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VOLUME 1

PHASE 1 ENGINEERING GEOLOGICAL INVESTIGATION
BELFAST MALL
BELFAST - MPUMALANGA

PROJECT: LL2441
Version: 1A
DATE: December 2015
BY: B Cilliers ***Pr. Sci. Nat.***
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Report Type: Phase 1 Geotechnical Investigation

Project Title: PHASE 1 ENGINEERING GEOLOGICAL INVESTIGATION FOR BELFAST MALL, MPUMALANGA

Compiled for: Timberlands Beherend Pty (Ltd)

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3. I have no bias with regards to this project or towards the various stakeholders involved in this project.

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OUR REF: LL2441

Attention: Mr. O. Nkosi and Mrs. L. Swarts Pr.Pln.

RE: PHASE 1 ENGINEERING GEOLOGICAL INVESTIGATION FOR BELFAST MALL, MPUMALANGA.

Attached please find herewith a report on the phase 1 geotechnical investigation conducted for the new proposed Belfast Mall on portions of the farms Wemmershuis 379 and Berg-en-Dal 378, Mpumalanga.

This report details and comments on the results of an engineering geological investigation conducted for the proposed residential, commercial and light industrial development. The purpose of the study was to investigate and identify areas that are suitable for the proposed development. This report provides details of the investigation methods adopted and also elaborates on the results of the various tests that were carried out and finally, the categorization of the geotechnical zones of the terrain.

The report is presented in two volumes with Volume 1 containing the findings of the investigation, with the drawings attached as Volume 2.

Yours Faithfully,

………………………………………
B.D. Cilliers *Pr. Sci.Nat.*
Engineering Geologist

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VOLUME 1

PHASE 1 ENGINEERING GEOLOGICAL INVESTIGATION
BELFAST MALL, BELFAST
MPUMALANGA

1. INTRODUCTION AND TERMS OF REFERENCE

A Phase 1 Geotechnical Investigation was conducted on Portion 6 of the farm Wemmershuis 379 and the adjacent Portion 3 of the farm Berg-en-Dal 378 for the proposed Belfast Mall. The site is located south of Belfast and adjacent to the N4 between Machadodorp and Wonderfontein and south-east of the R33 towards Carolina – refer to the *Locality Plan, Figure 1* in the beginning of the report.

The investigation was conducted according to the GFSH-2, 2002 for single storey residential buildings of masonry construction for development on sites larger than 10 hectares. The scope of work was outlined within the quotation document Q15-283BC dated 14th of September 2015. The appointment was confirmed by e-mail, dated the 1st of October 2015 from Mrs. L. Swarts Pr.Pln., authorised representative of the town planning firm Korsman & Associates CC of Witbank.

The field work was conducted during the last week of September 2015 and on completion of the test pitting and sampling, the soil samples were submitted to Letabalab (PTY)Ltd which is located in Witbank. The final results of the soil tests were received on the 20th of November 2015.

The objectives of the geotechnical investigation were to: -

i) Determine the soil and rock profile across the site and evaluate its engineering properties and influence on the design of light single storey structures.
ii) Establish depth to bedrock where not exposed.
iii) Evaluate the workability of the site materials with regard to their excavatability and compactability.
iv) Comment on predicted safe bearing capacity values, expected heave and settlement of the different potential founding horizons and recommend founding depths.
v) Assess the groundwater conditions, including surface run-off, ponding, seepage and perched or permanent water tables.
vi) Demarcate the site into various geotechnical zones with applicable NHBRC site classes and building procedures.

The report is presented in two volumes with *Volume 1* containing the findings of the investigation, with the drawings attached as *Volume 2.*

1.1 Site Details and Assumptions

The proposed development area of approximately 118ha in extent is located on Portion 6 of the farm Wemmerhuis 379 and the adjacent Portion 3 of the farm Berg-en-Dal 378. The N4 highway runs along most of the northern boundary of the terrain with the western boundary bordering the R33 district road from Belfast to Carolina. The eastern and southern areas border on to other portions of the two farms – refer to the *Site Plan, Figure 2, Volume 2*.

Access to the site is via a gravel track turn-off from the R33 district road along the western boundary. The site is located at 25°43'6.33" Southings and 30° 3'37.90" Eastings.

The site is characterised by a local watershed more or less parallel to the N4, dividing the terrain into northerly and southerly drainage areas. Surface run-off flows into a small earth embankment dam along the southern boundary and drainage from the N4 flows into two small borrow pits along the northern boundary. These impoundments were only partially filled and were used by the free roaming livestock.

The area is predominantly covered by knee-length Highveld grass whilst the area to the west of the farm shed has been grubbed and cleared. Clumps of black wattle trees are located along parts of the southern boundary, with some mature Eucalyptus trees scattered around the farm shed which is located in the central-western portion of the area investigated.

Farm houses, a number of workers houses, an old steel warehouse structure, six boreholes, cattle kraals and a satellite tower are some of the features noticed on the site. At the time of the investigation, the existing farm shed and buildings located in the north-western portion of the farm were leased to Opti-Power Projects (PTY)Ltd.

Part of the old tarred road from Belfast to Machadodorp has been fenced in along the north-eastern section of the northern boundary.

