DRENNAN MAUD (PTY) LTD

GEOTECHNICAL ENGINEERS AND ENGINEERING GEOLOGISTS
Incorporating Drennan Maud & Partners (Est.1975) and GAP Consulting

Reg. No. 2014/038872/07

Durban Head Office
68 Peter Mokaba Ridge, Tollgate, 4001
P.O. Box 30464, Mayville, 4058
T: +27 31 201 8992 F: +27 31 201 7920

Info@drennanmaud.com
www.drennanmaud.com

Margate Office
Unit 7 Gayridge Business Park No. 2
13 Wingate Avenue, Margate 4275
T: +27 39 3122 588 F: 0866 0275 53

1OUR REF.: 23312
YOUR REF

9th March 2017

Tongaat Hulett Developments (Pty) Ltd
P.O. Box 22319
GLENASHLEY
4022

Attention : Mr H. Makwabe (Hlalelo.Makwabe@tongaat.com)

Dear Sirs,

GEOTECHNICAL INVESTIGATION PHASE 2 ASSESSMENT FOR THE PROPOSED DEVELOPMENT OF TINLEY MANOR ESTATE SOUTH BANK.

Further to your email of 9 March 2017, we confirm that we have reviewed the latest layout for the above-mentioned proposed development and report the on below.

1. Available Information

1.1 The following information was available for our assessment of the proposed development layout;

- Drennan Maud and Partners report Reference 23312, dated December 2012, titled, 'Report to Tongaat Hulett Development on the Geotechnical Desktop Phase 1 Assessment for the Proposed Development of Tinley Manor Estate South Bank.
- Drennan Maud and Partners report Reference 23312, dated May 2013, titled, 'Report to Tongaat Hulett Development on the Geotechnical Desktop Phase 2 Assessment for the Proposed Development of Tinley Manor Estate South Bank.

2. Development Layout

2.1 The proposed development layout was provided in the following format from yourselves;

- A copy o a PowerPoint slide showing the various changes to the original layout,
- A JPEG of the actual Block Layout Plan,
- A Land Use Table.

3. Geotechnical Assessment.

3.1 Based on our review of the current development layout and the results of our previous two geotechnical reports, we confirm that both reports, Reference 23312 dated December 2012 and Reference 23312 dated May 2013, are applicable to the current development layout and should be utilised in the design and implementation of the proposed Tinley Manor Estate South Bank development.

We trust that the above information meets you requirements. We will be pleased to furnish you with any additional information should this be required.

Yours faithfully
DRENNAN MAUD (PTY) LTD.

M.J. HADLOW Pr.Sci.Nat

/mh

REPORT TO

TONGAAT HULETT DEVELOPMENT

ON THE

GEOTECHNICAL INVESTIGATION

PHASE 2 ASSESSMENT

FOR THE PROPOSED

DEVELOPMENT OF

TINLEY MANOR ESTATE

SOUTH BANK.

DRENNAN, MAUD AND PARTNERS
CONSULTING CIVIL ENGINEERS AND
ENGINEERING GEOLOGISTS
68 Peter Mokaba Ridge
Tollgate, Durban, 4001

Ref № 23312

MAY 2013

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REPORT TO TONGAAT HULETT DEVELOPMENT ON THE GEOTECHNICAL INVESTIGATION PHASE 2 ASSESSMENT FOR PROPOSED DEVELOPMENT OF TINLEY MANOR ESTATE - SOUTH BANK.

1. INTRODUCTION AND TERMS OF REFERENCE

For the purpose of the proposed development of Tinley Manor - South, Drennan, Maud and Partners carried out a geotechnical investigation, in continuation of an initial desktop study, in order to fulfil the requirements of the Planning Authority.

We confirm that the Phase 2 - detailed investigation has been carried out and our observations and general recommendation guidelines for the development of the entire area are set out in this report.

2. INFORMATION SUPPLIED AND AVAILABLE INFORMATION

A compact disc containing various contoured and cadastral information of the site was supplied by Tongaat Hulett Developments for the purpose of the investigation.

In addition the following information supplied to us for the previous desktop geotechnical investigation, also relevant to this investigation, was available;

- 1: 15 000 scale 2009 Aerial photograph of the Tinley Manor South development area and surroundings provided by Tongaat Hulett Developments. The aerial photograph indicated the limit of the investigation area as well as wetland, flood plain and non-development area.

Furthermore, Drennan, Maud and Partners consulted various information during the coarse of the investigation which included the following;

- The 1:250 000 scale Geological map of Durban (2930)

- Relevant geotechnical reports of sites investigated in the nearby vicinity within similar geology and topography.

3. PROPOSED DEVELOPMENT

It is understood that the Tinley Manor South Bank, comprising an area of approximately 475 ha is to be developed for potential resort tourist, leisure, residential, commercial and mixed use activities. No master plan or zoning diagram for the proposed development area has been provided as yet. However, given the natural sloping topography of the area large scale cut and fill structural platforms will likely be necessary.

4. SITE DESCRIPTION

4.1 Location

The Tinley Manor South development area is located approximately 8km north east of Ballito on the Kwa-Zulu Natal north coast. The area lies directly adjacent to the Sheffield Beach area to the south and is bounded along its northern extend by the Tete River. The site which comprises an area of approximately 475 ha extends in a westerly direction from the coastline towards the N2 freeway approximately 2.5km inland, with a small portion of the development area located on the western side of the N2 freeway. The majority of the area is currently cultivated with sugar cane crop, however some areas covered in natural indigenous vegetation do occur mainly towards the eastern coastal portion of the site.

4.2. Topography and Drainage

The site comprises moderately to steeply undulating topography with slopes varying from having a convex to concave conformation. The convex topography represents well elevated topographic spurs generally trending in a north east - south west direction (parallel with the coast) while the concave topography generally represents the heads of minor non-perennial stream valleys etched into the underlying unconsolidated sediments and bedrock. The streams located across the central to western portions of the area drain in a north to north easterly direction towards the Tete River, while drainage lines across the more coastal portions of the site drain in a south easterly direction towards the ocean.

