

**REPORT TO MEADOWOOD TRADING 8 ON A
GEOTECHNICAL FOUNDING INVESTIGATION OF REM OF PTN 146 (OF
4) OF THE FARM UPPER END OF LANGEFONTEIN No. 980, ON ROAD
D819, FOR THE PROPOSED FAIRWAYS OFFICE
AND MINI FACTORY DEVELOPMENT**

1. TERMS OF REFERENCE

As requested, Drennan Maud & Partners provided a proposal for the geotechnical investigation of this 3.7 ha site on 23 October 2007, our reference 91. This was duly accepted and an appointment form was received from Meadowood Trading 8 on 26 October 2007. The investigation was to include inspection pits, penetrometer testing and materials sampling in order to provide recommendations for founding, earthworks, drainage and construction materials during development of this site. It is understood that on-site sanitation is being investigated by others so no percolation test was carried out.

2. SITE DESCRIPTION

The site is an elongated west-east rectangle, situated on a south and east grading slope. Natural gradients are in the order of 4-6° over the western and central portions of the site, steepening considerably to about 25° on the far eastern portion. Limited terracing of the land appears to have been carried out in the past on the gentler gradients in the centre and west.

Drainage is to the south and east, ultimately flowing into a tributary of the Nkutu River in the east.

At present the grassed site appears to be used for grazing of the neighbours horses and as storage space for farm implements and vehicles.

3. FIELD WORK

Sixteen inspection pits, designated IP1 to IP16, were excavated using a TLB at the approximate positions indicated on the site plan, Figure 1. Subsoil profiles are included as Appendix 1. Depths of between 0.7 and 3.0m were achieved with an average depth of about 2m. Early refusal often occurred on weathered or ferruginised sandstone. No refusal was met in a number of pits.

The pits were supplemented by 15 dynamic cone penetrometer tests, designated DCP1 to DCP15. Test results are included as Appendix 2. Depths of 0.6 to 3.0m were achieved before refusal on very stiff to hard soils or weathered bedrock.

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Bulk disturbed materials samples were collected from representative horizons in the inspection pits. These four samples were returned to Thekwini Soils Laboratory for testing to determine the full indicators, Modified AASHTO density and CBR values. These results are summarised, together with the materials analyses, in Appendix 3.

4. GEOLOGY

The region is underlain by Natal Group sandstone with subordinate siltstone bedrock and the associated sandy or clayey residual soils. On this site, the bedrock is typically a highly to completely weathered, red orange, yellow orange and very light grey, very soft rock sandstone. This may be ferruginised to varying degrees which can make it hard in places.

Where present, the residual soils tend to be red orange and/or yellow orange, firm to stiff, slightly fissured, silty or occasionally fine sandy clays. These too may include ferricrete nodules or cemented lenses. This horizon may extend to in excess of three metres depth.

Transported soils are typically dark grey brown, firm to stiff, silty or sandy clays. Layer thickness ranges between about 0.3m and 0.8m.

5. LABORATORY TEST RESULTS

5.1 Weathered Sandstone

Two samples were tested. These proved to range from a clayey sand to a sandy clay with 10 to 36% clay content and grading moduli of 0.83 and 2.17. The plasticity indices were 12 and 13 while linear shrinkages of 6.0 and 6.7% were measured. In terms of the Revised US Classification, these are A-2-6(0) and A-7-6(6) materials.

Modified AASHTO maximum dry densities of 1778 and 1946kg/m³ were recorded for the clay and sand with optimum moisture contents of 19.0 and 13.5% respectively. CBR values were 16 and 17 at 90% of Mod density, increasing to 39 and 40 at 100% of Mod AASHTO. The maximum CBR swell was 0.17%, hence these are G8 class materials in terms of the TRH 14 (1985) specifications.

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5.2 Residual Sandstone

Two samples collected graded as fine sandy or silty clays with 68 and 74% clay content and grading moduli of 0.13 and 0.22. The plasticity indices were 12 and 14. Linear shrinkages of 6.0 and 7.3% were recorded. In terms of the Revised US Classification, these are A-6(14) and A-7-5(13) materials.

The Modified AASHTO maximum dry densities were 1498 and 1404kg/m³ with optimum moisture contents of 26.3 and 32.0% respectively. CBR values were 7.6 and 9.0 at 90% of Mod AASHTO, increasing to 17 and 22 at 100% of Mod density. The maximum CBR swell was measured at 0.55%. Hence these are G9 class materials in terms of the TRH14(1985).

5.3 Colluvium

The single sample tested proved to be a fine sandy silty clay with 48% clay content and a grading modulus of 0.23. The plasticity index was 12 and the linear shrinkage 6.0%. This is an A-6(10) material in terms of the Revised US Classification.

