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30th September 2013

PGA
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Attention: Mr P. Govender

Dear Sir,

CENTRELINE SURVEY AND GEOTECHNICAL INVESTIGATION: NTATSHANA ACCESS ROAD AND BRIDGE, MZUMBE MUNICIPALITY

1. INTRODUCTION AND TERMS OF REFERENCE

Drennan, Maud cc. was requested by PGA Consulting to submit a work proposal and cost estimate to carry out the required geotechnical investigation for the proposed Ntatshana access road and bridge across the Mtwalume River, Ward 7 Umzumbe Municipality. The work proposal detailing our investigation and cost thereof was sent on the 14th May 2013 and subsequent appointment and approval to proceed was received on the 16th May 2013.

The site was visited during the month of August 2013 during which time the geotechnical investigation field work was carried out.

Our subsequent findings from the field investigation, geotechnical assessment and recommendations for development are set out in this report.

2. AVAILABLE INFORMATION

Information supplied to Drennan, Maud cc included a letter of appointment, coordinates for the start and end of the road, a Google Earth image of the proposed route, route drawings and that the road will be part gravel, part concrete.

3. SITE DESCRIPTION

The site is located near the Kwamaqikizane area, approximately 7 km south of Jolivet as the crow flies, 17 km by road.

The proposed route commences at the termination of a dirt road at the following coordinates, 30° 21' 19.38" S 30° 20' 45.70"E (supplied by PGA) and proceeds down into the Mtwalume River valley.

The proposed route is located on the steeply north to north easterly sloping side of the Mtwalume river valley, becoming less steep towards the valley bottom. The route traverses areas strewn with tallus boulders and drainage channels/erosional gullies and crosses the Mtwalume river, with alluvial boulders located on the southern bank and bedrock exposed on the northern bank.

Vegetation consists mostly of grassland with numerous aloes growing in the project area, which would have to be relocated prior to starting earthworks.

4. FIELD WORK

The fieldwork was undertaken between the 5^{th} and 6^{th} of August 2013 and comprised the excavation of inspection pits, Dynamic Cone Penetrometer testing and the sampling of in-situ subsoil materials and borrow pit materials for laboratory testing.

The inspection pits and field test positions were determined using a hand held GPS accurate to approximately 3 m. The positions of the field testing are indicated on the site plan, Figure 1 of this report.

4.1 Inspection Pits

A total of eight inspection pits, designated IP 1 - 8, were mechanically excavated using an excavator to investigate the subsoil conditions. Pits were excavated within the route designated at approximately 500 m intervals. The pits were taken down to a minimum depth of 2.5 m below existing ground level or until refusal of the excavator was met at shallower depths on underlying bedrock.

The subsoils exposed in the various inspection pits were examined and logged by an Engineering Geologist familiar with the procedures of soil logging in terms of the standard method for Soil and Rock Logging in South Africa recommended by Jennings, J.E., Brink, A.B.A. and Williams, A.A.B. (1973).

The detailed inspection pit logs are presented in Appendix A of this report.

4.3 Exposures

A total of two exposures, designated EXP 1 and 2, were logged in the river and on the northern flank respectively. The exposure logs are presented in Appendix A of this report.

4.2 Dynamic Cone Penetrometer Tests

A total of ten Dynamic Cone Penetrometer tests (DCP 1-10) were conducted in the area indicated for the proposed road, the approximate positions of which are indicated on Figure 1.

The results of the probes are presented graphically within Figures 2 - 11 of this report.

For ease of evaluation Table 1 below is provided to aid in the interpretation of the DCP results with regards to the consistency of the con-cohesive and cohesive soils underlying the route. However it must be understood that this should merely be used as a guide as it is empirically derived and based on values specific to Drennan, Maud & Partners equipment.

Table 1: Subsoil Consistency Inferred from the DCP Test Results

Non Cohesive Soils		Cohesive Soils	
No of blows/300 mm Penetration	**Subsoil Consistency**	**No of blows/300 mm Penetration**	**Subsoil Consistency**
<8	Very Loose	<4	Very Soft
8-18	Loose	4-8	Soft
18-54	Medium Dense	8-15	Firm
54-90	Dense	15-24	Stiff
>90	Very Dense	24-54	Very Stiff
		>54	Hard

4.3 Material Sampling

Sampling of representative soil and rock horizons encountered within the excavated inspection pits was undertaken during the course of the field investigation. Two samples were collected from a borrow pit, located near the proposed road. All the disturbed samples were returned to Thekwini Soils Laboratory in Margate for analysis which comprised the following:

- Indicator Testing
- Mod AASHTO Density testing
- CBR testing

A rock sample for UCS testing was taken from the granite exposed in the river. The sample was returned to Thekwini Soils Laboratory in Durban for UCS testing.