5. FIELD INVESTIGATION

The field work for the detailed geotechnical investigation was carried out over a total of 14 days spanning the months of October, November and December of 2012 due to various delays which included TLB availability and break down, staff availability as well as extended periods of very wet weather during the latter part of 2012.

The detailed geological field investigation comprised general geological mapping, the excavation of inspection and exposure logging, Dynamic Cone Penetrometer testing, percolation testing and material sampling for laboratory testing.

5.1 Inspection Pits

A total of 98 inspection pits were mechanically excavated using a TLB, either to refusal on rock or otherwise the full reach of the machine, approximately 3.0m. Given the cultivated nature of the majority of the investigated area as well as steep topography and marshy conditions in places, inspection pits were generally restricted to along open cane tracks. The locations of the inspections pits were captured using hand held GPS device with approximate 3.0m accuracy and are indicated on the site plan included as Figure 1 of this report. The soil material removed from the inspection pits was examined and logged in detail according to the South African Guidelines for Soil and Rock Logging. The subsequent soil profiles are included in Appendix A of this report.

The purpose of the test pitting was to;

- profile the subsoils across the investigated area and identify any problem soil types with regard to foundation design and stormwater drainage.
- identify problems related to potential slope stability
- establish the depth to bedrock if occurring at shallow depths.
- determine the presence of any perched or shallow ground water tables.
- collect representative subsoil samples for laboratory testing.

5.2 Exposure Logging

In addition to the 16 exposures described in the initial desktop phase report a further 9 exposures were identified across various portions of the site and logged. The GPS positions of all 25 exposures, designated Exp 1 - 25, are indicated on the site plan, Figure 1 of this report with the soil profile logs also included in Appendix A.

5.3 Dynamic Cone Penetrometer Testing

A total of 92 Dynamic Cone Penetrometer (DCP) tests, designated DCP 1 - 92, were carried out through out the study area at the GPS acquired positions indicated on the site plan, Figure 1. The DCP results are graphically presented in Appendix B.

The aim of the DCP testing was to;

- establish the consistency of the subsoils.

- establish the depth to bedrock if occurring at shallow to moderate depths.

In order to facilitate in the interpretation of the DCP results with respect to the consistency of the non-cohesive and cohesive soils underlying the investigated area, the following table is provided. However, it must be noted that this table is intended merely as a guide as is specific to DMP equipment.

Table 1 : Subsoil Consistency Inferred from the DCP Test Results

Cohesive Soils		Non-Cohesive Soils	
DCP Blow Count Blow / 300mm	**Subsoil Consistency**	**DCP Blow Count Blows / 300mm**	**Subsoil Consistency**
0 - 4	Very Soft	0 - 8	Very Loose
4 - 8	Soft	8 - 18	Loose
8 - 15	Firm	18 - 54	Medium Dense
15 - 24	Stiff	54 - 90	Dense
24 - 54	Very Stiff	>90	Very Dense
> 54	Hard		

5.4 Material Sampling

A total of 51 representative subsoil samples, subdivided into 30 bulk samples and 21 indicator samples, were collected from various inspection pits during the coarse of the field investigation and returned to Thekweni Soils Laboratory in Durban for analysis. All 51 samples were subjected to full grading analysis for classification purposes. Further analysis comprising density testing and California Bearing Ratio (CBR) testing was conducted on the 30 bulk samples.

The aim of the material sampling was to;

- identify potentially problematic soil horizons.

- determine the near surface materials as well as any potential borrow pit materials suitability for use in the proposed development.

The laboratory results summary tables as well as graphical representation of the grading analyses are included within Appendix C of this report. Furthermore the laboratory results are discussed in detail in Section 7 of this report.

5.5 Subsoil Percolation Testing

Several percolation tests were carried out across the study area within the various geological units that occur on site and the materials derived therefrom. The positions of the tests, designated PT 1 - PT9, are indicated on the site plan, Figure 1. The test were conducted to determine the in-situ percolation characteristics of the respective subsoils underlying the various portions of the site. As per the recommendations laid out in SABS - 400 (1987) the subsoils were soaked for the prescribed 4 hour time period prior to testing.

The subsoil percolation properties so derived are discussed in further detail under Section 8.4.

6. GEOLOGY AND SOILS

6.1 General

Based on the regional geological map, previous site drive over and exposure and inspection pit logging it is evident that the site is underlain by inter-stratified sandstone, siltstone and shale of the Vryheid Formation and the colluvial and residual material derived therefrom. The sedimentary bedrock has been regionally intruded by Jurassic aged dolerite bodies of the Karoo Supergroup which present as dykes or sills. Furthermore Recent Aeolian dune sand which caps underlying Berea and Vryheid Formation material was encountered across large portions of the site. The inferred geology of the area is shown on the geological plan, Figure 1 of this report and is discussed further below.

6.2 Vryheid Formation

This rock formation generally exists as a sequence of micaceous fine grained sandstones, very thinly bedded siltstones and shales. Vryheid Formation sandstone was by enlarge the most commonly intersected unit.

Bedding orientation of the stratified sequence was found to be highly variable, this being attributed to disturbances associated with the emplacement of the intrusive dolerite across the site. Where bedrock was encountered and not too deeply weathered, the location of which are indicated on the site plan, Figure 1, measured orientations of bedding were taken.

6.2.1 *Sandstone*

Across the site the strata of the Vryheid Formation was generally deeply weathered and thus the sandstone bedrock was only occasionally intersected within the excavated inspection pits. Where encountered the bedrock was located at depths ranging between 1.8 - 2.2m below existing surface level and described as light orange and yellow, completely weathered, very soft rock recovered as gravelly, clayey sand to silty, clayey sand. As such bedding planes were often not observed. Where exposed at or near the surface, typically out cropping on the mid to lower portions of steep valley sides the weathered sandstone was described as light brown to yellow brown, highly weathered, medium jointed, soft to medium hard rock. The weathered bedrock was often directly overlain by a thin mantle of dark brown, clayey sandy colluvium.

Residual material overlying the deeply weathered sandstone was generally encountered at depths ranging between 1.0 - 2.0m and extended to depths in excess of 2.5 - 3.0m. The material comprised of slightly moist to moist, orange and brown occasionally reddish brown, mottled grey, intact, firm to stiff, sandy clay to clayey sand.