The Modified AASHTO maximum dry density was noted to be 1448kg/m³ and the optimum moisture content 28.4%. CBR values were 4.2 and 14 at 90 and 100% of Mod density respectively. The maximum CBR swell was considered excessive at 1.69% and hence this material cannot be classified in terms of the TRH14.

6. GEOTECHNICAL ASSESSMENT

The site is considered stable in its current conformation and capable of the one and two storey office and mini-factory development as proposed provided the development is carried out with all due caution to accommodate the prevailing geotechnical conditions. Potential constraints would include the ferruginised layers possibly being difficult to excavate, potentially active clay soils affecting founding, variable depths to competent founding and seasonal shallow seepage affecting sub-basement levels.

7. RECOMMENDATIONS FOR DEVELOPMENT

7.1 Excavatability and Earthworks

The site is currently gently terraced. From the final floor levels provided by the engineer, it is evident that cuts of up to three metres are planned for the creation of platforms.

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Excavation to depths of 1.05 to in excess of three metres were confirmed in confined trenches using a TLB but it is probable that greater depths could be achieved in an open platform excavation or using an excavator (the shallow rock at 1.05m is under fill so will not pose a problem for excavation).

The ferruginised layer is erratic in depth and degree of cementation and will have to be treated symptomatically as and when found. Competent sandstone bedrock also varies in depth. Pneumatic tools may be required to remove bedrock or ferricrete at depth in places. Blasting is not expected to be necessary.

Fills of up to about two metres are planned for levelling of platforms. Prior to the placing of fill, all vegetation must be grubbed clear and the new embankment must be constructed in compacted layers. On the steeper eastern portion of the site, where natural gradients exceed 10° (or 1:6), the fill must be benched in to suitable insitu materials to enhance overall slope stability.

Where clay soils are used as fill the layers should not exceed 200mm loose thickness and a pad-foot roller should be used to achieve no more than 90% of Mod density compaction. The sandier materials may be placed in layers of 300mm loose thickness and compacted to 93-95% of Mod density.

Unsupported cut and fill batters should be restricted to about three metres height. These should be trimmed to no steeper than 1:1.5 and vegetated to reduce the risk of erosion in the long term. If this gradient cannot be accommodated within the geometry of the site boundaries or if the batter chases the slope in the east, a gravity type retaining wall must be considered to support the bank.

7.2 Subsurface Seepage

While slight seepage was only detected in two inspection pits (2.4 and 2.8m in IP10 and IP8 respectively) at the time of the investigation, the widespread presence of ferricrete suggests long term, seasonal soil moisture level fluctuations. All retaining walls and surface beds must be suitably damp-proofed and toe drains are recommended behind all retaining walls.

7.3 Site Drainage

Storm water run-off from roofed and paved areas must be controlled, collected and piped to discharge into an attenuation pond or equivalent structure, ultimately to discharge into the stream on the north eastern boundary. Alternatively, storm water soakpits may be used but the predominantly clayey subsoils may prove to have a limited permeability for the use of on-site soakpits for the expected large volumes of water that will be generated by the extensive hardened areas in this development.

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Soils derived from the Natal Group sandstone are typically prone to erosion by wind and/or flowing water. While the cohesive clayey soils on the central and western portions of this site are not considered highly erodible, the steep slope to the north east has limited soil cover and concentrated flow over the crest of this slope will almost definitely cause significant scouring. Surface run-off should be directed back onto the platform rather than over the crest of the batters.

7.4 Founding

Depths to competent founding are highly variable : from about 0.7 to in excess of three metres below present ground levels. Shallower refusal of the DCP in some locations unfortunately cannot be guaranteed as competent founding on this site due to the presence of the partially cemented ferricrete layer; competent founding is inferred from visual assessment of materials exposed in the inspection pits. In places, founding depths will be reduced by up to three metres by cutting or increased by up to two metres by filling. The engineer has advised expected loads in the order of 200kN.

Structures founded across platforms underlain by cut, natural soils and fill will be subjected to differential settlements that can lead to cracking of the building. Ideally, foundations must be taken into like material all round to reduce the potential for differential settlement.

Any new fill can be expected to continue to settle into the medium term (being predominantly clayey in nature) at up to about 1% of fill thickness if initially well placed and compacted. For the two metre fills in the centre and east, this amounts to about 20mm settlement for fill consolidation alone.

