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Report to GM Turner & Associates on the Results of a Geotechnical Investigation for the Proposed Bhudlu Vehicular Bridge over the Mtamvua River, KwaZulu-Natal

Reference: 092-15.R02

Dated: 16 July 2015

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List of abbreviations and expansions utilised in report

Abbreviation	Expansion
AASHTO	**American Association of State Highway and Transportation Officials**
BH	**Borehole**
CL	**Inorganic Clays of low to medium plasticity**
E	**East**
EGL	**existing ground level**
GM	**Grading Modulus**
GPS	**Global Positioning System**
IMC	**Insitu Moisture Content**
km	**kilometre**
kN/m^2	**kilo Newton per metre square**
LL	**Liquid Limit**
LS	**Linear Shrinkage**
m	**metre**
MAMSL	**metres above mean sea level**
mm	**millimetre**
MPa	**Mega Pascal**
No.	**number**
NWD4	**core barrel size**
NXC	**core barrel size, 75mm diameter**
PI	**Plasticity Index**
RHDHV	**Royal HaskoningDHV**
S	**South**
SANRAL	**South African National Roads Agency Ltd**
SANS	**South African National Standards**
SPT	**Standard Penetration Test**
UCS	**Uniaxial Compressive Strength**
USCS	**Unified Soil Classification System (1984)**

Report to GM Turner & Associates on the Results of a Geotechnical Investigation for the Proposed Bhudlu Vehicular Bridge over the Mtamvua River, KwaZulu-Natal

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Report to GM Turner & Associates on the Results of a Geotechnical Investigation for the Proposed Bhudlu Vehicular Bridge over the Mtamvua River, KwaZulu-Natal

Reference: 092-15.R02 **Date: 16 July 2015**

1. INTRODUCTION AND TERMS OF REFERENCE

Geosure (Pty) Ltd was invited by Royal HaskoningDHV (RHDHV) to tender to carry out a geotechnical investigation for the proposed Bhudlu Bridge and access road crossing the Mtamvua River near Harding in KwaZulu-Natal.

Geosure submitted a proposal and cost estimate to RHDHV in a letter referenced p050-15 – Rev 1/ng and dated 27 February 2015.

Subsequently, Geosure was appointed on 10 April 2015 by Mr S. Dhanbir on behalf of GM Turner and Associates to carry out the investigation as proposed by accepting and returning a signed copy of the Geosure letter referred to above.

This report provides an assessment of subsoil conditions encountered at the proposed bridge site. Comment is made on the general stability of the site. Recommendations for foundations, excavatability/rippability and general earthworks are given.

2. CODES OF PRACTICE AND STANDARDS

The services performed by *Geosure* were conducted in a manner consistent with the level of care and skill ordinarily exercised by members of the geotechnical profession practising under similar conditions in the locality of the project. No other warranty, expressed or implied, is made.

The investigation was carried out according to standard practice codes and guidelines relevant to geotechnical investigations.

The nature of geotechnical engineering is such that variations in soil conditions may occur even where sites seem to be consistent. Variations in what is reported here may become evident during construction and it is thus imperative that an appropriately qualified and experienced Competent Person inspects all critical stages of development including, but not limited to, excavations to ensure that conditions at variance with those predicted do not occur and to undertake an interpretation of the facts supplied in this report.

It is possible that certain indications of ground stability, contamination or groundwater levels were latent or otherwise not visible. Our opinions can only be based on what was visible at the time the investigation was conducted.

This report was prepared for use by the GM Turner & Associates, RHDHV and their professional team for the purpose stated and should not be relied upon for any other purpose.

3. INFORMATION UTILISED

For the purposes of assisting with this investigation, the following information was utilised:

- Global Positioning System (GPS) co-ordinates of the proposed road;
- An electronic copy of drawing no. T01.DUR.000413_GA_01 Rev A, titled "*Bhudlu River Bridge and Access Road*", dated 08 August 2015 and prepared by RHDHV to a scale of 1:1000;
- A 1:250 000 Geological Map titled "3028 Kokstad", dated 1988 and published by the Council for Geoscience of South Africa; and
- Low resolution aerial imagery sourced from Google Earth 2015.