An ESKOM servitude traverses the site from the R33 in a westerly direction, changing towards the north-east in the central portion of the site.

The report is based on geological and hydrogeological information, test pitting, profiling and sampling of test pits, as well as the assessment of on-site pavement construction materials.

2. INFORMATION CONSULTED

The following geological, hydrogeological, geotechnical and topocadastral sources were consulted:-

i. Topographical map, sheet 2530AC to a scale of 1:50 000;
ii. Geological map, sheet 2530 Barberton published in 1986 to a scale of 1:250 000;
iii. Hydrogeological map, sheet 2526 Johannesburg, published in 1999 to a scale of 1:500 000;
iv. Satellite Images in digital format from Google Earth ©2015;
v. Walk-and-drive over survey.

3. METHODS OF INVESTIGATION

3.1 Drive-over-survey

A drive-over survey was conducted to establish drainage features, access roads and generally to obtain an overview of the site. An aerial photograph was used during the drive-over survey to assist in the site orientation, to determine the boundaries and to identify the general outlay of the proposed development.

3.2 Test Pitting and Profiling

Sixty four (64) test pits were excavated on the site using two tractor-loader-backhoes (TLB's). The test pits were excavated to the maximum reach of the backhoes or where slow progress or refusal was encountered. The test pit depths range from 0.70m to 3.60m below present ground level. The shallow - that is ~1.50m and shallower test pits - were entered whilst the deeper excavations were profiled from surface. The profiling included typical visual and tactile observations such as moisture, colour, consistency, structure, soil type and origin.

The soil profiles are attached as *Appendix A of Volume 1* and the positions of the test pits are indicated on the *Site Plan, Figure 2, Volume 2* and are also graphically presented by *Figures 5A & 5B, Profiles.*

3.3 Sampling and Laboratory Testing

Laboratory tests were performed on soil samples that were deemed to be representative of the cover soils, residual soils and ferruginised materials. The tests were carried out to confirm the in situ assessments of moisture, grading, plasticity, consistency, structure and to ascertain the engineering characteristics of each horizon.

The following tests were carried out on the samples: -

i) **Thirty two (32) indicator tests** comprising particle size distribution analysis and Atterberg Limit tests. In addition were the chemical properties also tested.
These tests permit a basic classification of the soils and group them according to typical engineering properties.

ii) **Seven (7) compaction tests** comprising Modified AASHTO moisture/density relationships and California Bearing Ratio Values and three cement stabilised tests.
These tests evaluate the compaction characteristics of the site soils and permit an evaluation of their suitability for use as construction materials.

iii) **Two (2) consolidation tests** using a single consolidometer at saturation was carried out. *This test evaluates the swelling and consolidation characteristics of the foundation soils and measures their void ratio change under varying load and moisture conditions.*

The original soil laboratory test results and summaries thereof are attached as *Appendix B, Volume 1.*

3.4 Test Pit and Topographical Surveys

The test pit positions were surveyed using a Garmin Oregon 650 hand-held GPS and plotted on the drawings whilst the surface contours on the appended drawings were transferred from existing topographical data.

3.5 Dynamic Cone Penetration Tests (DCP's)

Sixty four (64) hand-held dynamic penetration tests (DCP's) were carried out adjacent to the test pits and numbered accordingly. The penetration test results are attached as *Appendix C, Volume 1.*

The DCP or dynamic cone penetrometer in which a 60° cone with diameter of 20mm is driven into the soil by a 7.815kg weight dropped through 575mm. The results are expressed as millimetres penetrated per blow and refusal is achieved when 1mm penetration is recorded after 10 blows. The DCP is most useful for estimating consistencies or for assessing subgrade soils for road design. A crude approximation of the consistency and strength as well as the in-situ inferred CBR values can also be obtained.

4. GEOLOGY AND SITE SOILS

4.1 Geology

The site is underlain by sediments of the Vryheid Formation, Karoo Sequence in the western and eastern portions with older intrusive diabase in the central portion of the site – *see Figure 3.1, Regional Geology attached as Volume 2.*

However, during the test pitting phase the following geological bedrock discrepancies with the published geological map, Sheet 2530 Barberton were discovered, namely:- i) tillite of the Dwyka Formation and older quartzite of the Lakensvalei Formation, Transvaal Sequence that were exposed in central portion of the site, and ;- ii) the aerial distribution of the Vryheid Formation sediments on site seemingly cover larger areas than originally mapped – *refer to Figure 3.2, Site Soils and Geology.*

TABLE 4.1: STRATIGRAPHIC SEQUENCE OF THE SITE

Lithology	Formation	Group	Sequence
Sandstone & shale	Vryheid	Ecca	Karoo
Tillite	Dwyka	Ecca	Karoo
Diabase	Post Transvaal Intrusion		
Quartzite	Lakensvalei	Pretoria	Transvaal

The site investigated falls within a region with a Weinert N-Value of 2.4 indicating that chemical decomposition would be the dominant mode of weathering. Deep weathering of the older bedrock was noticeable – especially the diabase, Lakensvalei quartzite and Dwyka tillite. A thinner weathering profile was recorded in areas underlain by younger sediments of the Vryheid Formation.