Ideally, the structures should be founded on ground beams spanning isolated footings or mini piles taken through all fill and clayey residual soils into the competent, highly weathered sandstone at depths estimated between 1.0m and some four to five metres below present ground levels (hence effective pile lengths of up to about 7m where through fill). Hard hand picking for removal of the sandstone will effectively provide a bearing capacity of 200 - 250kPa for the footings. Foundation trenches must be inspected and certified to ensure that the base of the trench is in weathered rock and not just in a partially cemented lense of clayey ferricrete as these lenses are not continuous. An auger bit (with new teeth) for a 200-300mm or larger pile should not be unduly impeded by the partially cemented clayey ferricrete lenses as seen in the inspection pits; hence refusal of the auger would be expected to be on competent sandstone bedrock.

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A beam-stiffened raft foundation could also be designed to accommodate differential movement due to consolidation and seasonal soil volume changes in the underlying soils. This would largely circumvent the potential challenges of the variable founding depths. Due to the size of the structures, the rafts would likely be compartmentalised and construction joints between the segments would accommodate any differential movements.

Alternatively, the structure may be founded on ground beams spanning isolated footings through all fill into stiff, natural soils at least 1.5m below platform level. At this depth, the soils are to all practical purposes moisture constant and seasonal variations in soil volume will be limited. Stiff clays would provide in the order of 100 - 150kPa bearing capacity. The structure must include judiciously placed construction joints to accommodate any seasonal movement or consolidation settlement as may still occur. This founding method may prove practically challenging where fill is two metres thick, taking footings to at least 2.5-3m depth to achieve stiff, natural soil. It is also possible that in places (for example the central southwestern portion) weathered rock may be encountered within that 1.5m depth. This would result in the structure being variably founded on weathered rock and stiff clay. Differential settlements could occur and to a degree could be accommodated by the construction joints; this situation is not ideal.

7.5 Construction Materials

Both the residual soils and deeply weathered sandstone bedrock are considered fair to poor as subgrade below pavement layer works or surface beds. The colluvium has a slightly heave potential if over compacted. Layer works and slab design should take this shortfall into account.

8. CONCLUSION

The gently grading site is considered stable in its current conformation and capable of the proposed low level office / mini factory development. The steep eastern portion, while also considered stable in its natural state, is not recommended for future development.

Depths to competent founding across this site are quite variable particularly after cutting and filling to create the platforms, ranging from about 1.0m to 4 - 5m below natural ground. Structures should ideally be founded into competent bedrock by means of isolated footings or mini piles. Effective founding depths of about 1.0m up to about 7m are expected. Alternatively, the structure may be founded on a suitably stiffened raft designed to accommodate the seasonal soil volume fluctuations and consolidation settlement.

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Storm water run-off must be controlled to prevent erosion of the steeper slopes to the east or below this site to the south. The clayey soils also have a limited potential for seasonal soil activity / volume changes so standing water near the foundations could pose a problem in the future.

Insitu clay soils and deeply weathered bedrock are considered fair to poor subgrade. All engineering designs must take that into account. All other construction materials will have to be imported from a suitable, local, commercial source.

(Mrs) D.J. ABEL Pr.Sci.Nat.

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**DRENNAN, MAUD AND PARTNERS
68 Ridge Road, Tollgate,
DURBAN, 4001**

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NOVEMBER 2007

DRENNAN, MAUD AND PARTNERS
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CONTENTS

	Page N°
1. TERMS OF REFERENCE	Page 1
2. SITE DESCRIPTION	Page 1
3. FIELD WORK	Page 1
4. GEOLOGY	Page 2
5. LABORATORY TEST RESULTS	Page 2
5.1 <u>Weathered Sandstone</u>	Page 2
5.2 <u>Residual Sandstone</u>	Page 3
5.3 <u>Colluvium</u>	Page 3
6. GEOTECHNICAL ASSESSMENT	Page 3
7. RECOMMENDATIONS FOR DEVELOPMENT	Page 3
7.1 <u>Excavatability and Earthworks</u>	Page 3
7.2 <u>Subsurface Seepage</u>	Page 4
7.3 <u>Site Drainage</u>	Page 4
7.4 <u>Founding</u>	Page 5
7.5 <u>Construction Materials</u>	Page 6
8. CONCLUSION	Page 6
<u>APPENDIX 1</u> -	INSPECTION PIT PROFILES (IP1 TO IP16)
<u>APPENDIX 2</u> -	DCP TEST RESULTS (DCP1 TO DCP15)
<u>APPENDIX 3</u> -	LABORATORY TEST RESULTS
<u>FIGURE 1</u> -	SITE PLAN

APPENDIX 1

**INSPECTION PIT PROFILES
(IP1 TO IP16)**

APPENDIX 2

**DCP TEST RESULTS
(DCP1 TO DCP 15)**

APPENDIX 3

LABORATORY TEST RESULTS

FIGURE 1

SITE PLAN