

GEOTECHNICAL REPORT

FOR THE PROPOSED DIKIDIKINI PEDESTRIAN BRIDGE, NTABANKULU LOCAL MUNICIPALITY, PROVINCE OF THE EASTERN CAPE

31 January 2023 (Rev 1)



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Declaration of independence:

The author of this report is independent professional consultant with no vested interest in the project, other than remuneration for work associated with the compilation of this report.

General limitations:

1. The investigation has been conducted in accordance with generally accepted engineering practice, and the opinions and conclusions expressed in the report are made in good faith based on the information at hand at the time of the investigation.
2. The contents of this report are valid as of the date of preparation. However, changes in the condition of the site can occur over time as a result of either natural processes or human activity. In addition, advancements in the practice of geotechnical engineering and changes in applicable practice codes may affect the validity of this report. Consequently, this report should not be relied upon after an elapsed period of one year without a review by this firm for verification of validity. This warranty is in lieu of all other warranties, either expressed or implied.
3. Unless otherwise stated, the investigation did not include any specialist studies, including but not limited to the evaluation or assessment of any potential environmental hazards or groundwater contamination that may be present.
4. The investigation is conducted within the constraints of the budget and time and therefore limited information was available. Although the confidence in the information is reasonably high, some variation in the geotechnical conditions should be expected during and after construction. The nature and extent of variations across the site may not become evident until construction. If variations then become apparent this could affect the proposed project, and it may be necessary to re-evaluate recommendations in this report. Therefore, it is recommended that Outeniqua Geotechnical Services is retained to provide specialist geotechnical engineering services during construction in order to observe compliance with the design concepts, specifications and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. Any significant deviation from the expected geotechnical conditions should be brought to the author's attention for further investigation.
5. The assessment and interpretation of the geotechnical information and the design of structures and services and the management of risk is the responsibility of the appointed engineer.

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1. Introduction

1.1 Background information

Outeniqua Geotechnical Services (OGS) was appointed by Uhambiso Consult to conduct a geotechnical drilling investigation for the proposed new Dikidikini pedestrian bridge over the Mzintlava River approximately 68km northeast of Mthatha in the Province of the Eastern Cape (see **Figure 1**). The proposed bridge was a single span Mabey pre-fabricated steel frame bridge with reinforced concrete abutments. The geotechnical nature of the site was investigated to facilitate the structural design of the bridge foundations.

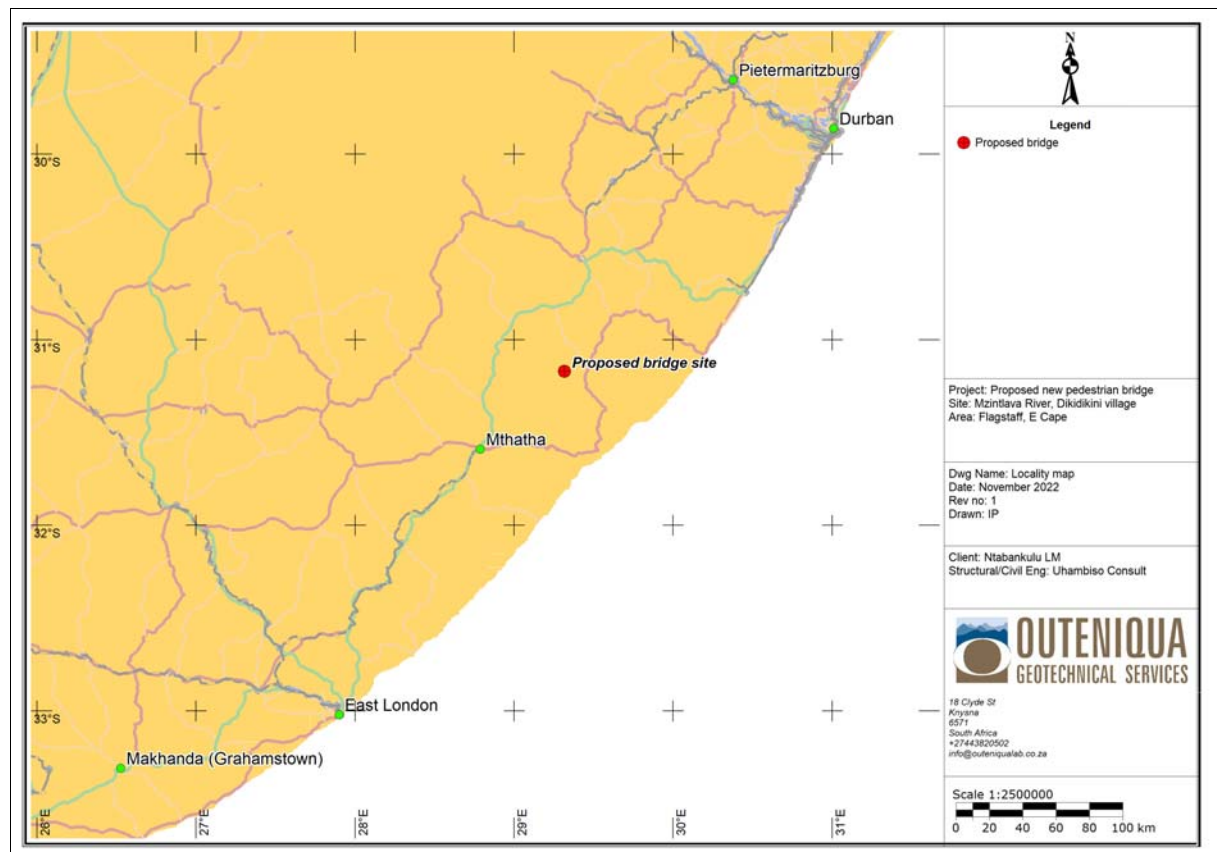


Figure 1: Locality map

1.2 Scope of work

The scope of work for the investigation was prescribed as follows:

1. Review the geological and geotechnical data for the area.
2. Conduct a drilling investigation consisting of the following methods:
 - 4x Rotary core boreholes to depth of 15m.
 - Log and photograph core by qualified engineering geologist according to the SAICE Guidelines for Soil and Rock logging.
 - Collect and transport of soil/rock samples for testing at SANAS-accredited civil engineering laboratory.
3. Analyse results and prepare a factual and interpretive report containing all information from the investigation and including recommendations for the design of earthworks and foundations.

1.3 Available information

The following maps & plans were available for consultation:

- 1:250 000 Geological map of the area, obtained from the Council for Geoscience.
- Topo-cadastral data for the area, obtained from the National Geospatial Institute (NGI).
- Aerial photos of the area, obtained from the NGI and Google Earth.
- Site layout plans including proposed bridge design, obtained from Uhambiso Consult.

2. Site description

The site for the proposed pedestrian bridge was accessed via gravel roads off the R61, approximately 23km northbound from Lusikisiki or 18km southbound from Flagstaff. The proposed bridge was located on the Mzintlava River at a place where the local community typically cross the river to access the remote community on the western side of the river. There was no other access to this community on the western side of the river, hence the need for a safe river crossing (see **Figures 2-3**).