4.2 Site Soils

A summary of the soil and bedrock profiles recorded during the test pitting phase of the investigation are presented by *Table 4.2* below.

TABLE 4.2 AVERAGE SOIL AND BEDROCK PROFILE

Soil/Bedrock Profile	Origin	*Average Thickness Range (m)	*Average Depth Range (m)	Engineering Characteristics
Imported material	Various origins	Surface to 0.6	Surface to 1.5	Low compressible, soft excavatable
Colluvium	Transported material	Surface to 0.60	Surface to 1.80	Medium to high compressible, soft excavatable, poor founding medium
Pebble marker	Transported material	0.25	0.1 – 2.1	Medium compressible, soft excavatable
Pedogenic material (hardpan ferricrete)	Pedogenic material	0.45	0.7 – 1.7	Intermediate to hard excavation class, good founding medium, susceptible to shallow perched water table
Residual Sandstone	In situ decomposed	1.0	0.9 – 1.9	Medium to low compressible, soft excavatable, fair founding medium
Residual Shale	In situ decomposed	0.85	0.9 – 3.2	Medium compressible, soft excavatable, poor to fair founding medium
Residual Diabase	In situ decomposed	1.65	0.1 – 3.4	Medium compressible, soft excavatable, fair founding medium with modifications
Residual Quartzite	In situ decomposed	1.75	1.4 – 1.8	Medium to low compressible, soft excavatable, fair to good founding medium
Residual Tillite	In situ decomposed	>1.2	>2.2	Medium to low compressible, soft excavatable, fair founding medium
Sandstone of the Vryheid Formation	Weathered Sandstone	N/A	>0.6 – 2.0	Intermediate to hard excavatable, generally very good founding medium
Shale of the Vryheid Formation	Weathered shale	N/A	>0.9 – 2.6	Soft to intermediate excavation class, good founding medium, susceptible to slaking
Quartzite of the Lakensvlei Formation	Weathered Quartzite	N/A	>1.95	Intermediate to hard excavation class, good founding medium
Tillite of the Dwyka Formation	Weathered Tillite	N/A	>2.2	Intermediate excavation class, good founding medium
Diabase	Weathered Diabase	N/A	>0.5 – 1.8	Intermediate to hard boulder excavation class, good founding medium

**Depths has been average using the test pit profiles.*

Copies of the test pit profiles are attached as *Appendix A* within Volume 1 and graphically indicated on *Figures 5A & 5B, Volume 2.*

The following soils and bedrock profiles were encountered on the site:-

4.2.1 Colluvium

The site is predominantly blanketed by transported silty sand (colluvium) to depths ranging between 0.20m to 1.80m with an average thickness of 0.60m. These transported soils consist of slightly moist, brown, loose to medium dense, silt-clay-sand mixes with a fissured structure. Grass roots appear in the upper 0.10m to 0.20m portion of the soil profile.

4.2.2 Pebble Marker

The colluvium is sequentially underlain by a pebble marker comprising sub-rounded ferricrete nodules and quartz gravels mixed with fine to medium grained silty sand. The pebble marker is well developed, usually occurs some 0.10m to 2.10m below surface with a recorded average thickness of 0.30m.

4.2.3 Pedogenic Materials

The transported materials – that is the colluvial horizon and the pebble marker - are underlain by low active, partially - to well cemented, ferruginised residuum. The latter occurs in some 15 test pits from as shallow as 0.70m to an average depth of 1.7m. The pedogenic material consists of soft powdery ferricrete concretions and nodules with soft ferruginised zones in a matrix of clayey, silty sand. Well cemented, honeycomb hardpan ferricrete was observed in sixteen test pits – refer to *Figures 5A & 5B Volume 2.*

4.2.4 Residual Sandstone

Moist light beige becoming ivory-beige with depth, medium dense, intact, medium grained silty sand derived from in situ decomposed sandstone of the Vryheid Formation was recorded in test pits along the eastern boundary of the site up to a depth of 1.90m below surface.

4.2.5 Sandstone Bedrock

Ivory-white and white yellowish beige, highly weathered to slightly weathered with scattered decomposed zones, widely bedded & jointed, medium grained, very soft to moderate hard sandstone rock of the Vryheid Formation was recorded in three test pits excavated within the eastern portion of the site (TP 52, TP54 & TP65). The sandstone pinches out further westwards – refer to test pit TP51.

4.2.6 Residual Shale

Slightly moist to moist, mottled yellowish, ivory creamy, orange brown, firm, intact and slickensided, fine grained sandy clayey silt with scattered flaky shale chips and in some test pits shale gravels with a snuff-box structure within a depth range of between 0.90m to 3.20m below surface. The shale residual soils were excavated within nineteen test pits and occurs in the western, northern and eastern portions of the site.

4.2.7 Shale Bedrock

Shale bedrock of the Vryheid Formation occurs in the western, northern and eastern portions of the terrain. The shale can easily be recognised by its beige colour with dark brown and light greyish stains, its thin and horizontally disposed bedding and medium spaced jointing. The depth to bedrock generally ranges between 0.70m to 2.0m below surface and the moderately weathered bedrock was generally soft to intermediate excavatable.