The results of the materials testing are summarized in the laboratory test summary tables along with graphical representation of the grading analysis in Appendix B of this report.

5. GEOLOGY AND SOILS

5.1 General

Consultation of the 1:250 000 Port Shepstone (3030) geological map indicates that the investigated area is underlain by granite of the Oribi Gorge Suite and Natal Group sandstone with the residual and colluvial material derived therefrom.

5.2 Colluvium

The colluvium encountered on the route ranged between being dark brown, medium dense, clayey gravelly sand to slightly clayey to clayey sand containing weathered granite and sandstone.

5.3. Talus

On the lower slope talus was encountered being reddish brown, loose to medium dense, clayey sand containing weathered sandstone and granite cobbles and boulders.

5.4 Alluvium

Alluvium was encountered next to the Mtwalume River and comprised of brown, loose, gravelly sand containing cobbles and boulders of sandstone and granite, becoming dark grey, loose, slightly clayey to clayey sand.

5.5 Oribi Gorge Suite Granite

The granite encountered along the route ranges from being completely weathered to highly weathered, orange brown to pinkish white, soft rock to medium hard rock granite. The granite exposed in the Mtwalume river is moderately weathered, whitish brown and yellow, medium hard to hard rock.

The residual granite encountered along the route ranges from being reddish brown, medium dense, slightly clayey silty sand to reddish orange brown, firm to stiff, sandy silty clay.

5.6 Natal Group Sandstone

A sandstone outcrop is located in close proximity to the route, the sandstone being brown, moderately to slightly weathered, horizontally bedded (326 / 02°), moderately to closely jointed, medium to hard rock. The sandstone outcrop is the origin of the tallus boulders located along the route.

6. LABORATORY RESULTS

6.1 Talus

The talus material sampled in IP 6 and7, classifies as clayey sand with a clay content ranging between 16.7 and 30.8 % and a grading modulus of between 0.91 and 1.85. The material sampled has a Liquid Limit of between 32 and 37, a Plasticity Index of between 12 and 13 and a Linear Shrinkage of between 6 and 7 %.

In terms of the Revised US Classification, the material classifies as between A-2-6 (0) and A-6 (2), with a 'low' expansiveness potential in terms of van der Merwe's Classification System.

The maximum Mod AASHTO density of the tallus varies between 1921 kg/m³ and 2033 kg/m³ at an optimum moisture content of 9.4 % and 2.17 %. The CBR values of the material sampled varies between 2 and 6 at 90% Mod AASHTO dry density and between 13 and 14 at 100% Mod AASHTO dry density. The CBR swell of the talus material sampled ranges between 1 and 2.2 %, as such the material classifies as >G10 to G10 in terms of TRH 14 (1985).

6.2 Alluvium

Two samples of alluvium was sampled from IP 8 and classifies as a poorly graded silty sand to gravel with a clay content of between 4 and 6 % and a grading modulus of between 1.92 and 2.24. The material sampled has a Liquid Limit of between 22 and 23, a Plasticity Index of 0 and a Linear Shrinkage of 0 %.

In terms of the Revised US Classification, the material classifies as between A-1-a (0) and A-1-b (0), with a 'low' expansiveness potential in terms of van der Merwe's Classification System.

The maximum Mod AASHTO density of the alluvium varies between 2133 kg/m³ and 2173 kg/m³ at an optimum moisture content of 6.6 % and 6.7 %. The CBR values of the material sampled varies between 23 and 50 at 90% Mod AASHTO dry density and between 113 and 131 at 100% Mod AASHTO dry density. The CBR swell of the alluvium material sampled ranges between 0.1 and 0.3 %, as such the material classifies as G5 in terms of TRH 14 (1985).

6.3 Residual Granite

The residual granite material sampled from IP 2 and IP 4, classifies as silty sand to a high plasticity silt with a clay content ranging between 13.3 and 54.3 % and a grading modulus of between 0.55 and 1.04. The material sampled has a Liquid Limit of between 37 and 53, a Plasticity Index of between 10 and 19 and a Linear Shrinkage of between 5 and 9 %.

