of the site, particularly in the northwest corner of the site.

8.7 Roads and Paved Areas

Table 6 below, derived from the Technical Recommendations for Highways (TRH14) summarises the material requirements for various pavement layers.

Table 6: 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

The residual tillite encountered on site ranges from G7 to G8 in quality and may be used in general fills, as subgrade and as selected layer, as per Table 6 above. Material that lies at surface may be ripped to a depth of 200 mm for parking areas and lightly trafficked roads, and recompacted to 93 percent modified AASHTO dry density. Where fills are built from this material they should also be compacted to the same standard.

It is recommended that some additional laboratory testing be carried out during construction to confirm the above findings, as the recommendations given above are based on limited testing carried out for this investigation.

9. SUMMARY AND CONCLUSIONS

This report presents the findings of a shallow geotechnical investigation carried out on Lot 1359, Huntley Road, Queensburgh.

Test pits and DPL tests were carried out across the site. The site is underlain by residual soils and highly weathered tillite of the Dwyka Group.

The site is considered stable and suitable for development provided that the recommendations provided in this report are followed. It is understood that 7 storey high structures are to be constructed on the site, which comes with its own challenges in ensuring that the column loads are adequately transferred to the underlying geological substrate. The Dwyka Tillite is a good material for founding, but it is important that the foundations are taken down to W3 soft to hard rock with an unconfined compressive strength of at least 3 MPa. Foundation pressures should not exceed 750 kPa to mitigate the threat of settlement and global instability.

The *in situ* tillite comprises G7 and G8 material which is suitable for general fills, subgrade and selected layer use.

Finally, the ground conditions described in this report refer specifically to those encountered in the tests carried out on the site. It is therefore possible that conditions at variance with those described above may be encountered elsewhere on the property. 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.

15th November 2022

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Figures

Appendix A

Appendix B

Appendix C