The topography on the eastern side of the river was described as a gentle slope down to the rivers edge (see **Figure 4**). By contrast, the topography on the western side was characterised by a steeper incline up from the waters edge onto a flood plain at an elevation of about 3-4m above the river level. See **Figure 5**.

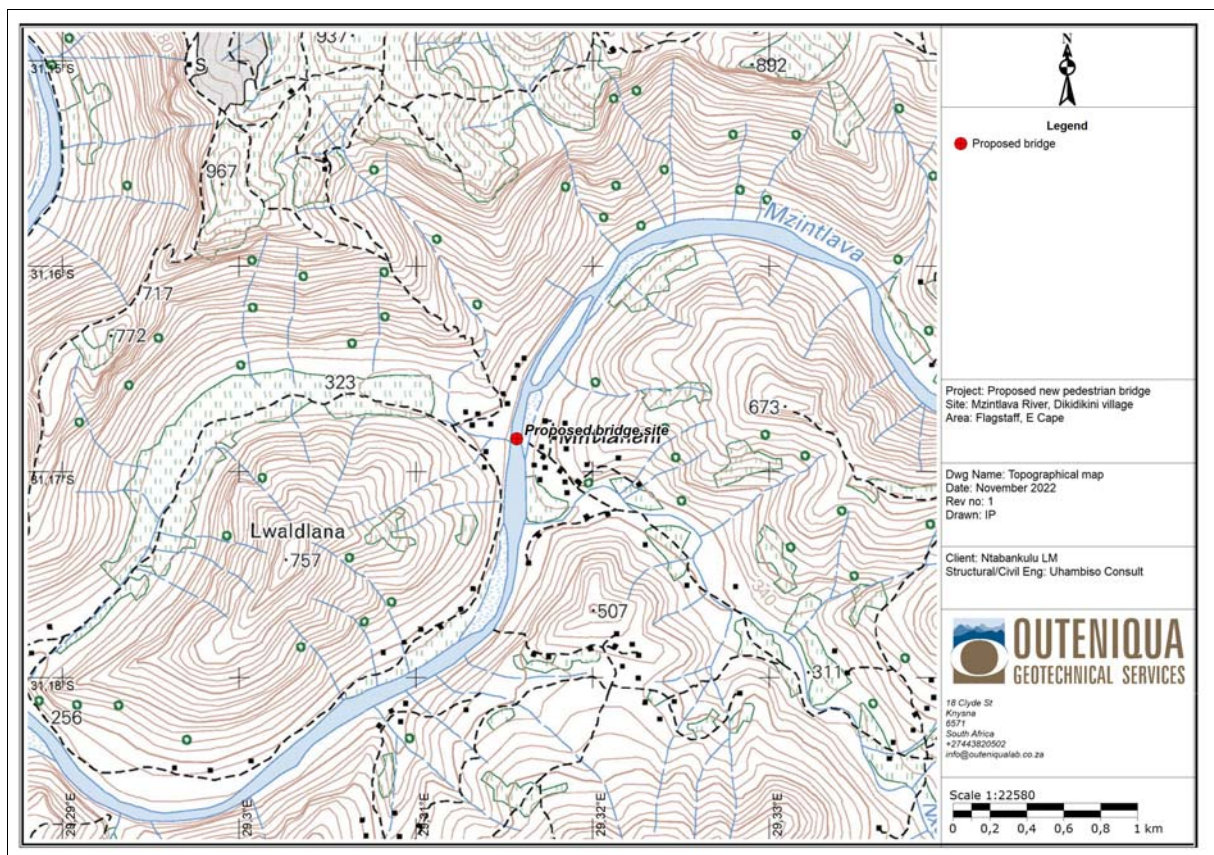


Figure 2: Topographical map of the site

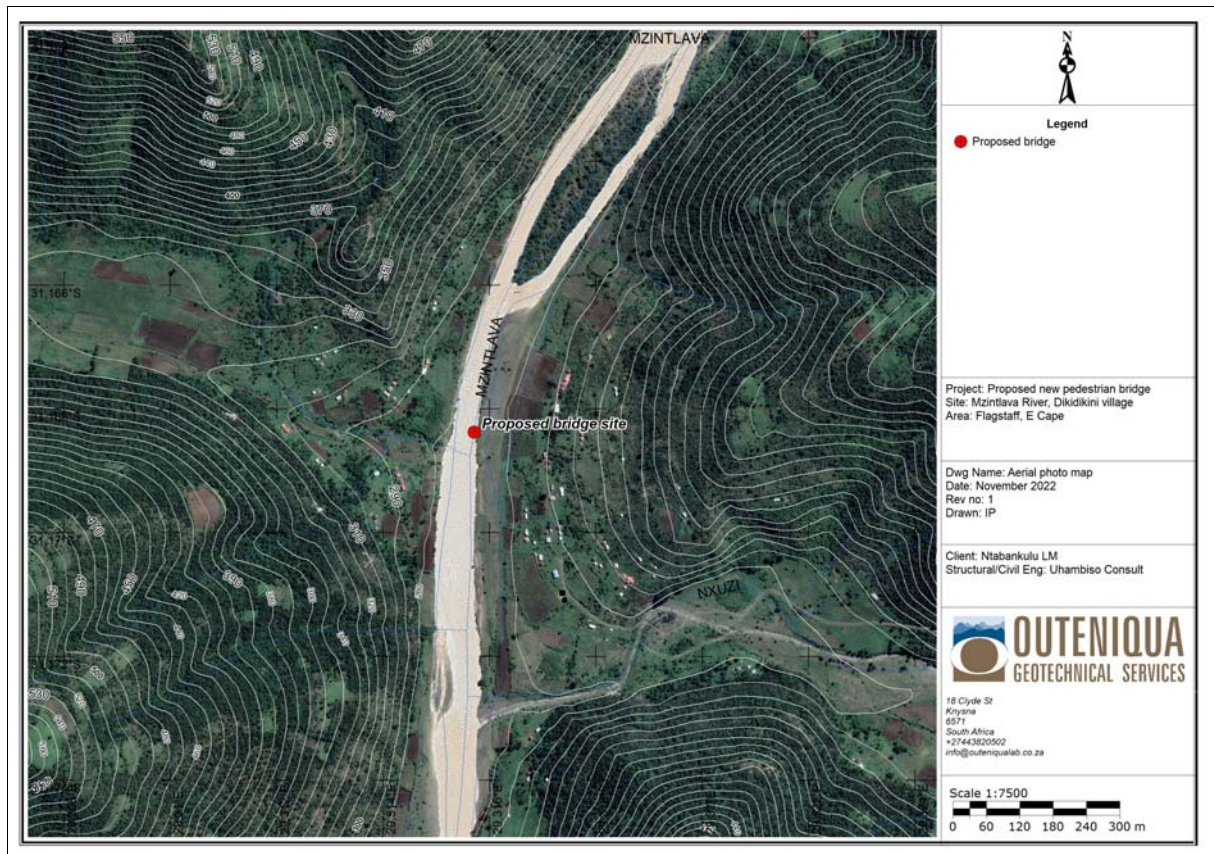


Figure 3: Aerial photo map of site



Figure 4: View of the proposed bridge site from the eastern side, looking across to the western side



Figure 5: View of the western side of the river

3. Methods of investigation

An initial desk study of available information was conducted to assess the site topography, geology and any obvious geotechnical issues. This was followed by a subsurface investigation consisting of four rotary core boreholes, drilled vertically to a depth of 15m to determine the subsurface profile. Three boreholes were drilled on the east side of the river (BH1, 2 & 4) and one was drilled on the western side (BH3 - see **Figure 6**). The borehole logs were included in **Appendix 2** of this report.