4.2.8 Residual Quartzite

Moist, maroon-orange-brown stained pink and streaked yellowish, medium dense, intact, fine grained silty sand was encountered test pits TP8, TP 13 and TP25 at an average depth of 1.60m below surface. The residual quartzite were excavated near the western boundary and central portion of the site.

4.2.9 Quartzite Bedrock

Dull ivory with maroon and orange brown relict stained fracture surfaces, thinly bedded and close to medium jointed, very soft to moderate hard quartzitic bedrock was encountered in the central portion and within the north-western corner of the site. The soft to intermediate excavatable bedrock ranges between 1.40m to 1.80m below surface.

4.2.10 Residual Tillite

Tillite of the Dwyka Formation was encountered at an average depth of 0.90m in three test pits, namely TP23, TP35 and TP55 – that is two in the western portion and a single test pit (TP55) in the eastern portion close to the boundary. The Tillite comprises scattered hard angular and sub-angular black stained clayey quartzite gravels and pebbles in a matrix of slightly moist to moist, ivory streaked dull grey and light yellowish brown, firm, intact, sandy clay.

4.2.11 Residual Diabase

The profile of the residual diabase with its predominantly maroon colour comprises soft to firm, intact, fine slickensided and pin-holed, fine to medium grained, sugary textured sandy clayey silt. Widely scattered to abundant spheroidal to sub-angular diabase gravels, cobbles and boulders do occur within the profile and also tend to form prominent north trending ridges. The residual diabase occurs from surface to depths in excess of 3.40m – the maximum reach of the TLB's boom.

4.2.12 Diabase Bedrock

Fractured diabase was encountered in eight test pits, namely TP5, TP18, TP28, TP29, TP43, TP44, TP49 and TP53. However as mentioned above, some boulders also occur on surface at these localities– refer to *Appendix A, Soil Profiles attached to Volume 1*. Fractured diabase is generally intermediate to hard excavatable – even at a shallow depth of 0.80m.

5. GEO-ENVIRONMENTAL FACTORS

5.1 Groundwater

Groundwater conditions were not investigated in detail, as this would form part of a hydro-geological investigation, which was not included in the brief.

5.2 Seepage, Surface Run-off and Subsurface Drainage Conditions

The site is located on a local watershed and falls within two Quaternary catchments, namely X11D and X21F. The largest part of the site drains in a southerly direction, whilst the northern portion drains northwards towards the N4 highway - refer to *Figure 4, Site Drainage* attached to Volume 2. A number of road culverts draining towards the northern boundary of the site were noted along the N4 highway and are indicated on *Figure 2, Site Plan, Volume 2.*

A superficial deposit of hardpan ferricrete seemingly present as a continuous horizon blankets the central-western portion of the site, covering some 30.9ha. This indurated and strongly cemented, massive rock-like horizon of less than a metre thick is commonly associated with a fluctuating water table. Although no seepage was encountered during the investigation which was conducted in the dry part of the year, a rebound of a shallow water table is expected during good rainfall periods in the area underlain by hardpan ferricrete.

Elsewhere, groundwater seeps were absent and the test pits were ostensibly dry. These dry conditions are attributed to the sloping topography, good run-off and the fact that the investigation was carried out during the dry season of the year.

5.3 Hydraulic Conductivity

No specific hydraulic conductivity tests were undertaken on site. However, the following hydraulic conductivity parameters, estimated from the soil classifications are provided:-

TABLE 5.3.1: HYDRAULIC CONDUCTIVITY

Material Type	Permeability *K* cm/s
Sandy gravelly colluvium - weakly cemented sandy residuum and imported materials	1×10^{-1} to 1×10^{-3}
Sandy and silty residuum - well-cemented pedocrete.	1×10^{-3} to 1×10^{-6}
Silty clayey residuum – sedimentary and metamorphic bedrock	1×10^{-6} to 1×10^{-9}

Permeabilities are expected to be high in the overburden materials due to the high sand fraction and fine gravel content. Any contamination is likely to move fairly rapidly within the colluvial cover soils and pebble marker, while the partially cemented pedogenic zones and sedimentary residuum will be less permeable.

5.4 Undermining

The site is not undermined and the nearest, non-operational open cast mine is situated approximately 800m north of the site on the farm Geluksoord 343.

6. FOUNDATION ASSESSMENT

The geotechnical appraisal is based on the field observations, soil laboratory test results and hand-held penetration tests.

The transported soil, residuum and bedrock strata have been tested and examined to determine their suitability as founding horizons for the proposed development according to the following criteria:-

- Strength and bearing capacities of the founding materials determined from estimated field consistencies and inferred from tabulated strength values.
- Compressibility of the founding materials measured from laboratory test results, expressed in terms of their coefficients of compressibility and estimated deformation moduli.
- Potential heave, where appropriate, in the residual soils as determined by the van der Merwe and Weston methods.
- Predicted displacements (settlement/collapse/heave) from the above factors.
- Slope stability.

A summary of the foundation assessment is provided on the following page.

6.1 Estimation of Allowable Bearing Capacity

The site is blanketed by transported and imported soils of various origins and thicknesses, underlain by residual and pedogenic soils, generally followed by decomposed to moderate weathered shale, sandstone and older quartzite, tillite and diabase bedrock.