The residual sandstone was generally overlain by a variably thick mantle of loose to medium dense, brown to light brown, fine to medium grained colluvial sand or Recent Aeolian dune sand containing organic material towards the surface.

6.2.2 *Shale*

Shale bedrock was generally intersected as dark grey weathered brown, orange brown and reddish brown, highly to completely weathered, laminated to thinly bedded, closely jointed, very soft to soft rock. The shale was typically encountered towards the north western and central portions of the site in close proximity to the central dolerite intrusive body. Where exposed at the surface or within inspection pits with clearly present bedding planes, the strata was locally inclined up to 30 - 40° in some places due to the dolerite intrusion.

Shale bedrock, typically near to the main western tributary, was overlain by grey mottled yellow and orange brown, sandy to silty clay. The residual material was encountered to depths ranging between 1.10m to in excess of 2.7m in more deeply weathered areas. Where exposed on the more elevated ridge tops the shale was encountered at an average depth of 0.5m and overlain by a thin horizon of grey and brown clayey residual and in turn colluvial material. Colluvial material was generally encountered as dark brown, fissured, gravelly, clayey sand to sandy clay and encountered as a thin mantle on the ridge tops and valley bottoms becoming slightly thicker on the mid portion of the slopes.

6.2.3 *Siltstone*

Weathered siltstone was not commonly encountered across the investigated area. However where exposed comprised grey and orange to reddish brown, highly to completely weathered, closely jointed, very soft rock often recovered as silty clay. The material was generally excavated to depths in excess of 3.0m below the surface. Where exposed at the surface near IP 81 towards the north western corner of the site the highly weathered bedrock dipped gently out of the steep slope in a northerly direction.

The deeply weathered siltstone is overlain by thick deposits of residual material ranging from 1.0 - 2.5m thick. The residuum was characterised as light orange and grey, intact, soft to stiff, silty clay to clay silt. The residual siltstone is generally overlain by a relatively thin mantle of dark brown, sandy silty clay.

6.3 Karoo Dolerite

The intrusive dolerite is limited to the western and central portions of the site. The main occurrence of Karoo dolerite on site was exposed within the existing centrally located borrow pit area. Logging of Exp 13 conducted within the quarry area described the bedrock as slightly weathered to highly weathered in places, closely jointed dolerite ranging from soft to very hard rock. More deeply weathered dolerite was encountered nearby within IP 70, 73 and 82 and was described as dark grey, weathered orange brown and brown, highly to completely weathered, closely jointed, very soft to soft rock. The weathered bedrock was generally excavated to depth in excess of 2.5m however refusal was met at shallower depth on less weathered rounded dolerite corestone which were occasionally encountered within the weathered bedrock.

Residual dolerite overlying the weathered bedrock was encountered as red to orange brown, intact, firm to stiff, silty clay. The material ranged from in the order of 1.0m thick to greater than 2.5m thick in more deeply weathered areas. Reddish brown to dark brown, fissured, clayey colluvium ranging from 0.4 - 1.0m was generally encountered overlying the residual dolerite.

6.4 Berea Formation

The sediment of the Berea Formation was generally encountered along the coast line and elevated central portions of the site. The material was rarely encountered at the surface but rather buried below a mantle of wind blown sediment.

Where intersected the material was typically characterised as red brown, orange and orange brown, loose to medium dense, fine to medium grained slightly clayey to clayey sand. The Berea Formation, where not exposed in man-made exposures at the surface, was usually intersected at depths ranging between 0.8 - 2.5m below the surface. The clayey sand has been known to extend to depths in excess of 30m below existing ground level and likely overlies the weathered Vryheid Formation.

6.5 Recent Aeolian Dune Sand

The Recent aeolian dune sand encountered at the surface to varying depths across the site was identified by its characteristic light orange to yellow brown and brown colouration. The sandy material typically overlies the Berea Formation and residual Vryheid Formation sediment. Excavation within the unconsolidated material was extremely easy. However excavation rarely extended to depths in excess of 2.0m due to significant sidewall collapse restricting further removal. Where underlain at relatively shallow depths by more clayey material greater depth could be achieved.

6.6 Alluvium

On the lower slopes adjacent to valley bottoms alluvial material was occasionally encountered and often comprised of moist to wet, grey and brown, loose, silty to clayey sand. Within saturated valley bottoms dark grey and dark brown, silty clayey material was also encountered.

7. LABORATORY RESULTS

Given the number of samples taken during the coarse of the detailed investigation the results have been sorted and summarised according to geological unit and included in Tables 2.1 - 2.8 below for ease of reference.

Table 2.1 : Summary of Material Analyses - Vryheid Formation Samples

IP	Material	Description	LL %	PI	LS %	% Clay	Revised US Classification	
							Group	Subgrade Rating
40	Colluvium	Sandstone	20.4	N.P.	N.P.	2.4	A-3	Good
45		Sandstone	17.6	N.P.	N.P.	4.9	A-3	Good
71		Shale	38.5	9.9	4.7	16.8	A-2-4	Good
30	Residual	Sandstone	23.6	8.5	3.3	25.1	A-2-4	Good
36		Sandstone	36.2	15.6	6.7	51.2	A-6	Poor
42		Sandstone	66	33.5	14	56.5	A-7-5	V. Poor
45		Sandstone	41.4	18.8	10	38.7	A-7-6	V. Poor
52		Sandstone	29.1	11.6	6.7	31.5	A-6	Poor
53		Siltstone	69.6	27.2	12.7	43.0	A-7-5	V. Poor
54		Sandstone	25.3	11	4.7	26.4	A-2-6	Good
62		Sandstone	40.5	17.7	9.3	33.1	A-7-6	V. Poor
65		Sandstone	35.1	14.7	7.3	37.7	A-6	Poor
72		Shale	40.1	11.2	8.7	40.7	A-7-6	V. Poor
78		Shale	51.6	24	9.3	53.8	A-7-6	V. Poor
79		Siltstone	42.9	20.1	8.7	51.4	A-7-6	V. Poor
81		Siltstone	45.3	16.5	8.7	29.4	A-7-6	V. Poor
83		Shale	76.2	43.4	14.7	56	A-7-5	V. Poor
90		Sandstone	64.2	27.4	10.7	50.9	A-7-5	V. Poor
38	Weathered Bedrock	Sandstone	30.1	10.6	4	15.5	A-6	Poor
42		Sandstone	26.9	9	1.3	4.6	A-2-4	Good
60		Sandstone	28	17.1	8.7	28.7	A-6	Poor
69		Siltstone	63.7	23.2	8.7	27.4	A-7-5	V. Poor
71		Shale	47.7	14.5	5.3	20.5	A-7-5	V. Poor
72		Shale	31.7	10.8	4.7	18.7	A-6	Poor
78		Shale	28	9.7	4.7	9.4	A-2-4	Good
79		Siltstone	29.3	6.7	3.3	17.6	A-4	Fair
85		CW Sandstone	37.2	15.2	7.3	17.5	A-2-6	Good