Figure 6: Map of borehole positions

Representative samples of rock core were collected from the boreholes for unconfined compressive strength tests with strain measurements (UCM). The tests were performed

at Rock Lab, in accordance with the ASTM methods. No other tests, such as triaxial, consolidation or Foundation Indicator tests were performed as the nature of the formations did not warrant such tests. Details of the tests were included in **Appendix 3** of this report.

In situ standard penetration tests (SPT) were attempted during drilling but generally abandoned due to the presence of large cobbles and boulders which frustrated the tests.

An assessment of the data was then performed by a qualified and experienced geotechnical professional. The outcome of the investigation was a formal factual and interpretive report.

4. Results of the site investigation

4.1 Regional and local geology

The 1:250 000 geological map indicated that the area was underlain by an extensive system of dolerite sills and dykes of Jurassic age (indicated in red in **Figure 7**), which had intruded into dark grey shale of the Eccia Group (indicated in brown in **Figure 7**). The dolerite and shale were well exposed in road cuttings near the site (See **Figures 8 & 9**). Locally, thick deposits of alluvium covered the bedrock formations along the river banks (See **Figure 10**).

There were no major geological faults within a radius of 50km of the site and the risk of seismic activity in the area was low.

The geology of the site was generally considered suitable for the proposed development with due consideration to local geotechnical constraints.