4. SITE DESCRIPTION

The proposed bridge site is located approximately 21km south west of Harding at latitude and longitude 30°42'15.10"S and 29°44'26.3"E respectively. The Mtamvua River is approximately 20m wide where the proposed bridge crossing is located. The site comprises dense grass with minor tree vegetation. The site is accessed by either Roads D862 or D1100.

Topographically, the site is situated in a low-lying area.

The locality of the site is shown in Plate 1 below.

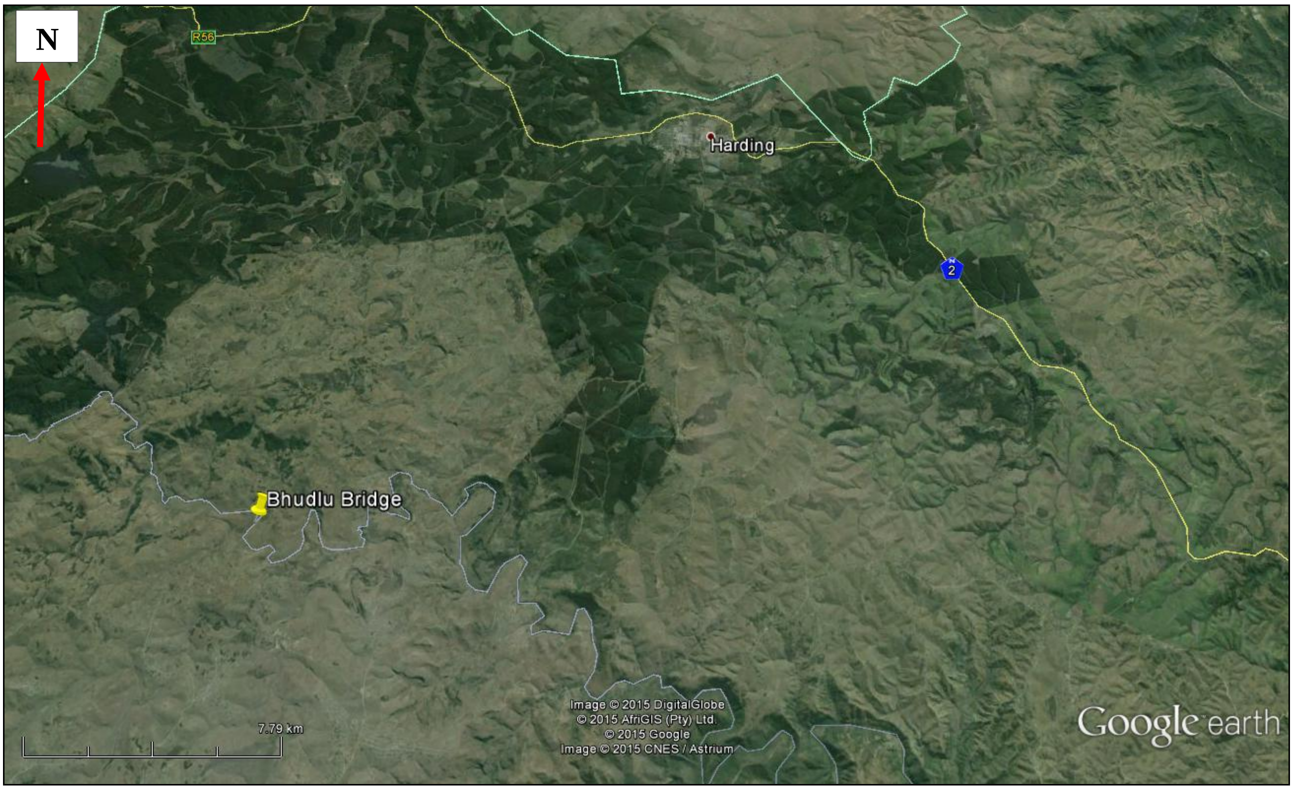

Plate 1: Locality Plan (*Source: Google Earth 2014*)

5. FIELDWORK

The fieldwork was carried out over the period 13 May 2015 to 26 May 2015 and comprised the following:

- Terrain appraisal; and
- Rotary core borehole drilling.