Table 2.2 : Summary of Mod AASHTO Density and CBR Test results - Vryheid Formation Samples

IP	Material	Description	Mod AASHTO Density (kg/m^3)	OMC (%)	CBR Results			TRH 14
					90 %	98 %	Swell %	
40	Colluvium	Sandstone	1733	13.7	10	18	0.1	G8
45		Res Sandstone	1756	13.7	2	3.1	5.74	>G10
36	Residual	Sandstone	1800	13.6	0.9	1.6	7.9	>G10
52		Sandstone	1726	15.5	2	6	1.26	G10
65		Sandstone	1768	16.9	2	10	1.6	>G10
38	Weathered Bedrock	Sandstone	1869	12.7	3.7	5.3	3.2	>G10
42		Sandstone	1929	8.5	9	14	1.5	>G10
60		Sandstone	1846	12.5	1.2	1.8	5.81	>G10
69		Siltstone	1592	16.6	2.1	2.7	6.48	>G10
71		Shale	1650	19	4	5.6	2.56	>G10
72		Shale	1938	10.6	3	5	3.94	>G10
78		Shale	1996	10	1.6	2.3	4.72	>G10
79		Siltstone	1823	12.9	1.6	1.9	4.10	>G10
85		CW Sandstone	1970	9.8	12.5	36	0.21	G8

Table 2.3 : Summary of Material Analyses - Dolerite Samples

IP	Material	Description	LL %	PI	LS %	% Clay	Revised US Classification	
							Group	Subgrade Rating
74	Colluvium	Dolerite	44.2	24.5	11.3	55.8	A-7-6	V. Poor
75			38.3	13.9	8.7	50.4	A-6	Poor
88			27.1	10.9	5.3	33.0	A-6	Poor
74	Residual	Dolerite	61	18.5	10.7	40.1	A-7-5	V. Poor
75			57.3	23.3	12.7	53.4	A-7-5	V. Poor
88			49.2	17.5	10.7	56.1	A-7-5	V. Poor
70	Weathered	Dolerite	29.6	9.9	2.7	1.7	A-2-4	Good
73			31.9	11.4	2.7	10.5	A-2-6	Good
82			34.1	10.6	6.0	13.7	A-6	Poor
S			29.6	8.3	4.0	0.9	A-2-4	Good

Table 2.4 : Summary of Mod AASHTO Density and CBR Test results - Dolerite Samples

IP	Material	Description	Mod AASHTO Density (kg/m^3)	OMC (%)	CBR Results			TRH 14
					90 %	98 %	Swell %	
75	Colluvium	Dolerite	1665	19.9	2	11	2.36	>G10
74	Residual	Dolerite	1378	28.5	0.1	1.8	8.8	>G10
70	Weathered Bedrock	Dolerite	1959	12.5	6	16	0.46	G9
73			1884	12.3	10.5	21	1.1	G8
82			1893	13.1	10	29	0.0	G7
S			2178	6.3	18	26.5	11.5	>G10

Table 2.5 : Summary of Material Analyses - Berea Formation Samples

IP	Material Description	LL %	PI	LS %	% Clay	Revised US Classification	
						Group	Subgrade Rating
3	Clayey Sand	21.6	3.7	2	22.5	A-2-4	Good
7	Sl Clayey Sand	19.3	N.P	N.P	8.3	A-2-4	Good
9	Clayey Sand	23.9	11.2	4	25.5	A-2-6	Good
14	Clayey Sand	24.9	10.2	4.7	26.7	A-2-6	Good
16	Sl Clayey Sand	20.4	N.P	N.P	5.1	A-3	Fair
19	Clayey Sand	20.2	8.3	3.3	23.8	A-2-4	Good
26	Clayey Sand	23.3	7	3.3	25.4	A-2-4	Good

Table 2.6 : Summary of Mod AASHTO Density and CBR Test results - Berea Formation Samples

IP	Material Description	Mod AASHTO Density (kg/m^3)	OMC (%)	CBR Results			TRH 14
				90 %	98 %	Swell %	
3	Clayey Sand	1894	11	4.8	14	1.53	>G10
14	Clayey Sand	1866	12	6	12	0.69	G9
16	Sl Clayey Sand	1650	12.7	2	7.5	0.0	>G10

Table 2.7 : Summary of Material Analyses - Recent Aeolian Dune Samples

IP	Material Description	LL %	PI	LS %	% Clay	Revised US Classification	
						Group	Subgrade Rating
2	Clayey Sand	19.6	N.P	N.P	8.5	A-2-4	Good
6	Fine - med Sand	19.5	N.P	N.P	4.1	A-3	Fair
18	Fine - med Sand	18.8	N.P	N.P	3.7	A-3	Fair
23	Fine - med Sand	19.9	N.P	N.P	3.7	A-3	Fair
24	Fine - Coarse Sand	15.4	N.P	N.P	5.8	A-3	Fair
29	Fine - med Sand	17.3	N.P	N.P	5.0	A-3	Fair
51	Fine - med Sand	19.3	N.P	N.P	2.2	A-2-4	Good

Table 2.8 : Summary of Mod AASHTO Density and CBR Test results - Recent Aeolian Dune Samples

IP	Material Description	Mod AASHTO Density (kg/m^3)	OMC (%)	CBR Results			TRH 14
				90 %	98 %	Swell %	
2	Clayey Sand	1680	16	1	9	0.0	>G10
6	Fine - med Sand	1698	6.7	1.6	7.7	0.45	>G10
23	Fine - med Sand	1646	9.8	4.3	13	0.0	G10
29	Fine - med Sand	1759	11.1	3	12	0.0	G10
51	Fine - med Sand	1665	14.6	19	22	0.1	G7

The above results are discussed in further detail with regard to problem soils and material suitability in Sections 8.2 and 8.5 below respectively.