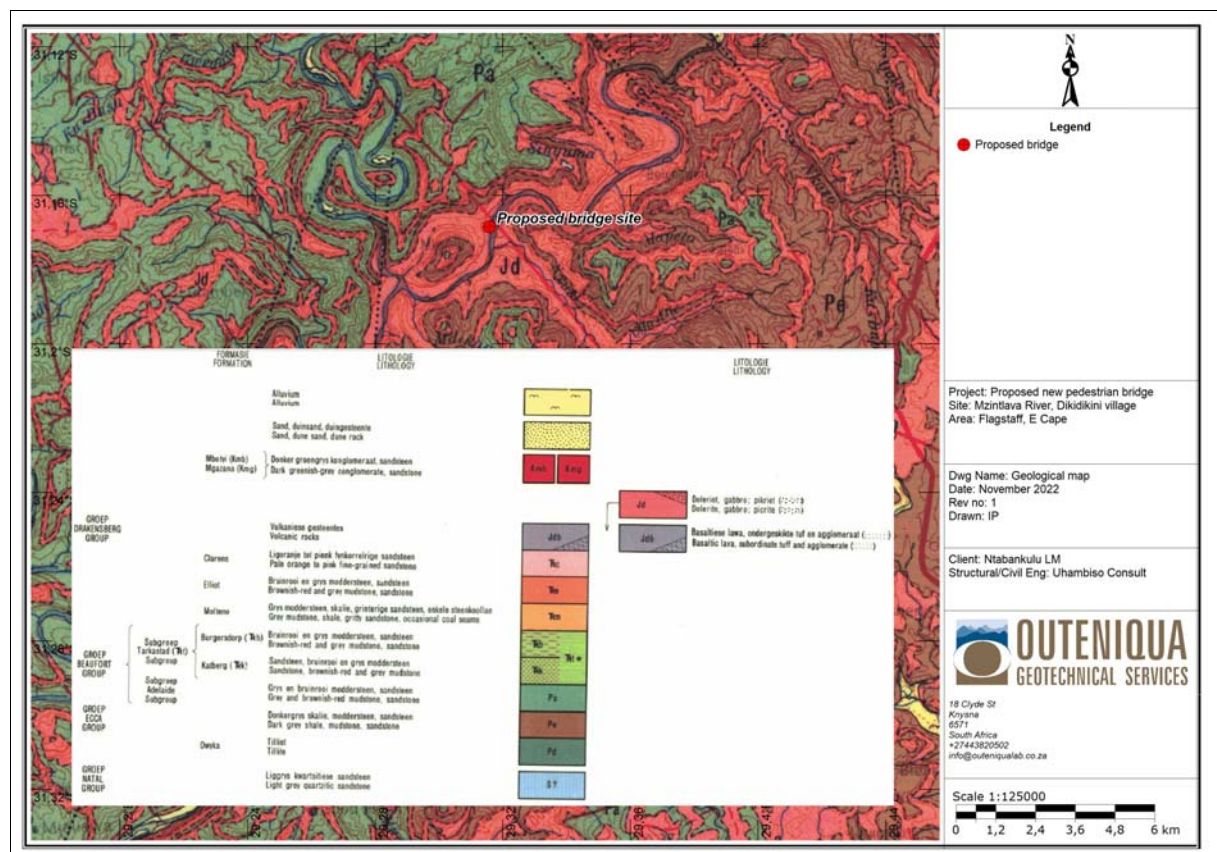


Figure 7: Geological map of site

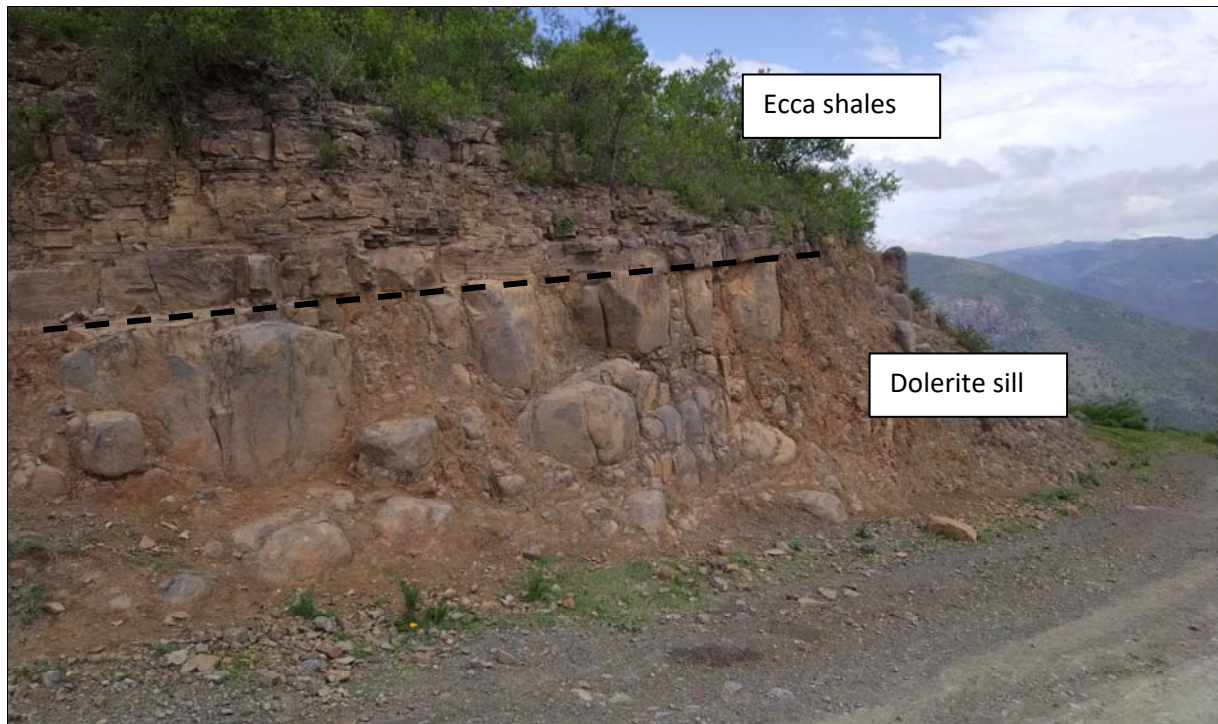


Figure 8: Dolerite sill intruded into shale of the Ecce Formation exposed in road cutting near site



Figure 9: Grey shales of the Ecce Group exposed in local borrow pit



Figure 10: Alluvial cobbles and boulders covering the riverbanks

4.2 Drilling data

The three boreholes on the eastern side of the river encountered coarse alluvium (cobbles and boulders in a sandy/gravelly matrix) to a depth of approximately 6-8m before penetrating into the underlying shale and dolerite rock. The borehole on the western side encountered similar profile with coarse alluvium to a depth of approximately 8m and then shale or dolerite rock below this depth. Shallow water tables were recorded in all boreholes due to the close proximity of the river.

A summary of the borehole data was given in **Table 1**.

Table 1: Summary of borehole data (in m)

<i>BH No.</i>	<i>Alluvium</i>	<i>Rock</i>	<i>Final depth</i>	<i>Water Table</i>
BH1	0-7.45	7.45-14.81	14.81	1.03
BH2	0-7.79	7.79-14.20	14.20	1.50
BH3	0-8.00	8.00-15.00	15.00	2.00
BH4	0-10.80	10.80-15.50	15.50	1.50

Core recoveries in the alluvium were low due to the extremely difficult drilling and most of the sandy matrix was lost in the process. The exact contact of the underlying rock was also difficult to determine due to the broken nature of the rock. Fracture frequency was typically high and the rock was soft to medium hard or hard with slight to moderate weathering. RQD's were typically low (<50%) and often zero, indicating a highly fractured rockhead.

4.3 Laboratory tests

Intact rock samples were collected for unconfirmed compression and point load tests to estimate bearing capacity. The results of the tests were shown in **Table 2**.

Table 2: Summary of UCS & PLT test results

<i>BH No.</i>	<i>Sample Depth (m)</i>	<i>Rock type</i>	<i>UCS MPa</i>	<i>PLT I_{s50} MPa</i>	<i>Indicative UCS</i>
BH1	8.84-9.05	Shale	430.7		
BH1	12.00-12.17	Shale	449.8		
BH1	9.42 - 9.51	Dolerite		3.8 ^d	87.4
BH1	9.42 - 9.51	Dolerite		8.5 ^a	195.5
BH1	11.57 - 11.66	Shale		6.4 ^d	147.2
BH1	11.90 - 11.97	Shale		0.9 ^d	20.7
BH1	11.90 - 11.97	Shale		3.4 ^a	78.2
BH1	11.90 - 11.97	Shale		9.1 ^a	209.3
BH1	13.92 - 14.01	Shale		0.6 ^d	13.8
BH2	10.00-10.12	Shale	202.2		
BH2	10.68-10.80	Shale	118.2		
BH2	10.25-10.40	Shale		4.2 ^d	96.6
BH2	10.25-10.40	Shale		8.2 ^d	188.6
BH2	10.25-10.40	Shale		5.0 ^a	115.0
BH2	10.25-10.40	Shale		3.6 ^a	82.8
BH2	10.25-10.40	Shale		9.3 ^a	213.9
BH2	10.45-10.50	Shale		8.8 ^d	202.4
BH2	10.45-10.50	Shale		11.5 ^a	264.5
BH2	10.86-10.95	Shale		8.9 ^d	204.7
BH2	10.86-10.95	Shale		5.2 ^a	119.6
BH2	10.86-10.95	Shale		3.1 ^a	71.3
BH2	11.09-11.15	Shale		6.7 ^d	154.1
BH2	11.09-11.15	Shale		7.8 ^a	179.4
BH2	11.09-11.15	Shale		5.6 ^a	128.8
BH3	8.60-8.77	Shale	467.3		
BH3	10.80-11.00	Shale	485.7		
BH3	8.01-8.15	Shale		11.3 ^d	259.9
BH3	8.01-8.15	Shale		11.2 ^a	257.6
BH3	8.01-8.15	Shale		9.5 ^a	218.5
BH3	8.40-8.45	Shale		11.5 ^d	264.5
BH3	8.40-8.45	Shale		6.0 ^a	138.0
BH3	8.40-8.45	Shale		2.9 ^a	66.7
BH3	9.63-9.70	Shale		9.0 ^d	207.0
BH3	9.63-9.70	Shale		11.2 ^a	257.6
BH3	9.63-9.70	Shale		3.1 ^a	71.3
BH3	11.71-11.80	Shale		10.0 ^d	230.0
BH3	11.71-11.80	Shale		2.9 ^a	66.7
BH3	11.71-11.80	Shale		2.9 ^a	66.7
BH4	11.60-11.65	Shale		10.2 ^d	234.6
BH4	11.60-11.65	Shale		9.4 ^a	216.2
AVERAGE			359.0	6.8	156.7
MAX			485.7	11.5	264.5
MIN			118.2	0.6	13.8

a-axial d-diametral

The UCS results indicated very hard rock (shale indurated due to contact metamorphism) but the samples tested were typically larger and more intact than the

average core recovered.

The axial PLT results, once converted to equivalent UCS values (by multiplying by a factor of 23 – ISRM, 1984), indicated a range of values from soft to very hard rock.

5. Geotechnical assessment

5.1 General site suitability

The site for the proposed bridge was generally deemed suitable but the presence of wide alluvial banks on either side indicated possible flooding or scouring of the approach roads, depending on the expected flood levels.