The transported and imported cover soils are characteristically loose to medium dense and are usually underlain by loose (soft) and medium dense (firm) residual soils. Remnants of the old tarred road's pavement layers adjacent to the northern boundary were found to be dense to very dense. The flat, western portion of the site is characterised by shallow superficial, well-cemented hardpan ferricrete less than a meter in thickness – *refer to Figure 3.2, Site Soils and Geology, Volume 2.*

The presumed bearing capacities are provided in the table on the following page, based on in situ profile observations correlated with the DCP and CBR tabulated values. Note that bearing capacity refers to the ability of the *foundation soil* to withstand the load imposed without undergoing catastrophic shear failure. It therefore does not indicate the settlement that may occur in the soil under the applied pressure, which could lead to performance failure of the *structure*.

Bearing capacity tables provide an indication of the soil's bearing capacity based on it's composition and consistency and allow for settlement of up to 25mm. Detailed settlement analyses for a variety of footing designs are therefore required to optimize the bearing pressures to provide a tolerable settlement of the proposed structure(s).

TABLE 6.1.1: ESTIMATED PRESUMED BEARING VALUES OF SILTY SANDS (TERAZAGHI & PECK, 1967)

Soil/Bedrock Profile	*Average Thickness Range (m)	*Average Depth Range (m)	Consistency	DCP mm/blow	In situ CBR (%)	Estimated Bearing Values (KPa)
Imported material	Surface to 0.6	Surface to 1.5	Medium dense to dense	12 – 30 5 – 12	6 – 20 20 – 50	200
Colluvium	Surface to 0.60	Surface to 1.80	Loose to medium dense & medium dense	30 – 75 12 – 30	2.5 – 3.5 6 – 20	75
Pebble marker	0.25	0.1 – 2.1	Medium dense	12 – 30	6 – 20	75
Pedogenic material (hardpan ferricrete)	0.45	0.7 – 1.7	Medium dense to dense	12 – 30 5 – 12	6 – 20 20 – 50	150 – 200
Residual Sandstone	1.0	0.9 – 1.9	Medium dense	12 – 30	6 – 20	100 - 150
Residual Shale	0.85	0.9 – 3.2	Soft and firm	30 – 75	2.5 – 3.5	35 – 75
Residual Diabase	1.65	0.1 – 3.4	Soft and firm, Firm	30 – 75 12 – 30	2.5 – 3.5 6 – 20	75
Residual Quartzite	1.75	1.4 – 1.8	Loose to Medium dense	12 – 30	6 – 20	75 - 150
Residual Tillite	>1.2	>2.2	Firm	12 – 30	6 – 20	75 - 150
Sandstone of the Vryheid Formation	N/A	>0.6 – 2.0	Medium to hard	0	>110	>200
Shale of the Vryheid Formation	N/A	>0.9 – 2.6	Soft to medium hard	0	>110	>200
Quartzite of the Lakensvlei Formation	N/A	>1.95	Medium to hard	0	>110	>200
Tillite of the Dwyka Formation	N/A	>2.2	Soft to medium hard	0	>110	>200
Diabase	N/A	>0.5 – 1.8	Medium to hard	0	>110	>200

* ***Estimated allowable bearing capacity from DCP N-value and CBR values.***

Note 1. The estimated presumed bearing values of the foundation materials are only an empirical guide to the maximum load that can be placed on the soil/weathered rock without shear failure, and as such do not account for settlement (or heave) that may occur at foundation pressures up to the bearing capacity of the soil.

Note 2. The allowable bearing capacity includes a factor of safety of approximately 2 to 3 on design loads (presently not known), which in addition to reducing the likelihood of shear failure, accommodates predicted settlement to within tolerable limits.

Note 3. The presumed bearing values above are based on the materials exposed in situ in the test pits and ignore any improvement, which may be obtained by compacting, or treating the site soils.

The cover soil's consistencies vary between loose and loose to medium dense, therefore these materials are not recommended as a general founding layer without some remedial action. The residual shale and diabase recorded soft to firm consistencies to depths ranging from 0.10m to 3.20m below surface. These materials are considered unsuitable for founding in its natural state - even light structures with foundation pressures less than 75KPa will require some foundation improvements – refer to *Figure 6 – Potential Collapsible soils, Volume 2.*

The site's underlying pedogenic soils, sandstone-, quartzite- and tillite residuum (average depth range between 0.70m to 1.20m) and weathered bedrock are medium dense to dense in consistency and may be used for foundations, provided they are in the medium dense substrate, otherwise some compaction of the foundation materials will be required – refer discussion below on settlement.

The hardpan ferricrete, highly weathered shale-, sandstone-, quartzite-, tillite- and diabase-bedrock that are present from >0.60m to >2.20m have a dense consistency and hence a higher presumed bearing capacity adequate for double storey structures. Larger structures with multi-storeys will require *individual investigations.*

*Note that the above founding depths have been averaged over the site and this has been based on the soil profile information. However, the actual founding depths could vary – especially where terraces along the steeper slopes are implemented.

6.2 Estimated Compressibility

The results of the hand-held DCP tests and the visual assessment of the soil profiles have been interpreted into the compressibility descriptions given in *Table 6.2.1* below.