In terms of the Revised US Classification, the residual granite material sampled classifies as between A-2-4 (0) and A-7-5 (13), with a 'low' expansiveness potential in terms of van der Merwe's Classification System.

The maximum Mod AASHTO density of the residual granite varies between 1851 kg/m³ and 1868 kg/m³ at an optimum moisture content of 12 %. The CBR values of the material sampled varies between 2 and 4 at 90% Mod AASHTO dry density and 6 at 100% Mod AASHTO dry density. The CBR swell of the residual granite material sampled ranges between 3.57 and 4.76 %, as such the material classifies as greater than G10 in terms of TRH (1985).

6.4 Weathered Granite

Weathered granite was sampled from IP 1, 3 and 5 and classifies as a well to poorly graded sand to a clayey gravel with a clay content of between 5 and 26 % and a grading modulus of between 0.67 and 2.18. The material sampled has a Liquid Limit of between 21 and 40, a Plasticity Index of between 4 and 15 and a Linear Shrinkage of between 1 and 8 %.

In terms of the Revised US Classification, the material classifies as between A-1-b (0) and A-6 (4), with a 'low' to 'moderate' expansiveness potential in terms of van der Merwe's Classification System.

The maximum Mod AASHTO density of the weathered granite varies between 1722 kg/m³ and 2200 kg/m³ at an optimum moisture content of 7.7 % and 19.6 %. The CBR values of the material sampled varies between 10 and 57 at 90% Mod AASHTO dry density and between 18 and 366 at 100% Mod AASHTO dry density. The CBR swell of the weathered granite material sampled ranges between 0.1 and 2 %, as such the material classifies as G5 to >G10 in terms of TRH 14 (1985).

The results from the UCS test carried out on a granite sample taken from the bedrock exposed in the river indicate that the rock strength is 59 MPa.

6.5 Borrow Pit (30° 19' 49.49" S 30° 19' 46.85" E)

Two weathered granite samples were collected from the borrow pit and classify as poorly graded to well graded silty sand with a clay content of between 4 and 5% and a grading modulus of 2. The material sampled has a Liquid Limit of between 28 and 31, a Plasticity Index of 4 and a Linear Shrinkage of 2%.
In terms of the Revised US Classification, the material classifies as A-1-b (0), with a 'low' expansiveness potential in terms of van der Merwe's Classification System.

The maximum Mod AASHTO density of the weathered granite varies between 2113 kg/m³ and 2114 kg/m³ at an optimum moisture content of 5.8 % and 6.2 %. The CBR values of the material sampled varies between 18 and 28 at 90% Mod AASHTO dry density and between 122 and 143 at 100% Mod AASHTO dry density. The CBR swell of the weathered granite material sampled is 0.01 %, as such the material classifies as G5 to G6 in terms of TRH 14 (1985).

7. GEOTECHNICAL ASSESSMENT

7.1 Excavatablity

'Soft' excavation along the route is expected to depths in excess of 2.0m with normal earthmoving equipment in terms of SABS 1200.

Boulder excavation class will apply in areas marked as tallus boulder zones and alluvial boulder zones as shown on Figure 1.

7.2 Slope Stability and Erosional Potential

Numerous erosional gullies were encountered along the proposed route, indicated on Figure 1, some showing signs of material slumping.

A significant proportion of the proposed route is located in areas of talus.

Appropriate storm water control is essential on site both during and after construction phase.

7.3 Seepage

Seepage was only encountered in IP 8, in the river flood plain, which caused pit sidewall collapse. Seasonal seepage may be encountered along the contact between soil and rock during the rainy season.

7.4 Subsoil Consistency

The results of the DCP probe testing indicate that bedrock is encountered between 1.2 and 6.0 m below current ground level.

The DCP test results for probes conducted along the upper portion of the route, DCP 1- 5, indicate medium dense to dense subsoil consistency before refusal.

DCP 6 to 8 indicate that loose to medium dense material is present before refusing on bedrock, with DCP 9 refusing on alluvial boulders.

The results from DCP 10 indicate that loose to medium dense subsoil is present, before refusal of granite bedrock at 1.0 m

7.5 Material Suitability

It is evident from the laboratory test results that the materials sampled along the route ranges from G5 to >G10.

The tallus material tested classifies as G10 to worse than G10, thus the tallus could be used for general fill and subgrade purposes, care should however be taken in avoiding using the more clay rich tallus.