8. GEOTECHNICAL ASSESSMENT

With regards to the consolidated and unconsolidated materials underlying various portions of the site, there are a number of geotechnical constraints for the respective materials that need to be taken into consideration during the planning and implementation of the proposed development. These include but are not limited to the following;

8.1 Slope Stability

Our initial preliminary investigation drew attention to potential instability within various units encountered on site by the identification of likely previous slope failures. Further investigation during the detailed investigation identified areas where parameters are favourable within the respective units for potential instability. These are discussed further below with respect to the individual units on site.

8.1.1 *Recent Aeolian Dune Sand and underlying Berea Formation*

In general, the topography across the eastern coastal portion of the site was characterised by moderately to steeply sloping valley sides ranging between 10 - 17°. However, some slopes, highlighted on the site plan, are significantly steeper and range from 18 - 28°. These areas are typically marked by concave topography and as such suggest areas where previous slope instability has taken place. Given the steepness of the slope, significant depth of unconsolidated material, likely perched water table aided by concave topography of the slope and natural angle of repose of the unconsolidated sandy material typically in the order of 28 - 30° these areas are considered highly unstable and should be strictly avoided during development.

Moderately steep slopes (10 - 17°) can be developed provided all due caution and good engineering practices are exercised during construction as any injudicious cutting and/or loading or mass removal of binding vegetation within these areas, although only moderately sloping, can increase instability and induce slope failure.

8.1.2 *Vryheid Formation*

Areas underlain by Vryheid Formation, for the most part where found to be deeply weathered. As such, well preserved bedding planes along which orientation reading could be recorded were seldom encountered.

In general the sedimentary bedrock dips in a south easterly direction at and average inclination of 12°. However, where bedding readings were retrieved the values were found to vary greatly from this across the site. This can be attributed to the volatile intrusion of the dolerite which has led to the disruption of the bedding of the Vryheid Formation host rock.

Within IP's 69, 81 and Exp 8, 22 and 26, shale, siltstone and sandstone bedding was found to dip at angles ranging between 5° - 40° and ranging in direction from northerly, south easterly and westerly. In some instances the bedding was found to dip unfavourably out of the moderate to steep slopes. These areas have been highlighted on the site plan and should be considered as potentially unstable. Furthermore, the intrusive dolerite may likely have caused fractures within the bedrock generally promoting the development of clay lenses along open fracture planes which further increases the likelihood of slope failure.

With respect to the above for planning purposes all easterly facing slopes across the eastern and western portions of the site underlain by Vryheid Formation bedrock with natural slope angles greater than 1:3 (18°) should be considered as potentially unstable. Similar slopes across the central, dolerite intruded, area should also be considered as potentially unstable. More detailed site specific slope stability analyses will be required once more detailed development plans are provided.

8.1.3 *Karoo Dolerite*

As mentioned in the initial Phase 1 report, evidence of slope instability was noted within deeply weathered residual dolerite material. As such steep, deeply weathered slopes underlain by thick in-situ residual dolerite should be considered as potentially unstable. Areas underlain or in close proximity to dolerite bedrock, as mentioned above, will require site specific stability assessments once detailed development plans are available.

8.1.4 *General*

It must be noted that at the time of the drive over survey large portions of the development area were covered by mature uncultivated sugar cane and/or thick natural vegetation. As such further evidence of unstable slopes may have been obscured from view and may therefore only become evident during more site specific geotechnical investigations or during the development phase of the project.

However, in general slopes greater than 18° underlain by deeply weathered Vryheid Formation, Karoo Dolerite, or thick deposits of loose Berea Formation and capping Recent Aeolian dune sand should be considered as potentially moderately to highly unstable and should not be considered for development.

8.2 Problem Soils

8.2.1 *Collapsible Soils*

DCP's conducted across the southern, eastern, and central areas capped by Recent Aeolian Dune sand and underlain by sands and clayey sand of the Berea Formation indicate the materials are in general very loose to loose to depths ranging between approximately 3.0 - 6.0m below existing ground level.

As such the unconsolidated Recent aeolian dune sand and underlying Berea Formation sands are likely to have a moderately high to high collapse potential in the sense that when subjected to a critical increase in moisture content under load, they undergo a densification and subsequent settlement. Although little to no DCP testing was conducted within the base of drainage valleys alluvial material likely contained therein is considered to have a similar collapse potential.

8.2.2 *Active Soils*

From laboratory results it is evident that colluvial silty clays in areas underlain by dolerite, residual silty clays, clayey silts and sandy clays of all bedrock types as well as completely weathered sandstone, siltstone and shale bedrock of the Vryheid Formation are in general considered to be moderately to highly active in the sense that the materials will likely undergo volume change upon fluctuations in the materials in-situ moisture content.

In particular residual and completely weathered sandstone and siltstone material sampled from IP’s 60, 62, 69, 81 and 90 are classified as moderately active with residual shale and sandstone material sampled from IP’s 42, 53 and 83 being classifying as highly active.

Where these materials are encountered they should be removed to spoil as much as practically possible. Any structures developed therein must take this into account and be designed so as to accommodate potential heave.

Clayey sand to sandy clay of the Berea Formation is considered to be slightly active.

8.2.3 *Erosive Soils*

The very loose to loose consistency, low cohesion between individual particles and fine to medium grained particle size of the Recent aeolian Dune sand, sandy Berea Formation and sandy colluvium results in these material being highly prone to erosion via wind and flowing storm water run-off, especially given the sloping nature of the site. Erosion gullies extending across cane tracks were often encountered during the field investigation in areas underlain by the above mentioned sandy material where concentrated flow of stormwater was present.