5.2 Bridge founding conditions

With a depth of approximately 7.5-10m to a solid substrate and a shallow water table, the drilling investigations indicated that the most suitable foundation system was a piled foundation socketed into the bedrock. In terms of bearing capacity assessment, rock failure generally occurs along joints or discontinuities such as bedding planes. Where joints and other planes of weakness exist in the rock mass, the practice that is normally followed is to classify the rock mass according to RQD (Rock Quality Designation – refer to Table 3).

Table 3: Relation of RQD and insitu rock quality (Peck et al., 1974)

RQD %	Rock Quality
90-100	Excellent
75-90	Good
50-75	Fair
25-50	Poor
0-25	Very Poor

An assessment of the RQD can therefore provide an indication of the allowable bearing capacity of the rock – see Table 4.

Table 4: Allowable bearing pressure (q_a) on jointed rock (after Peck et al., 1974)

RQD	q_a Ton/ft ²	q_a MPa
100	300	29
90	200	19
75	120	12
50	65	6.25
25	30	3
0	10	0.96

The minimum RQD recorded in boreholes was 0, which according to Table 4 equates to an allowable bearing pressure of 0.96MPa.

Another practice that is commonly followed is to base the allowable pressure on the unconfined compressive strength, q_u , of the rock obtained in a laboratory test. A factor of safety of 5 to 8 is applied to q_u to obtain q_a . This method is satisfactory so long as the

rocks in situ do not possess extensive cracks and joints or there are global instability issues such as potential sliding stability problems. In such cases a higher factor of safety may have to be adopted (Goodman, 1989). Using the minimum UCS values obtained in the tests (including calculated UCS from PLT tests), and using a FoS of 8, a safe vertical bearing pressure of 1.7MPa was then calculated.

Table 5 provides presumed allowable bearing pressures on rocks recommended by various building codes in the USA. From this table, it is evident that bearing pressures on bedded sedimentary rocks are generally limited to the range of 1-2.5MPa and soft or broken bedrock is 1MPa (LA codes are lower due to seismic risk). This lower-bound estimate appeared to be applicable to the site under investigation.

Table 5: Presumptive allowable bearing pressures on rocks (MPa) as recommended by various building codes in the USA (after Peck et al. , 1974)

Rock type	Building Codes			
	BOCA (1968)	National (1967)	Uniform (1964)	LOS Angeles (1959)
1. Massive crystalline bedrock, including granite diorite, gneiss, basalt, hard limestone and dolomite	10	10	$0.2 q_u^*$	1.0
2. Foliated rocks such as schist or slate in sound condition	4	4	$0.2 q_u$	0.4
3. Bedded limestone in sound condition sedimentary rocks including hard shales and sandstones	2.5	1.5	$0.2 q_u$	0.3
4. Soft or broken bedrock (excluding shale) and soft limestone	1.0	–	$0.2 q_u$	–
5. Soft shale	0.4	–	$0.2 q_u$	–

* q_u = unconfined compressive strength.

5.3 Excavations

Based on the borehole data, excavations to a depth of at least 7.5m would be classified “soft” according to SABS1200D, and this could be safely extrapolated for the entire site with a very low risk of shallow hard rock occurring within that depth range. Sidewalls of excavations were likely to be highly unstable at angles exceeding 35°.

6. Recommendations

The design of foundations is the structural engineer’s responsibility. The following recommendations are based on limited information gained from the site investigation and although the confidence in the information is high, some variations can occur between information points. All geotechnical information must be confirmed during the construction process and any significant variations are to be brought to the attention of the authors for comment or further recommendations. It is recommended that the structural engineer discuss his/her conceptual design with the geotechnical specialist to ensure that any calculations and recommendations are in line with current thinking.

6.1 Earthworks and access roads

It was recommended that the positioning of the bridge and approach roads take into account the existing access roads and foot-paths in the area, as well as the flood lines and local contours.

Earthworks and access roads should be designed and conducted in accordance with SABS 1200D, COLTO or any site-specific specifications provided by the engineer.

For site clearance, it was recommended that all vegetation and the upper 150mm portion of the soil profile (topsoil) be cut to spoil from the footprint of the bridge abutments and the approaches. Soil obtained from excavations is likely to be suitable for re-use as general backfilling (presumed to be at least G6-G7 quality) material around foundations, behind retaining walls/abutments and in road subgrade fills, but large boulders should be screened out where possible to facilitate compaction. All potential fill material obtained from site should be stockpiled for approval by the engineer before being used. It is however, recommended that allowance is made for some imported higher-quality fill material of G5 quality for shortfalls in general fill material and road surfacing (this can possibly be sourced locally from nearby borrow pits).

For deep temporary excavations exceeding 1m, trench sidewalls should be battered to a minimum safe angle of 35°. Erosion control measures, such as silt fences, etc. are highly recommended when working on or near the river embankments. Excavations below a depth of ~1m on the river banks will most likely encounter a strong groundwater table associated with the river and will require lateral support and dewatering systems to maintain an open workspace.

Approach roads will probably consist of gravel surfaced foot-paths, requiring G5 or G6 gravel wearing course layerworks, compacted to 95%MDD.

6.2 Foundations

The recommendation for the bridge foundations was permanently cased, rotary bored, reinforced concrete ODEX piles, socketed into the underlying bedrock. ODEX piling is a percussion drilling system which uses an eccentric drilling system to advance a steel casing through the overburden soil and also allows hammering of rock to form the required socket (See **Figure 11**). The steel casing being necessary to isolate the concrete from the groundwater. A minimum 1.5m rock socket and minimum pile length of 10m was estimated for preliminary costing purposes but should be checked by the pile designer against the calculated moments and shear forces.

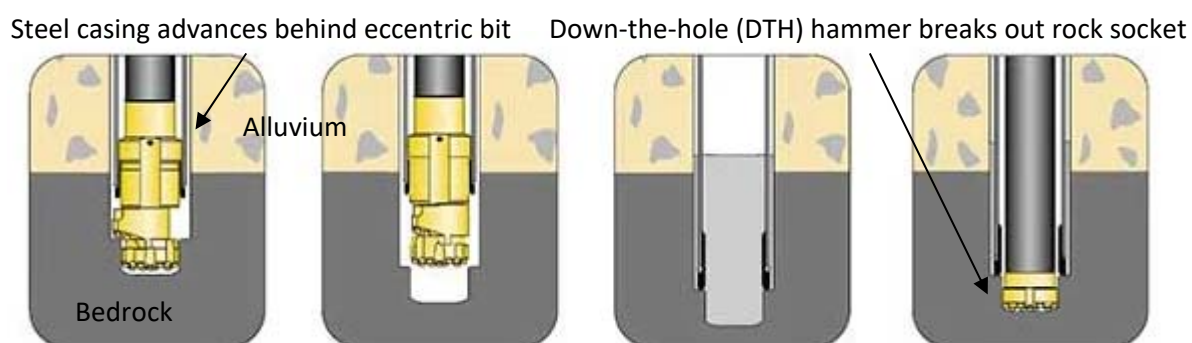


Figure 11: Odex piling system (www.prdrigs.com/odex-drilling)

Due to access restrictions to the western side, consideration should be given to the size of the rig (and therefore the diameter of the piles) as the rig may have to be floated across the river on a barge or across a temporary jetty (A possible alternative if the river is shallow enough at the time of construction, is that the rig be driven across the river). In such cases, pile diameters in the order of 200-300mm should be considered. A Casagrande C6 rig would be appropriate for such applications, but again this would have

to be checked by the piling contractor. Access by land from the western side was restricted to a very steep, narrow foot path which would be difficult to negotiate, even with some road-building. Consideration would also have to be given to getting concrete and other materials to the site. Alternative piling systems could include reticulated micro-piles, but these have limited capacity to resist shear forces.