The alluvium sampled classified as G5 material and is thus suitable for use as sub-base material.

It is recommended that the residual granite not be used as a general fill and moved to spoil. The weathered granite material sampled from the route classified as G5 to worse than G10, and will be suitable for use as sub-base. However care should be taken with regards to the clay and silt rich completely weathered granite, which should be avoided.

The weathered granite sampled from the borrow pit classifies as G5 to g6 and is suitable for use as sub-base material.

Base course material will have to be source from a commercial source.

8. DEVELOPMENT RECOMMENDATIONS

8.1 <u>Earthworks</u>

8.1.1 *Cuts*

Cutting is considered feasible for the proposed route, provided the cut banks are trimmed to a satisfactory batter. It is recommended that for construction, the following batters be adopted; colluvial soils 1:2 (26°) and weathered bedrock 1:1.75 (may be increased subject to inspection).

8.1.2 *Fills*

Prior to the placing of any fill, the area must be stripped of all vegetation. Thereafter the fill should be placed in layers not exceeding 300mm loose thickness and compacted to 95 % Mod AASHTO density for sandy materials.

Rock fragments greater than two thirds of the layer thickness i.e. boulders (>200mm), must be removed to spoil.

Where fill embankments are to be constructed on relatively steep slopes, it is recommended that the fill is benched into competent weathered bedrock to promote compaction and increased stability. Benches should be a maximum of 3m wide to allow for appropriate roller compaction.

It is recommended that all fills be battered back at a maximum slope angle of 26° (1:2) provided that the fill material is well compacted in layers. Fill embankments must be suitably top-soiled and vegetated to limit the risk of erosion of the subsoils.

8.2 Lateral Support

Where large cuts are to be made exposing weathered granite, an assessment of the stability thereof may be required. Lateral support measures in these instances may include bolted meshing with or without guniting, however should be assessed on a site to site basis. Rock fall control may also be required in the tallus zone.

8.3 Site Drainage

Control of both surface and potential subsoil seepage is essential on this site to protect the proposed road layer works against the ingress of water.

Where the road cut intercepts seepage, subsoil drainage may be required.

Control of surface drainage should be by means of the installation of surface drains on the cut portion of the proposed road. A drainage pioneer layer prior to placing earthworks also aids in this regards.

8.4 Bridge Founding

Founding for the proposed bridge across the Mtwalume River should be taken into the hard granite bedrock underlying the site. However, due to collapse of the inspection pit sidewall on the southern flank of the river course, the investigation was unable to determine the exact depth to bedrock on the southern flank. Within the river course and on the northern flank of the river, hard rock granite was encountered.

It is our considered opinion that the bedrock levels on the southern flank of the Mtwalume River will vary between 3 and 6 m below current ground level as seen in the inferred geological cross section, Figure 12, attached to this report .

If the degree of uncertainty is unacceptable to the bridge design engineer then a drilling investigation will have to be conducted to determine the depth to bedrock on the southern flank.

All pier and abutment bases must be doweled at least 1.5 m into the underlying bedrock to prevent up-lift during periods of flooding. The positions of the dowels should be agreed with the Geotechnical Engineering.

Construction should be undertaken during the winter months to minimize the effects of seepage.

Excavations for the pier bases and abutment foundations on the southern flank will require shoring and continuous dewatering / pumping to construct.

The foundations should be air cleaned of all loose material along the top of the hard granite and a blinding layer should be cast prior to shuttering of the bridge bases.

River flood levels are important to determine bridge levels and approach fill protection.

We trust this meets with your immediate requirements in this matter however please feel at liberty to contact the author should you have any questions.

Yours faithfully
DRENNAN, MAUD CC

M J-F BÉNET Pr.Sci.Nat.

Encls.	Appendix A	-	IP’s 1 - 8; EXP 1-2
	Appendix B	-	Laboratory Test Results
	Figure 1	-	Site Plan
	Figures 2 - 11	-	DCP’s 1 - 10
	Figure 12	-	Inferred Geological Cross Section

/hn/gn

APPENDIX A

IP'S 1-8 & EXP 1-2

APPENDIX B

Laboratory Test Results

FIGURE 1

SITE PLAN

FIGURES 2 - 11

DCP’s 1 - 10

FIGURE 12

INFERRED GEOLOGICAL CROSS SECTION