TABLE 6.2.1: ESTIMATED COMPRESSIBILITY / DEFORMATION

Material Description	Consistency	Deformation Modulus (MPa)	Foundation Rating
Imported material	Medium dense to dense	26 – 40	N/A
Colluvium	Loose to medium dense & medium dense	11	Fair
Pebble marker	Medium dense	11	Fair
Pedogenic material (hardpan ferricrete)	Medium dense to dense	26 – 40	Good
Residual Sandstone	Medium dense	26	Good
Residual Shale	Soft and firm	4 – 11	Poor
Residual Diabase	Soft and firm, Firm	11	Fair
Residual Quartzite	Medium dense	11 - 26	Fair to Good
Residual Tillite	Firm	11 – 26	Fair to Good
Sandstone of the Vryheid Formation	Medium to hard	>68	Very Good
Shale of the Vryheid Formation	Soft to medium hard	>68	Very Good
Quartzite of the Lakensvlei Formation	Medium to hard	>68	Very Good
Tillite of the Dwyka Formation	Soft to medium hard	>68	Very Good
Diabase	Medium to hard	>68	Very Good

** Depth recorded from final box-cut*
*** Foundation rating scale for footings located on or in the respective soil/rock horizons*:

Very Poor	*Insufficient bearing capacity and excessive settlement (>25mm)*
Poor	*Marginal bearing capacity, but excessive settlement (>25mm)*
Fair	*Adequate bearing capacity, but moderate to high settlement (15- 25mm)*
Good	*Adequate bearing capacity and manageable settlement (5-15mm)*
Very Good	*Adequate bearing capacity and negligible settlement (<5mm)*

The least compressible horizons are the well cemented pedogenic material and slightly weathered bedrock materials. The residuum is generally medium dense and has a low to medium compressibility and is therefore acceptable for single storey structures with some foundation modifications.

6.3 Active Clays

The cover soils, quartzite- and sandstone residuum were tested for activity and found to be either non - or low active and therefore problems of heave beneath foundations are not anticipated. Active clays occur in the residual clayey soils with the highest activity being "Medium" for the shale and "Medium to High" in the diabase residuum according to the Van der Merwe classification - refer to *Appendix B, Laboratory Results*.

6.4 Evaluation of Potential Settlement

Assuming an average wall load of 12.5kN per metre length of strip footing, the settlement within the loose colluvium and transported soils is expected to be between 10 – 15mm – hence the 'modified normal construction technique' requirements. Major modifications are recommended where the very loose (soft), clay-silt-sand soils are expected to collapse (settlement 15 - 25mm). The non-active residual soil horizon's collapse is expected to be less than 10mm, requiring normal construction techniques. In the dense, decomposed to highly weathered bedrock and hardpan ferricrete, settlement is not expected to exceed 5mm.

A summary of the estimated settlements of the various founding horizons and their corresponding (average) depths below surface is presented below.

TABLE 6.4.1: SETTLEMENT ANALYSIS SUMMARY

Soil/Rock Description	*Predicted Void Ratio	Collapse Potential	Estimated Settlement (mm)
Imported material	N/A	N/A	Not recommended
Colluvium	0.53 – 0.9	Low to medium	15 ~ suitable
Pebble marker	0.5 – 1.2	Low	5 – 10 ~ suitable
Pedogenic material (hardpan ferricrete)	0.34 – 0.45	Low	5 ~ suitable
Residual Sandstone	0.3 – 0.4	Low to medium	10 – 15 ~ suitable
Residual Shale	0.37 – 0.62	Low	10 ~ suitable
Residual Diabase	0.62 – 1.62	Medium to high	15 – 25 ~ suitable (require modification)
Residual Quartzite	1.2– 2.0	Low to medium	10 – 15 ~ suitable
Residual Tillite	0.59 – 1.83	Low to medium	10 – 15 ~ suitable

**Predicted void ratios are inferred from tabulated values.*

Based on the predicted settlement analyses, it is recommended that spread and/or strip footings be founded at a depth not less than 0.5m below surface, and that bearing pressures are not to exceed 50kPa (maximum contemplated for single storey structures). Footing bases should be compacted prior to construction of the foundations, where these footings are still in loose (soft) residuum. As an approximate guide, footing excavations should be deepened by some 100mm for every additional 10kPa contemplated above 50kPa.

Where heavier loads are anticipated, the choice of foundation solution will depend largely on individual sites, for which separate investigations will be required.

6.5 Workability of Site Materials

6.5.1 Excavation Characteristics

Based on the excavation depths achieved by the TLB, the excavatability of the site soils and bedrock is presented on the *Excavation Map*, *Figure 8, Volume 2* and classified according to SABS 1083 as follows:

TABLE 6.5.1: EXCAVATABILITY SUMMARY

Material Type	Excavatability SABS 1083	Proposed Excavation Method
Imported material	Soft	TLB
Colluvium	Soft	Hand, TLB
Pebble marker	Soft	Hand, TLB
Pedogenic material (hardpan ferricrete)	Soft to intermediate	TLB, excavator
Residual Sandstone	Soft	Hand, TLB
Residual Shale	Soft	Hand, TLB
Residual Diabase	Soft	Hand, TLB
Residual Quartzite	Soft	Hand, TLB
Residual Tillite	Soft	Hand, TLB
Sandstone of the Vryheid Formation	Intermediate to Hard	Excavator, Blasting
Shale of the Vryheid Formation	Intermediate to Hard	Excavator, Blasting
Quartzite of the Lakensvlei Formation	Intermediate to Hard	Excavator, Blasting
Tillite of the Dwyka Formation	Intermediate to Hard	Excavator, Blasting
Diabase	Intermediate to Hard Boulder	Excavator, Blasting

Pick/shovel and TLB mechanical excavation operations will be adequate to excavate through the transported and residual materials to a suitable founding material for single storey structures.