5.1 Terrain Appraisal

Prior to the subsurface investigation, a site walkover assessment was carried out to identify the following major features significant to the geotechnical character of the site:

a) Surface geology;

b) Topography, surface drainage patterns and related major geotechnical features relevant to the proposed development.

5.2 Borehole Drilling

Three boreholes were carried out across the bridge site at the positions given in Figure 201. The boreholes, designated BH1 through BH3, were carried out by a specialist contractor and advanced by a rotary drilling rig utilising NXC and NWD4 size core barrels. Standard Penetration Tests (SPT) were carried out at 1m intervals in the subsoil.

The boreholes were advanced to final depths in the range 7.0m (BH2) to 10.5m (BH3) below EGL.

All the borehole positions were surveyed by RHDHV prior to the commencement of the drilling contract. Borehole numbers, positions and elevations are summarised in Table 1 below.

The material recovered from the boreholes was profiled using the South African Geoterminology Guidelines (2002)[1].

Copies of the detailed borehole profiles are given in Appendix A.

Table 1: Summary of borehole positions (WGS84 Lo 31)

Borehole No.	Y - Co-Ordinate	X- Co-Ordinate	Z (Elevation in MAMSL)
BH1	70220.307	3399215.029	755
BH2	70220.307	3399207.006	752
BH3	70207.124	3399198.847	751

MAMSL - Metres above mean sea level

6. ANTICIPATED SUBSOIL CONDITIONS

Inferring from the geological map (refer to Plate 2), the general geology within the immediate vicinity of the site comprises alluvial deposits, Jurassic age dolerite and Estcourt Formation shale.