Regardless of which system is employed, the pile group formed below each abutment would then be tied together with a large RC pile cap to form the basal support for the abutment structure.

Alternative shallow foundation systems, such as RC spread/base footings, would likely involve deep excavations with attendant lateral stability problems, mainly due to shallow groundwater tables, requiring significant dewatering systems which may be difficult to achieve (See **Figure 12**).



Figure 12: Deep excavations, lateral support and dewatering for bridge pad foundations

7. Conclusions

The site for the proposed bridge site was generally considered suitable in terms of the geology and general topography but the local geotechnical conditions were likely to impose significant constraints on the appropriate type of foundations, i.e. piles or deep base excavations, which would have significant cost implications for the project. Some preliminary recommendations were provided for consideration by the designers, but all information should be verified on site during construction.

References

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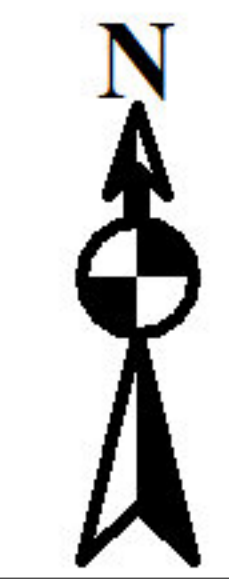
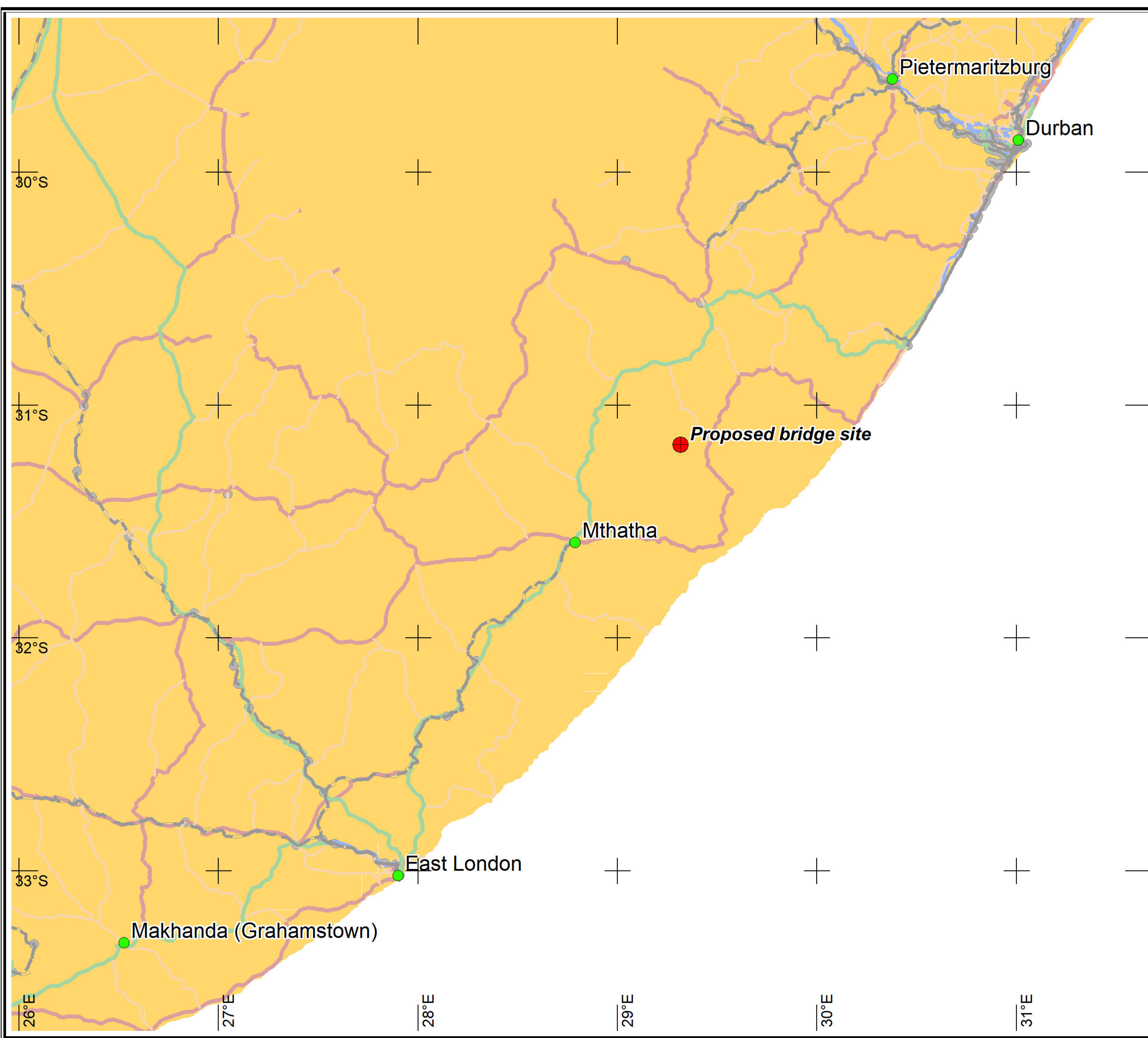
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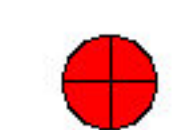
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Appendix 1

Maps



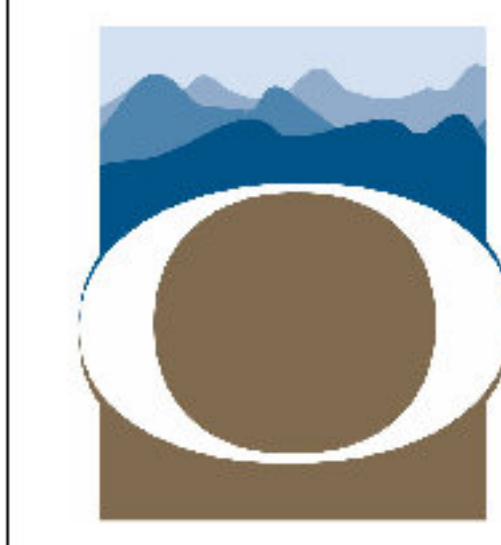
Legend

 Proposed bridge

Project: Proposed new pedestrian bridge
Site: Mzintlava River, Dikidikini village
Area: Flagstaff, E Cape