No problems are anticipated with excavating to an average depth of 1.5m below surface in residual soil with TLB's. However, deep service trenches within shallow, intermediate to hard excavatable hardpan ferricrete and bedrock will require hard ripping, powerful excavator or blasting.

Boulder excavation will be required where diabase outcrops along the thin ridge traversing the site at test pit positions TP5, TP12, TP34 and TP24 – refer to *Figure 8, Excavation Map, Volume 2.*

6.5.2 Compaction Characteristics

Seven (7) disturbed samples representative of the various site soils were taken and submitted for compaction tests – refer to the laboratory test results attached as *Appendix B*. The transported, residual and weathered bedrock material present at the various depths were tested and found to comply with the operational requirements of the following pavement construction material classes – refer to *Table 6.5.2* below.

TABLE 6.5.2: COMPACTION TEST RESULTS

Test pit No	Sample No	Depth (m)	Origin	MDD	OMC	CBR/UCS at % Compaction							Soil Classifications	
						100	98	97	95	93	90	Swell	*Unified	COLTO
TP2	DS2B	0.8-1.7	Diabase	1762	14.7	31	28	25	22	15	9	0.24	SC	G8
TP10	DS10A	0.7-2.5	Diabase	1883	15.4	18	16	15	12	11	8	0.26	SC	G8
TP32	DS32A	0.3-0.7	Ferricrete	2038	7.1	84	75	71	63	37	17	0.14	SM/SC	G5
TP48	DS48A	0.0-0.3	Remnants of Imported base & subbase	2039	6.5	55	47	43	37	24	12	0.14	SP	G6
TP59	DS59A	0.6-2.1	Shale	1938	15.1	18	13	12	9	8	6	0.81	ML	Spoil
TP61	DS61A	0.1-1.8	Colluvium	1935	8.8	74	53	44	31	24	16	0.12	SM	G7
TP65	DS65A	0.7-1.9	Sandstone	1905	6.2	49	39	34	27	22	17	0.31	SM	G7

Note: MDD – Maximum dry density OMC – Optimum Moisture Content CBR – California Bearing Ratio

The transported (colluvial) material generally classifies as G6-G7 class pavement construction material, whilst the hardpan ferricrete classifies as G5 type which is suitable for the construction of base - and subbase pavement layers.

The diabase-, sandstone and shale residuum classify as G7, G8 and spoil class pavement construction materials and are mainly suitable for fill or selected layers.

Materials for base course and sub-base materials for roads of the proposed development can be obtained from the remnants of the old TPA road located adjacent to the northern boundary where some 1170m^3 of G6 class pavement construction material is available – refer *to TP48, Appendix A and Figure 7*, *Available Pavement Layer Construction Materials, Volume 2.*

6.6 Slope Stability

Much of the eastern portion of the site has a gentle to moderate slope towards the south. While steep slopes are not in themselves reason to restrict development, they can result in additional development costs, due to:

1. Steep surface run-off and associated flood and erosion protection measures.
2. Possible slope instability, especially where the natural slope has been altered.
3. Large cut-to-fill terraces and potential for differential settlement where compaction is not well controlled.
4. Requirement for retaining walls, or flattening of slopes where deep (>3m) terrace cuts are required.

Although no shear strength tests were carried out at this stage of the investigation, the grading and relatively low plasticity of the founding material indicates that angles of internal friction of between 10º-30º can be anticipated – Reference 14 in the Bibliography. Consequently, natural slope failures are not anticipated, however the cautionary comments above should be noted when service and foundation excavations > 1.50m are undertaken, especially in areas where seepage is anticipated.

6.7 Special Precautionary Measures

It is important that the surface water be collected and disposed of in well-designed stormwater channels to minimize ingress, the wetting up of foundation soils and to nullify future collapse settlement of loose soils or heaving of cohesive soils subjected to loading.

The test pits were backfilled using the TLB without proper compaction. Should the foundations of the structures be positioned on these test pit excavations, the backfilled material must be properly compacted to prevent differential settlement – refer to *Figure 2, Site Plan* for test pit positions.

7. DEVELOPMENT RECOMMENDATIONS

The site has been categorized as a development zone with specific geotechnical characteristics – that is Zones 1A and 1B, 2A, 3A, 4A to 4D depending on the individual geotechnical constraints – refer to *Tables 1 & 2 attached as Appendix D and the Zonal Map – Figure 9, Volume 2.*

Zone 1A - Normal Founding = Size 38.4 Ha

This zone comprises low compressibility soils of less than 1m thick, with collapsing sandy soils less than 750mm thick that occur at foundation level, which will require little, or no modifications to normal building construction techniques. Foundation settlement in these areas is not expected to exceed 5mm - 10mm and in terms of the NHBRC site specifications, this portion of the terrain is defined as NHBRC Classes C and S - refer to *Tables* 1 & 2 in *Appendix D*.