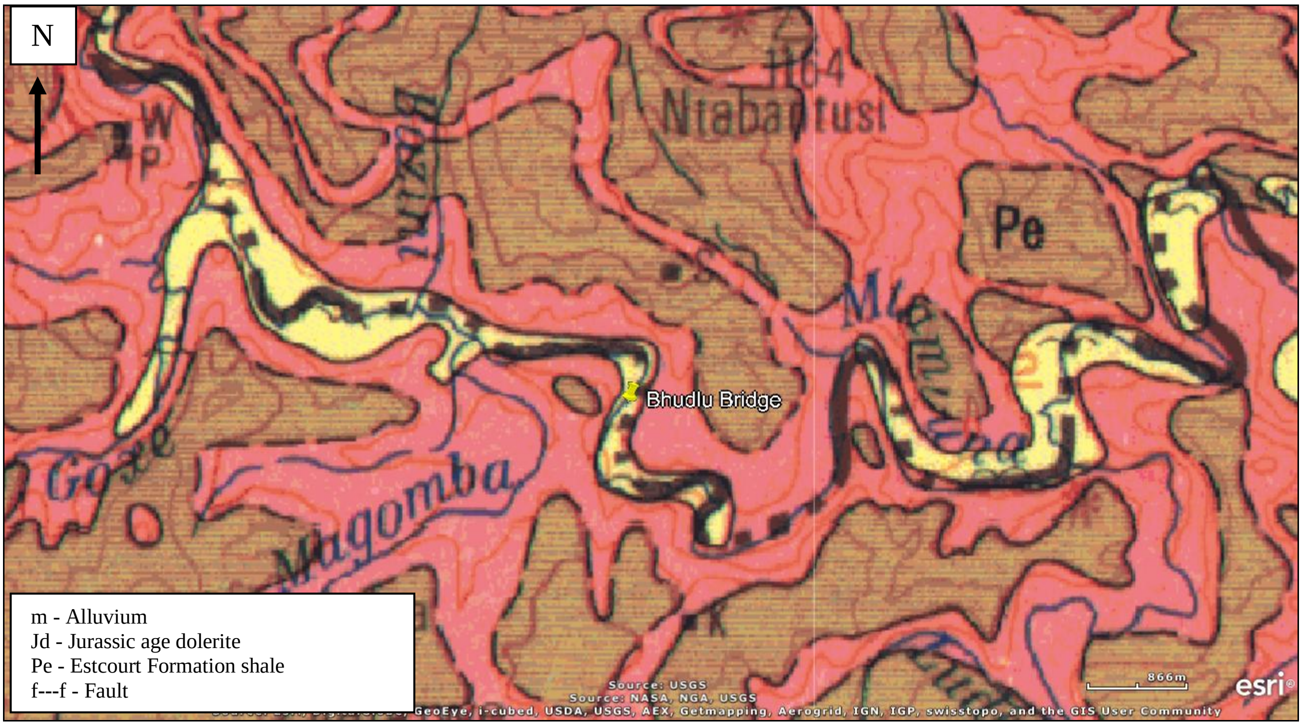

Plate 1: Regional geological map of Port Shepstone (Council for Geoscience, 1988)

Field observations are generally consistent with the above; however, no shale bedrock was encountered in any of the boreholes. The material recovered from the boreholes has been briefly described below:

- **<u>Alluvium (Unit 1)</u>** – Dark reddish brown, stiff, slightly gravelly to slightly silty sandy CLAY with roots. This unit was only observed in BH1 and extended to a depth 2.3m below EGL.

- **<u>Alluvium (Boulder Bed) (Unit 2)</u>** – Dark grey to bluish, sub-rounded cobbles to boulders comprising dolerite and shale. Boulder sizes range from 25mm to 550mm in diameter. This unit was encountered in all boreholes and extends to bedrock level at depths in the range 3.0m (BH2) to 6.5m (BH3) below EGL.

- **<u>Weathered Dolerite Bedrock (Unit 3)</u>** – Dark grey to bluish grey to dark greenish grey speckled white, moderately to slightly weathered, fine to medium grained, moderately fractured, hard to very hard rock. This unit was encountered in all boreholes at depths in the range 3.0m (BH2) to 6.5m (BH3) below EGL.

7. GROUNDWATER

Groundwater seepage was encountered in all boreholes at the depths indicated in Table 2 below. Groundwater levels are likely to fluctuate seasonally.

Table 2: Summary of Approximate Depth to Groundwater Table

Borehole No.	Approximate Depth (m) to Groundwater Table below EGL	Approximate Elevation of Groundwater Table in MAMSL
BH1	2.2	752.8
BH2	2.0	750
BH3	2.0	749

8. LABORATORY TESTS

The following laboratory tests were carried out on soil/rock samples retrieved during the investigation:

- Grading Analysis and Atterberg Limits;
- Hydrometer Analysis;
- Moisture Content; and
- Uniaxial Compressive Strength Tests.

The laboratory results are summarised in Tables 3 and 4 below and detailed results are given in Appendix B.

Table 3: Summary of Results of Particle Size Distribution Analysis, Atterberg Limit Determinations and Insitu Moisture Contents

BH No.	Depth (m)	Description	Particle Size %				*Atterberg Limits %			GM	IMC (%)	Material Code & Classification
			Clay	Silt	Sand	Gravel	LL	PI	LS			
BH1	0-1.5	Dark reddish brown, silty CLAY - Alluvium	58	25	14	3	43	17	8.5	0.2	20.9	A-7-6(15) CL *Low
BH2	1.5-1.95	Dark reddish brown, silty CLAY - Alluvium	40	26	22	12	40	16	8	0.6	27.5	A-7-6(9) CL *Medium

LL - Liquid Limit
PI - Plasticity Index
CL - Organic Clay with LL<50
IMC - Insitu Moisture Content
LS - Linear Shrinkage
*Low - Expansiveness According to van der Merwe (1964)
A-3 (0) - Revised U.S Classification
GM - Grading Modulus

Table 4: Summary of UCS Test Results Carried out on Rock Core Samples from Boreholes

BH No.	Depth (m)	Strength (MPa)
BH1	4-4.1	71.9
BH1	4.9-5.0	77.9
BH1	5.5-5.6	84
BH1	6.65-6.7	78.6
BH2	3.2-3.3	66.8
BH2	4.1-4.2	72.9
BH2	4.9-5.0	184.5
BH2	5.6-5.7	72.4
BH3	6.5-6.6	116.9
BH3	7.6-7.7	92.7
BH3	8.2-8.3	49.2
BH3	9.1-9.2	47.8