Dwg Name: Locality map
Date: November 2022
Rev no: 1
Drawn: IP

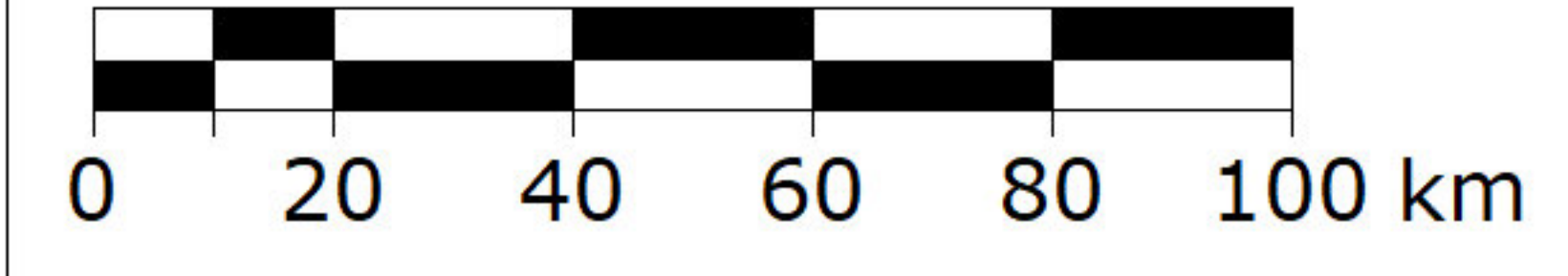
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Structural/Civil Eng: Uhambiso Consult