Furthermore, the likelihood of such erosion will increase dramatically once the site is cleared of covering vegetation for the purpose of the development, which has a binding action on the underlying soils.

As such, strict measures should be in place both during and after construction to control storm water run-off across the site. Post construction all batters and unpaved areas should be vegetated in order to keep the erosion of upper soils to a minimum.

Due to the likely moderately high clay content within the more clayey colluvial and residual materials, these soils are not as susceptible to erosion, however , if subjected to concentrated surface flow, erosion is possible.

8.3 Subsoil Seepage

Subsoil seepage was intersected within a number of inspection pits excavated across the central to western potions of the investigated area underlain by Vryheid Formation bedrock at depths ranging between 1.3 - 2.8m below existing ground surface.

Pits in which seepage was encountered were generally located within elevated areas towards valley heads or on the lower portions of the side slopes in close proximity to where surface water was often observed within the valley bottoms.

Seepage generally coincided with the contact of the upper more permeable colluvium and underlying residual material or at the interface of overlying soil and weathered bedrock.

Although no ground water seepage was encountered in the sandy materials underlying the eastern portion of the site, seepage should not be excluded in this area as it may become perched on or within the more clayey sands of the Berea Formation.

Although subsoil seepage may be problematic in development across the various portions of the site, the presence of which does not preclude the development of the area unless the area falls within the “wetland” area as defined by the Environmental Consultant.

Within developable areas, where subsoil seepage is encountered, the seepage can be curtailed or managed through the suitable placement of adequate subsoil drains. Herringbone subsoil drainage and rock fill blankets will be required extensively where potential fill materials are to be constructed across channels or towards valley heads.

8.4 Percolation Characteristics

Selective percolation testing was carried out across the proposed development area during the coarse of the detailed investigation. The positions of which are indicated on the site plan, Figure 1. Where possible percolation tests were situated within the various subsoils encountered across the site. The results of the percolation testing are tabulated below.

Table 3 : Subsoil Percolation Tests Summary

PT №	Material Type	Percolation Rate (Average time for 25mm drop in test water level)	Rate of Application of effluent to subsoil infiltration area (Litres/m^2/day) *
1	Dolerite Colluvium	12 min	79 - 65
2	Residual Dolerite	28 min	39 - 33
3	Residual Sandstone	36 min	Not Permitted
4	Recent Aeolian Dune Sand	12 min	79 - 65
5	Residual Siltstone	38 min	Not Permitted
6	Sandstone Colluvium	12 min	79 - 65
7	Berea Formation	14 min	79 - 65
8	Recent Aeolian Dune Sand	10 min	99 - 80

*** - Based on Table 4 of the National Building Regulations Act - 1977**

From the above tabulated results, percolation testing in the various units revealed the following;

- The Recent Aeolian Dune sand, sandy colluvium overlying residual material and Berea Formation clayey sand are all suitable for the use of soak pits and french drains should they be required therein.

- The clayey residual sandstone and siltstone is deemed not suitable for the use of soak pits /french drains. Additionally, although the residual dolerite passed and is considered marginally suitable, the placement of soak pits and french drain system therein is highly not recommended. Similarly areas with high water tables, as is anticipated at the base of valleys are also not suitable for subsoil waste water/stormwater disposal.

In light of the above, in general it is considered feasible to incorporate subsoil percolation for waste water and stormwater disposal across the eastern and southern portions of the site. However, cut-to-fill platforms must be accounted for as removal of upper sandy material may result in less permeable material at platform level.

Conversely across the majority of central and western portions of the site, subsoil percolation is likely not permitted in clayey material likely exposed at the surface once platforms have been cut and filled. Therefore, in such areas underlain by residual and clayey colluvial material or shallow water table conditions, we recommend that in the planning phase of the development, provisions are made for stormwater systems and a waterborne sewage option or alternatively on site package treatment plants.

8.5 Construction Materials

From laboratory results it is evident that the materials underlying the deeply weathered site are in general not good quality for use for construction purposes.

8.5.1 *Vryheid Formation*

In general the clayey sandy colluvial material overlying Vryheid Formation sediment was deemed to classify as G8 -G10 type material. As such the material is considered suitable for use as lower selected layers and subgrade material.

The typical residual silty clays, clayey silts and sandy clays derived from the in-situ weathering of Vryheid Formation sandstone, siltstone and shale range from A-6 to A-7-6 materials and classify as >G10 gravel soil due to significant CBR swell values and low CBR values. Therefore these clayey soils are deemed not suitable for use as subgrade material or bulk fill.

The typically deeply weathered, shale, siltstone and sandstone bedrock encountered within the majority of inspection does not meet the minimum requirements of a G10 type material due to significant CBR swell values. As such the material should not be used for construction purposes. Less weathered varieties of sandstone material, as encountered and sampled from IP 85 are present locally across the site. The material classifies as a G8 material and therefore considered suitable for use in lower selected layers and as subgrade material. However, this material may not be readily available in large quantities at shallow to moderate depths across the site.

8.5.2 *Karoo Dolerite*

The clayey colluvium encountered in areas underlain by Karoo dolerite classifies as A-6 to A-7-6 material and > G10 material. Therefore the material is deemed unsuitable for use as subgrade or bulk fill material. Given the high clay percentage and linear shrinkage values of this material it should be removed to spoil where encountered as much as practically possible.

The residual dolerite consistently classifies as an A-7-5 material and does not meet the minimum requirements of a G10 type material. Therefore the material is not suitable for use as subgrade or bulk fill and should be removed to spoil as much as practically possible.

Weathered dolerite material encountered on site classifies as A-2-4 to A-6 material and G7 to >G10 type material depending on the degree of weathering of the material. As such weathered dolerite can selectively be used for construction purposes within upper and lower selected layers as well as subgrade and bulk fill material.

8.5.3 *Berea Formation*

Slightly clayey to clayey sand of the Berea Formation classified as A-2-4 to A-3 and ranges between G9 - >G10 material depending on the relative clay and sand content. As such the material ranges from being suitable for use in lower layer works, subgrade and bulk fill material to not suitable for use as a construction material.