Soft Rock: 3-10 MPa **Medium Hard Rock: 10-25 MPa** **Hard Rock: 25-70 MPa** **Very Hard Rock: 70-200 MPa**

9. DEVELOPMENT GUIDELINES

9.1 Proposed Development

It is understood that the proposed development is to comprise a bridge crossing the Mtamvua River. Information from RHDHV indicates that the proposed bridge will comprise the following:

- A 6 Span structure;
- Approximately 76m in length;
- A 1m thick deck that is 6.5m wide;
- A single vehicular lane and pedestrian walk way; and
- Piers that are 0.6m thick.

Foundation loads for the piers were not available at the time of preparation of this report.

Geosure will need to be given the opportunity to review the recommendations in this report once detailed information regarding the design and layout of the proposed development is available.

9.2 General Stability of the Site

Based on the results of the fieldwork undertaken during this investigation, it is considered that this site is generally stable and suitable for development in its current state provided the recommendations given in this report are adhered to.

9.3 Excavatability and Rippability Assessment

The excavatability across the extent of the site is likely to be variable. Excavatability of the various units encountered has been based on the current version of SANS 1200.

Alluvial subsoils (Unit 1 and Unit 2) is anticipated to classify as SOFT. These conditions are inferred to occur to depths in the range 0.00m to 6.5m below EGL.

However, boulders were identified within the alluvial layer and when encountered these may result in slower excavation rates. These conditions are inferred to occur at depths in the range 0.00m to 6.5m below EGL and is anticipated to classify as INTERMEDIATE TO HARD or even BOULDER CLASS excavation.

The dolerite bedrock (Unit 3) is anticipated to classify as HARD.

The type of excavation plant and nature of the underlying bedrock will determine actual trenchability depths. Excavations within the alluvial units are likely to display rapid sidewall collapse, particularly below the groundwater table. Slow excavation rates are therefore considered likely.

9.4 General Earthworks

All earthworks should be carried out in a manner to promote stable development of the site. It is recommended that earthworks be carried out along the guidelines given in SANS 1200 (current version).

All vegetation should be removed from the areas over which fills are to be built. Where natural ground slopes are steeper than $1_{(vertical)}:6_{(horizontal)}$ (6 degrees), the fill must be benched into the slope. Benches should be minimum 0.5m deep and 2.0m wide. A minimum of three benches per fill is recommended.

Placement of fill layers should be undertaken in layers not exceeding 200mm thick when placed loose and compacted using suitable compaction plant to achieve 93% Modified AASHTO maximum dry density at ±2% optimum moisture content. Boulders larger than $^{2}/_{3}$ of the layer thickness must not be included in the fill material. A carefully engineered fill embankment should not settle more than 0.5% of its height due to self-weight.

Density control of placed fill material should be undertaken at regular intervals during fill construction.

Cut in subsoils and fill slopes soils should be formed to batters of 1 (vertical) to 2 (horizontal) and to a height not greater than 3 metres where retaining walls are not provided.

Cuts in weathered bedrock should be formed to batters of 1 vertical to 0.75 horizontal and to a height not greater than 3 metres where retaining walls are not provided.

Although not encountered in the boreholes, shale bedrock is likely to occur in close proximity to the site. If encountered, shale slopes may be susceptible to slope instability, in particular block sliding on thin clay layers along bedding planes that daylight in cuttings. In similar areas, seepage was often found to be the triggering mechanism. Therefore, the control of stormwater above cuttings will need to be carefully designed with stormwater guided away from the crest and face of cuttings. It is recommended that a geotechnical specialist be appointed to inspect the cutting and assess the global stability of the slope during construction. Shale bedrock is also known to slake and breakdown to a soil when exposed, and therefore consideration should be given to covering the cutting with shotcrete immediately.

Engineered fill slopes should be over constructed and thereafter trimmed back to the required position.

Cut and fill heights greater than 2 metres would need to be inspected and approved by an engineering geologist or geotechnical engineer.

Sidewall collapse of excavations not battered back or shored is considered likely.