Normal precautions including adequate drainage away from the building, flexible water connections, grass, or concrete aprons around the buildings and moderate compaction in the base of foundation excavations prior to the casting of the foundations are recommended.

Zone 1B – Modified Normal Construction = Size 29.97 Ha

This zone comprises moderately compressible soils between 0.75m and 1.5m thick, with collapsing sand sandy soils more than 750mm thick. Consequently loose to medium dense soil, being moderately compressible occurs beneath the foundations to a depth of 1.5m which is expected to induce settlements of between 5mm and 10mm unless construction is modified to accommodate these differential movements. NHBR Classes C1 and S1 – refer to *Tables 1 & 2 in Appendix D.*

Precautions including, but are not limited to the compaction of foundation soils to at least 93% of Modified AASTHO density to a depth of 1.5 times the foundation width, light reinforcement in foundations and masonry, articulated joints at doors and lintels and additional drainage, service and plumbing precautions.

Zone 1C – Comprehensive Modified Construction = Size 45.08 Ha

This zone comprises moderately compressible soils more than 1.5m thick, with collapsing sandy soils more than 750mm thick. Consequently loose to medium dense soil, having a moderate to high compressibility occurs beneath the foundations to a depth which will cause settlements of more than 20mm unless construction is modified to accommodate these differential movements. NHBR Classes C2 and S2.

Precautions including, but are not limited to the compaction of foundation materials to at least 95% of Modified AASTHO density to a depth equal to twice the foundation width, reduced bearing pressures (not to exceed 50kPa), moderate reinforcement in foundations (stiffened strip footing) and masonry, articulated joints at doors and lintels and additional drainage, service and plumbing precautions. Piling may even be considered as an option for movement sensitive structures – refer to *Tables 1 & 2* attached as *Appendix D*.

Zone 2A – Modified Normal Construction = Size 114.2 Ha

In terms of the NHBRC's site class, this zone classifies as H2. The foundation design and building procedures for single storey residential structures founded upon expansive soil horizons with heave in excess of >15mm require stiffened or cellular rafts, soil rafts or piled foundations. Site drainage and plumbing/service precautions apply. Where this zone overlaps with the zones above similar NHBRC site classes and building precautions apply – refer to *Tables 1 & 2 in Appendix D.*

Zone 3A – Shallow Bedrock = Size 8.68 Ha

This zone has relatively shallow bedrock and intermediate to hard excavatable diabase boulders, quartzite and hardpan ferricrete may be encountered within the foundation excavations – see test pits TP5, TP6, TP12, TP18, TP20, TP21, TP22, TP24, TP34, TP52 and TP58 – refer to Figure 8, Excavation Map, Volume 2. The anticipated NHBRC Site Class refers to difficult excavation shallower than 1.5m. NHBRC Class R.

Development may however proceed in these areas, although the Developer should be made aware that additional costs might be incurred for the excavation of service trenches.

Zone 4A – Susceptible to sub-surface seepage = Size 30.92 Ha

This zone is associated with a fluctuating seasonal water table and sub-surface seepage. Cut-off drains, subsurface and good surface drainage control measures will have to be implemented to prevent flooding, ponding and erosion of the loose cover soils. Note that where this zone overlaps with the zones above, similar NHBRC site classes and building precautions apply. NHBRC Class P.

Zone 4B – Remediation = Size 0.3 Ha

This zone has been previously used for borrow pit materials and was seemingly backfilled with loose imported materials. Impact rolling is recommended to pre-collapse overburden materials; note that compaction control is a prerequisite. NHBRC Class P.

Zone 4C – ESKOM Servitude = Size 10.10 Ha

This zone is reserved for the existing over-head ESKOM power line and no development is recommended within this servitude. NHBRC Class P.

Zone 4A – Earth Embankment dams = Size 0.75 Ha

This zone encompasses the earth embankment dams and no development is recommended within these features. NHBRC site Class P.

8. GENERAL

It is therefore recommended that foundation excavations be inspected at the time of construction by a competent person, to ensure that the materials are adequate for the proposed structures and that they are in accordance with the recommendations stated in this report. Furthermore the excavation of terraces and road cuts, and the placement of engineered fills must be controlled with adequate field tests to ensure that the quality and specified densities are achieved during compaction.

Every effort was made during the site investigation to ensure that generally accepted practices of our profession were used in the sub-surface evaluation of the site, and that the sampling and testing was representative of the soil/rock conditions observed on-site. However it is impossible under the constraints of a restricted investigation of this nature to guarantee that zones of poorer geological materials were not identified that could have a significant bearing on the outcomes of this investigation. The investigation has therefore attempted, through interpolation and extrapolation at known test locations, to identify problem issues of a geotechnical nature on which this report is based. Variances in soil and rock quality and quantity from those predicted may be encountered during construction and these should be recorded, however no warranty against these variations is expressed or implied, due to the geological changes that can occur over time due to natural processes, or human activity.

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