8.5.4 *Recent Aeolian Dune Sand*

The sandy material typically classifies as an A-3 type material and as a G10 - >G10 type material. As such the material is considered suitable for use as subgrade material or as bulk fill. Selective samples classify as better than a G10 material and therefore can be used as construction material in road and platform layers work where applicable.

8.6 <u>NHBRC Classifications</u>

Based on analysis of the excavated inspection pits, DCP tests and laboratory results, various portions of the site have been generally classified in terms of the NHBRC Classifications as the following;

Areas underlain by Recent Aeolian Dune Sand - Collapsable Soils (C2 - C3)

Areas underlain by potentially active residual Vryheid Fm - Heaving Soils (H2 - H3)

The extent of these generalised area have been marked out on the site plan Figure 1. However, it should be noted that cutting during earthworks may expose bedrock (R) or heaving soils underlying collapsible soils at platform level. As such during development of the site it will be necessary to more accurately determine site specific NHBRC Classifications once individual platforms have been created.

9. RECOMMENDATIONS FOR PLANNING AND DEVELOPMENT

9.1 <u>Earthworks</u>

At this planning stage, no details with regard to earthworks are available. However, given the undulating nature of the site area, significant earthworks are envisaged. In this regard the following general cutting and filling recommendations and excavatability conditions should be taken into account for planning purposes.

9.1.1 *Excavatability*

Excavatability within the very loose to loose Recent aeolian dune sands and clayey sands of the Berea Formation are expected to be ‘soft’ to considerable depth.

In general the highly to completely weathered Vryheid Formation sandstone, siltstone and shale is also considered to classify as ‘soft’ excavation according to SABS 1200D standards to considerable depth. Furthermore the generally closely jointed nature of the parent material will aid in its removal where intersected during platform creation. However, cognisance must be paid to the fact that local variations in hardness may occur due to the degree of weathering and mineral content or as a result of ‘baked contacts’ where in close proximity to the dolerite intrusion.

Highly to completely weathered dolerite and overlying residual and colluvial material is considered as ‘soft’ excavation according to SABS 1200D standards. Where slightly to medium weathered bedrock is encountered as exposed towards the existing borrow pit area excavation will become more difficult and may require blasting to remove. In the same regard large corestones may be encountered within the completely weathered to residual dolerite profile and also require more effort to remove.

9.1.2 *Cuts*

Permanent cut slopes in all unconsolidated colluvial, residual, alluvial and wind blown sediment should be restricted to a maximum slope batter of 1:2 (26°). Temporary slopes in the clayey materials can be steepened to 1:1,75 (30°) at the discretion of a responsible Geotechnical Engineer. Cuts in firmly bedded, favourably dipping (into the slope) sandstone, siltstone or shale, or dolerite bedrock, may be laid back to a batter of 1:1,5 (33°). Cut slopes should not exceed a maximum height of 3.0m without being assessed by a responsible Engineer or suitably retained if necessary.

Where the above mentioned batters cannot be accommodated the slope should be supported by a suitably designed retaining structure.

All cut embankments, especially those within sandy material prone to erosion must be protected against surface erosion by planting of vegetation after construction.

Given the slope stability concerns highlighted in Section 8.1 above cutting in these areas or areas with similar characteristics will need to be undertaken with great caution and with on going supervision from a geotechnical professional. Unstable areas identified during this investigation are of the greatest concern and should not be developed. Should areas with similar characteristics be developed continual assessment of cut slopes is strongly advised to determine at an early stage any additional stability requirements that may be necessary.

9.1.3 *Fills*

For preliminary design purposes all fill embankment batters should be restricted to 1:2 (26°) and a maximum height of 3.0m if not retained.

Given the moderately to steeply sloping nature of the site ensuring stable founding of the likely fill embankments will be crucial to the success of any future development on site.

Within coastal to central areas underlain by sandy material it is considered necessary that proposed fills on moderately steep slopes, greater than 1:6 (10°) are benched into suitably dense material. For deeply weathered areas underlain by Vryheid Formation bedrock or Karoo dolerite and the clayey materials derived therefrom, guideline for the founding requirements for the general subsoil conditions are presented below;

- In general, where fill embankments are intended on moderately steep slopes where clayey soils are < 3.0m thick, excavation to bedrock and the construction of a rockfill toe key must be expected. This would involve the excavation of all clays overlying the competent rock along the toe for a predetermined width and subsequent backfilling of the slot to a designated level. Using a coarse pioneer rockfill, preferably of imported, durable rock.

- Where filling is required on slopes characterised by clayey soils exceeding 3m thickness, excavation to rock for construction of a rockfill toe key may prove impractical and economically unfeasible. In this case a thin basal rockfill toe would be required in conjunction with geogrids placed at designated spacing within the new fill.

In light of the above at the detailed design stage stability analysis should be carried out for each proposed fill embankment to determine the site specific founding requirements thereof and the required design slope batters.

Fills should be designed and constructed as well compacted engineered fills with the intention of minimising internal settlements to the 1 - 2% of the fill thickness that is expected for well compacted fills. In this regard granular material of G10 or better quality should be favoured and positioned in areas where structures are proposed. The use of more clayey materials (>G10 quality) should ideally be avoided or at least minimized by restricting its use to areas which are not to be developed or for landscaping. In this regard the above may prove difficult given the generally very poor nature of the deeply weathered Vryhied Formation bedrock and overlying material which through laboratory testing has been identified are generally unsuitable. As such careful planning of available materials and their suitability will be required and may necessitate the import of suitable off-site material.

A clear record should be kept of where different material types are placed to aid in settlement determinations and structural positioning. Furthermore, we recommend that upon construction of the platforms, the prick of the cut-to-fill be surveyed if the platforms are to stand for some time before the construction of the proposed structures. Knowing the exact location of this transition across platforms will prove invaluable when designing the structural foundations.