Workers should not enter any excavation deeper than 1.2 metres that is not shored or battered back as described above. Steeper batters can be considered but will be subject to inspection and approval by a geotechnical professional on site during construction.

Due to the prevalent shallow groundwater condition and loosely consolidated nature of the Units 1 and 2, workers should not enter any excavations deeper than 1.2m that are not shored or battered back. It remains, however, the responsibility of the contractor/engineer on site to ensure excavations are safe and shored in line with requirements as set down in the current "Occupational Health and Safety" Act 85 (1993 as amended).

9.5 Foundation Recommendations

All foundation loads should be designed to act in end-bearing. It is recommended that foundations are taken down through the alluvial material (subsoil and boulders) and placed on or socketed into the underlying weathered dolerite bedrock.

The approximate depths of weathered bedrock identified in the boreholes are given in Table 5 below.

Table 5: Summary of approximate depth to weathered bedrock

Borehole No.	Approximate depth to bedrock below EGL (m)	Approximate Elevation of weathered bedrock in MAMSL
BH1	4.0	751
BH2	3.0	749
BH3	6.5	744.5

Dolerite bedrock was observed to occur at depths in the range 3.0m to 6.5m below EGL. Due to the significant depth to bedrock and potentially unstable sidewalls of excavations, it is considered that shallow spread footings will not be feasible for the site. The following foundation types have been considered for the proposed development:

- Caissons; and/or
- Piled Foundations

9.5.1 Caissons

The caissons must be taken down into competent weathered bedrock of at least medium hard rock strength, where a maximum allowable bearing pressure of 2000kN/m^2 is considered applicable. The need to anchor the caissons into bedrock will have to be assessed by the structural engineer.

Use of caissons could avoid the need for lateral support, but it is considered that de-watering will be necessary.

Care should to be taken when sinking the caisson through the boulders in order to minimise the risk of hang up on large boulders, and local damage to the cutting edge and the adjacent caisson wall. Installation of the caisson through these layers is likely to be time consuming.

It is recommended that all foundation excavations be inspected and approved by Geosure (Pty) Ltd prior to blinding and casting concrete.

9.5.2 Piled Foundations

It is preferred to support the bridge on a piled foundation due to the following reasons:

- Significant depth to competent bedrock in some cases up to 6.5m below EGL;
- Shallow groundwater conditions, with majority of the foundations being installed within the existing riverbed; and
- Significant thickness of boulder bed horizon, up to 6.5m below EGL.

It is quite likely that a number of the piles are to act in tension caused by debris loads on the bridge structure. In addition, the aspect of scour of the alluvial soils may also require that piles be socketed into the bedrock. Piles will need to penetrate the alluvial boulder bed in order to achieve the above.

It is therefore recommended that only the following pile types be considered:

- Oscillator piles; and/or
- Rotapiles.

Both these pile types are able to penetrate alluvial boulder beds of significant thickness.

Provided the piles are socketed into competent weathered bedrock of at least medium hard rock strength, a maximum nett allowable bearing pressure of 2000 kN/m^2 is considered applicable, subject to verification by ongoing laboratory testing. Higher allowable bearing pressures can be considered but will need to be subject to inspection and confirmation by the geotechnical professional. The approximate loads given in Table 6 may be adopted for the design of piles.

Table 6: Details of Various Pile Types

Pile Type	Diameter (mm)	*Approximate Allowable Pile Load (kN)	Maximum Rake
Oscillator Piles	900 (lined) 1100 (lined) 1350 (lined)	6500 9500 14250	1: 4
Rotapiles	255 (lined) 305 (lined) 355 (lined) 406 (lined) 457 (lined) 610 (lined)	300-450 450-600 600-900 800-1200 1000-1500 1500-2500	1: 4 1: 4 1: 8 1: 8 1: 8 1: 8

* - Working Loads calculated using a shaft stress of 10MPa which can be considered when socketed into hard rock.

For both pile types permanent lining is recommended in order to protect the wet concrete of the pile shaft from the strong flow of groundwater.

Consideration will need to be given to the correct selection of an appropriate pile size for the rotapile as slender pile sizes may be prone to buckling effects and will need to be carefully considered.