Working benches should be cut into the sideslopes and seated in competent material removing any unsuitable problem materials where necessary. Following which the fill material should be placed and spread in layers not exceeding a loose thickness of 300mm. While compaction requirements will vary between materials, a general compaction of 93% and 95% of the materials maximum Mod AASHTO density for more clayey and sandy materials respectively should be achieved prior to the placement of the next layer. The maximum particle size within the fill should not exceed two thirds of the layer thickness. Where piling will be the most likely means of founding boulders should not be incorporated into the fill.

More clayey materials (residual and colluvial soils) where included in the fill embankments should be limited to layers of 200mm loose thickness and where possible sandwiched between more granular material in the lower layers of the fill. As mentioned the clayey layers will exhibit increased consolidation and heave potential in comparison to the less clayey materials, hence should be confined to non-structural portions of the fill. With respect to material workability, moisture control will be critical in achieving compaction control of the more clayey and silty materials. As such both padded and smooth drum rollers may be required for satisfactory compaction of the variable materials.

Once complete the fill embankment should be vegetated to minimise surface erosion.

9.2 Site Drainage

Taking into account the percolation assessment of the subsoils on site, it is apparent that storm water and effluent disposal via subsoil percolation is feasible across the eastern coastal and central areas underlain or capped by sandy material. However, where underlain by more clayey colluvial and residual subsoils or completely weathered bedrock, storm water and effluent disposal via soak pits and french drain system is only marginally feasible to unfeasible. Furthermore, cognisance must be paid to areas where less permeable clays are underlain by a mantle of sandy material as it is probable that during earthworks this permeable material may be removed through cutting in its entirety and platforms are seated in less permeable material.

As such, across portions of the site, provision must be made for control of storm water whereby run-off is piped or carried in surface drains to discharge into the stormwater system, comprising suitably designed attenuation ponds which ultimately discharge into the Tete River. Effluent must also be piped in a suitably designed sewage system to discharge off site.

After construction of the respective sites, the area should be graded to facilitate effective and efficient run-off and prevent ponding of stormwater on surface adjacent to any structures.

9.3 Founding

Several factors will dictate the founding solutions adopted for new structures across the investigated area. These include the underlying parent rock type, thickness of overlying colluvial and residual material derived therefrom and final platform layout.

In terms of the above, at this stage the following general founding conditions can be expected;

- In less deeply weathered areas, for portions of the platform cut into competent rock, shallow founding of structures on strip footings will generally be feasible. A provisional conservative bearing capacity of 250 kPa can be assumed.

- For the transition of cut to fill (extending from competent cut rock into sections of shallow fill) founding into bedrock will likely require the use of ground beams spanning isolated pads.

- Where deep colluvial and residual clayey soils occur overlying weathered bedrock, as is likely across the majority of the site, or for sections of fill where bedrock exceeds the economic and practical limitations of column base foundations (generally 2.5m), end bearing piled foundations will be required to found into competent bedrock at some depth. Similarly for areas underlain by loose aeolian dune sand and/or Berea Formation sands and clayey sands to considerable depths, it is recommended structures be supported on reinforced ground beams spanning friction piles taken to the required depth.

- For large structures spanning large cut-to-fill platforms a combination of shallow and deep founding solutions may be required.

- Where relatively small structures are positioned entirely in fill, the structure must be able to tolerate relatively large total settlements (internal settlement of fill) as well as a degree of differential settlement (due to variable thickness of fill across the structure) for shallow founding in the fill to be feasible. At this early stage we would advocate a bearing pressure not exceeding 150 kPa in the fill. Alternatively, for compact structures, where deep clayey soils or loose sandy soils occur, structures may be supported on suitably designed reinforced concrete raft foundations.

- With the exception of the raft foundations, given the potential activeness of the deeply weathered clayey soils and completely weathered bedrock as well as potentially collapsible sandy soils present, the ground floor slabs of all structures supported on piles, strips or column bases should be isolated from all walls, columns and foundations and incorporate suitable articulation and joints to accommodate any heave or potential differential settlement as mentioned above that may occur.

- Notwithstanding the above, we consider it essential that detailed geotechnical investigations are carried out for the individual developments proposed across the site once the details of these developments are made available.

9.4 Retaining Structures

As mentioned above, it is likely that significant cutting and filling will be required across the site during the earthworks phase of the development. Where the above mentioned cut and fill batters cannot be accommodated due to space restrictions, cut and fill slopes must be supported by a suitably designed retaining walls. Where inclined bedrock is intersected, especially where unfavourably dipping out of the slope, rock anchors and gunite may be required at the discretion of the Geotechnical Engineer. The design of any retaining walls or rock stabilising measures should be carried out by an experienced Structural Engineer familiar with the site specific subsoil and ground water conditions. The lateral support should incorporate adequate drainage behind, above and through the wall and be suitably damp proofed, especially within the sandy materials underlying the eastern portion of the site.

The following conservative soil shear strength parameters are recommended for use in retaining wall design;

- Angle of internal friction (Φ) - 28°
- Soil cohesion (c) - 0 kPa

Site specific assessment and shear box testing will be required once a development plan is provided.

10. CONCLUSION

The proposed development of the Tinley Manor - South Bank area is considered feasible as no catastrophic geological flaws exist that would exclude the entire area from development, although some areas should be avoided in terms of slope stability and problem soils.

Not withstanding the above the development of the area should be considered as challenging due to the geological constraints associated with the prevailing subsoil and ground water conditions present on site.

As such for planning and construction of the proposed development, the recommendations given above should be strictly adhered to. These amount to no more than sound building practices appropriate for the geotechnical constraints associated with the on site subsoils conditions. Site specific geotechnical investigation will be required at a later date and should include provisions for regular supervision by a geological engineering professional during development.

M.J. HADLOW Pr.Sci.Nat.

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/aj/kc

DRENNAN, MAUD AND PARTNERS
68 Peter Mokaba Ridge, Tollgate,
DURBAN, 4001

APPENDIX A

SOIL PROFILES

(IP 1 - IP 98 & EXP 1 - EXP 25)

APPENDIX B

DYNAMIC CONE PENETROMETER TEST RESULTS (DCP 1 - DCP 92)

APPENDIX C

LABORATORY TEST RESULTS

FIGURE 1

SITE PLAN