Piles will need to be socketed into competent bedrock. Penetration into the bedrock will depend on the hardness of the rock and fracture frequency. Consideration should be given to socketing piles into bedrock by at least 2 to 4 metres. Taking this into account pile lengths are likely to be in the range 6 to 10 metres (for budgeting purposes). Piles should have a minimum length of 6 metres.

For the above pile types founded within the bedrock, it is anticipated that the maximum settlement will be less than 5mm. The need to install piles at raked angles to counteract horizontal loads will need to be determined from the hydraulic and structural analysis.

A detailed pile design will need to be carried out by the contractor. This design should be submitted to Geosure for comment.

10. QUALITY ASSURANCE TESTING DURING CONSTRUCTION

Due to the variations in geotechnical conditions encountered at the bridge site it is recommended that adequate supervision by Geosure be allowed for. This is to ensure that foundations for the bridge structure are properly socketed into bedrock and not founded on alluvial boulders.

Should the piling option discussed above be utilised for the proposed bridge structure, it is recommended that all piles be subjected to integrity testing that should comprise cross-hole ultrasonic testing. At least three 50mm steel tubes will need to be cast in the pile at even spacing around the perimeter. These tubes will need to be cleaned and filled with water prior to the test. The tests will need to be carried out by an independent consultant. This will need to be specified in the piling contract document.

Alternatively, if not practical to carry out the above, sonic impact tests should be considered.

11. ADDITIONAL BOREHOLE INVESTIGATION

Due to the variation in elevation of competent bedrock levels, in some cases up to 5.5m, it is recommended that additional boreholes be carried out at each pier position. This will provide a better understanding of the founding levels at each pier position and avoid the risk of placing foundations on boulders. Furthermore, no boreholes were carried out within the river bed and competent bedrock can occur at greater depths than the boreholes drilled on the embankment.

This investigation should be carried out in accordance with the South African National Roads Agency Ltd (SANRAL) requirement given in the following documents:

- Code of Procedure for the Planning and Design of Highway and Road Structures in South Africa, February 2002; and

- South African Pavement Engineering Manual, Chapter 7, Geotechnical Investigations and Design Considerations, January 2013.

12. CONCLUSION

This report provides an assessment of subsoil conditions encountered at the proposed bridge site. Comment is made on the general stability of the site. Recommendations for foundations, excavatability/rippability and general earthworks are given.

Based on the results of the fieldwork undertaken during this investigation, it is considered that this site is generally stable and suitable for development, provided the recommendations given in this report are adhered to.

The bridge site is observed to be underlain by alluvial subsoil, alluvial boulders and dolerite bedrock. Dolerite bedrock was observed to occur at depths in the range 3.0m to 6.5m below EGL.

Groundwater seepage was encountered in all boreholes at depths in the range 2.0m to 2.2m below EGL. Therefore, a shallow groundwater condition is considered likely. Due to close proximity to the river, groundwater levels are likely to fluctuate both during and after periods of rainfall.

All foundation loads should be designed to act in end-bearing, founded in the underlying competent dolerite bedrock.

Dolerite bedrock was observed to occur at depths in the range 3.0m to 6.5m below EGL. Due to the significant depth to bedrock and potentially unstable sidewalls of excavations, it is considered that shallow spread footings will not be feasible for the site. The following foundation types have been considered for the proposed development:

- Caissons; and/or
- Piled Foundations.

Due to the presence of boulders, strong groundwater flow and significant depth to bedrock, it is preferred to support all loads on a piled foundation. In this regard, it is considered that the following pile types will be suitable for the site conditions:

- Oscillator piles; and/or
- Rotapiles.

Due to the variation in elevations of bedrock level, in some cases up to 5.5m, it is recommended that additional boreholes be carried out at each pier position as per the guidelines provided in the SANRAL documents referred to in Section 11.

The ground conditions given in this report refer specifically to the field tests carried out on site. It is therefore, quite possible that conditions at variance with those given in this report can be encountered elsewhere on site during construction. It is therefore important that Geosure (Pty) Ltd be appointed to carry out a strict quality assurance program during construction. Any change from the anticipated ground conditions could then be taken into account to avoid unnecessary expense.