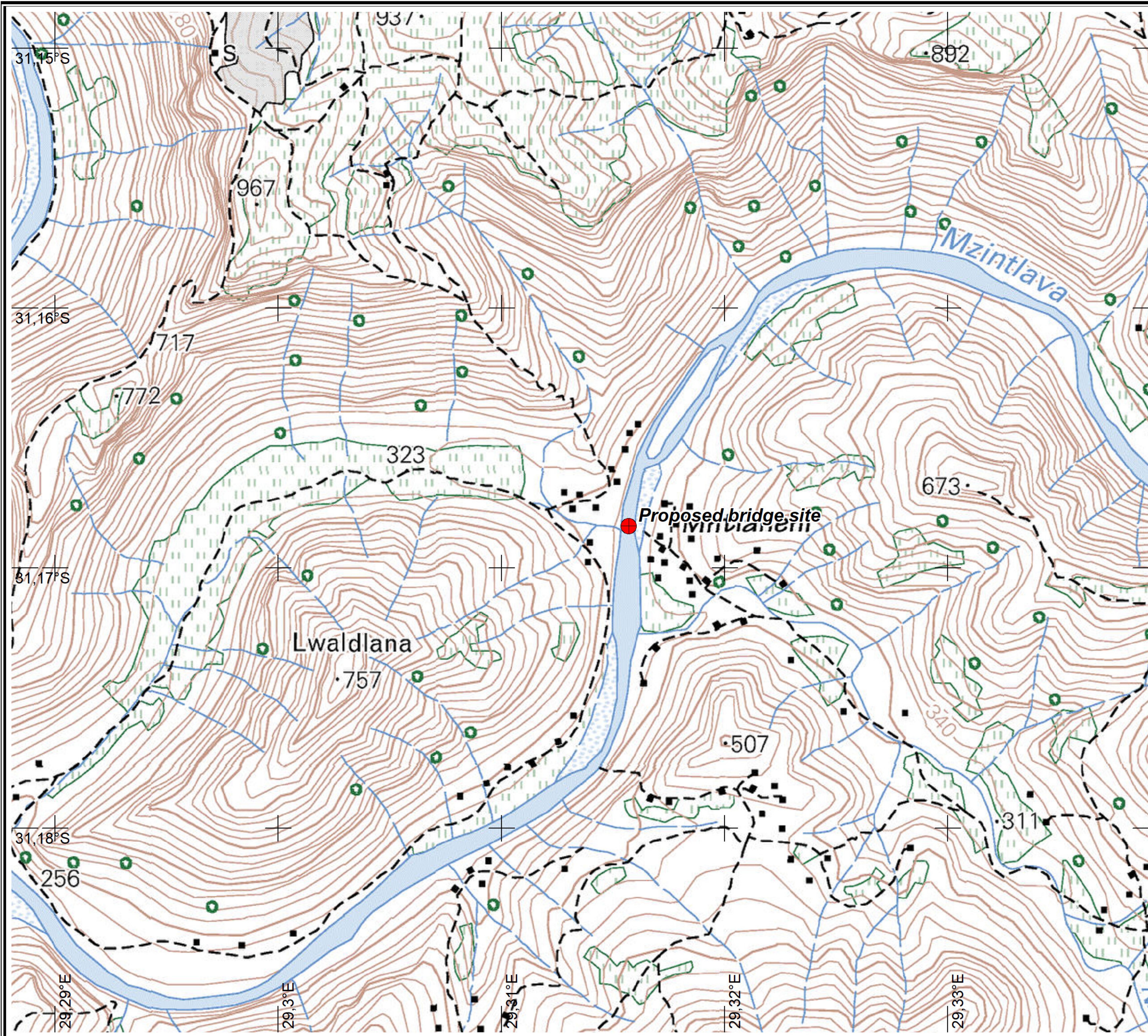


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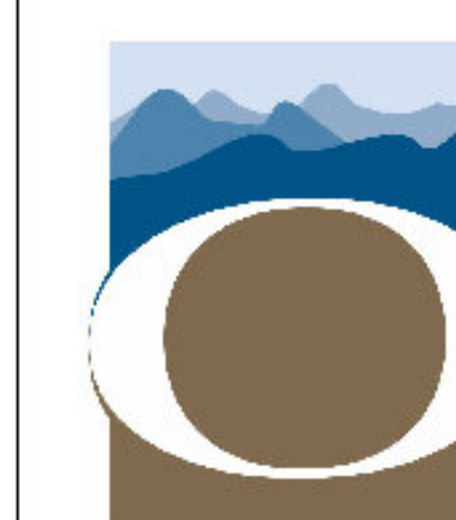
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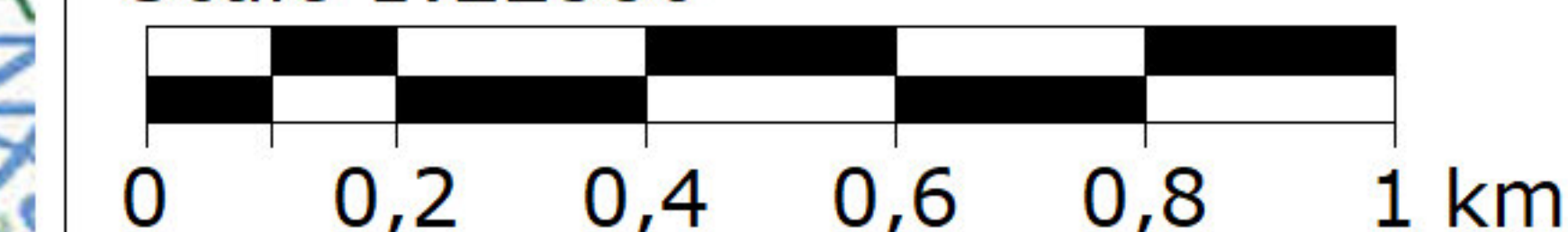
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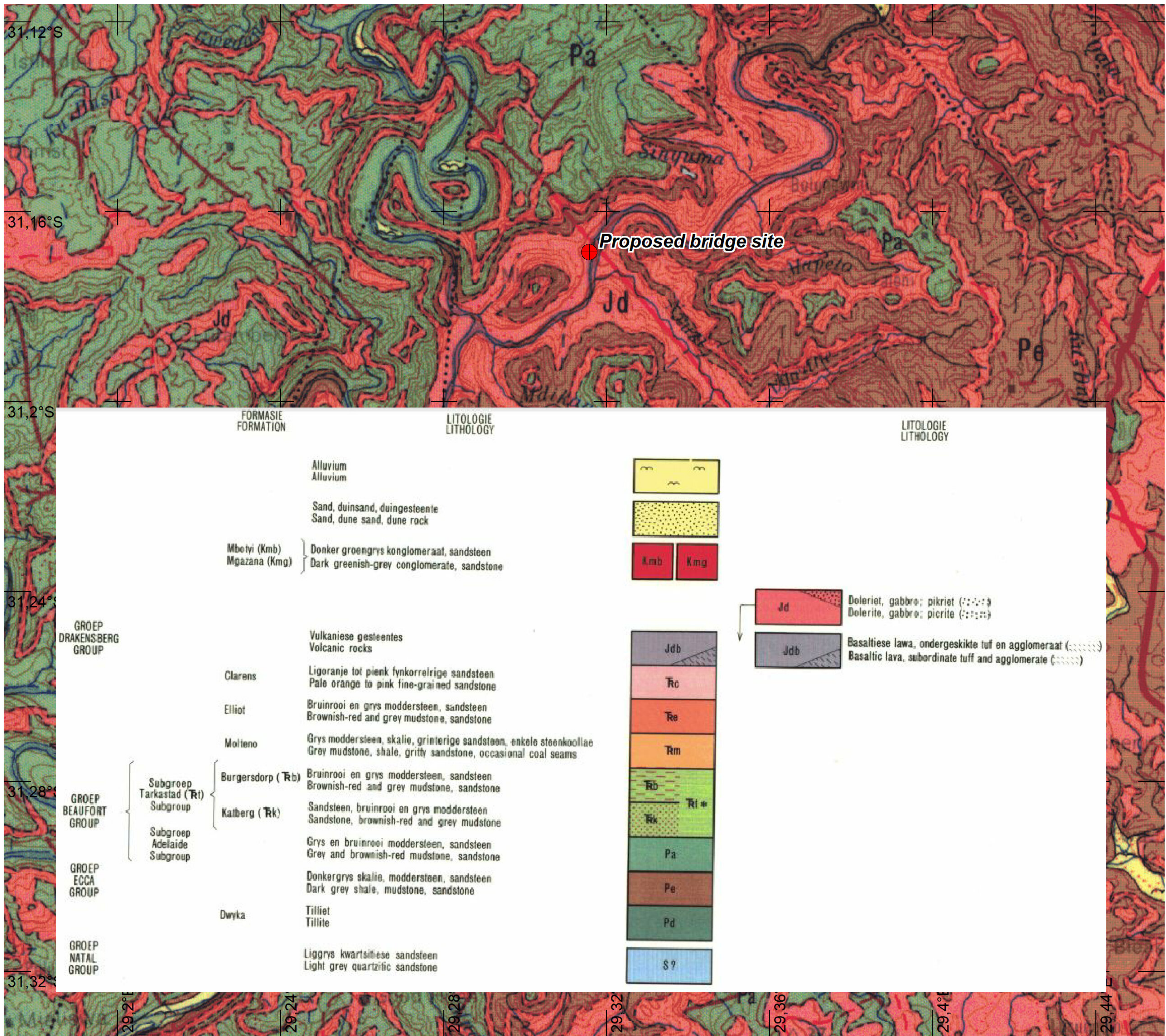


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Legend

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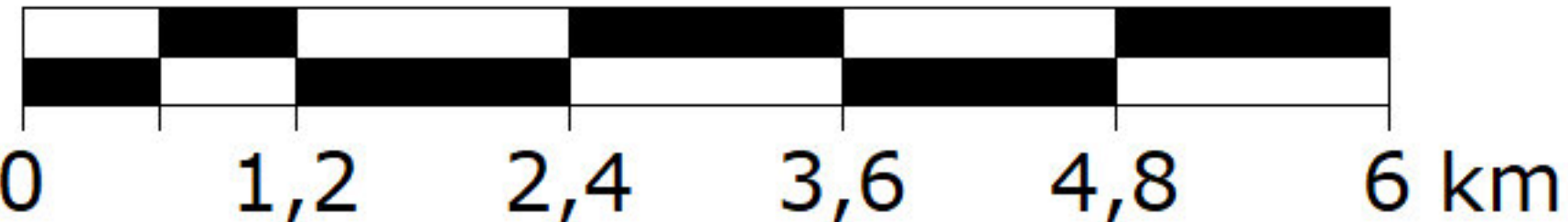
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Scale 1:125000





Legend

 Proposed bridge

Project: Proposed new pedestrian bridge
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Area: Flagstaff, E Cape

Dwg Name: Aerial photo map
Date: November 2022
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Drawn: IP

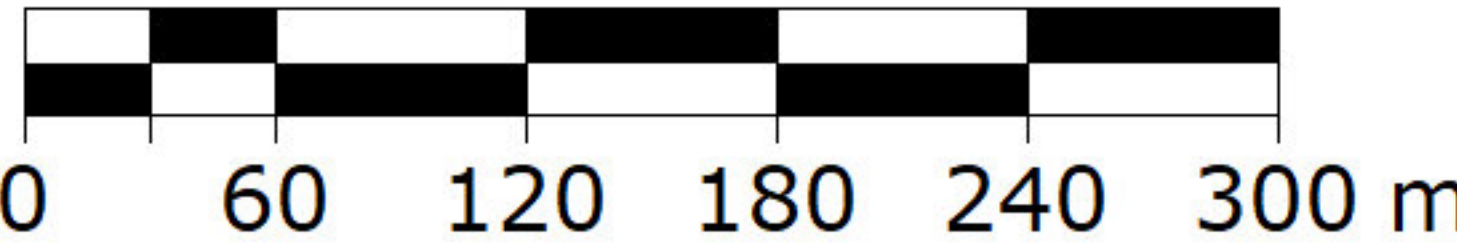
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Scale 1:7500





Legend

● Borehole positions

Project: Proposed new pedestrian bridge
Site: Mzintlava River, Dikidikini village
Area: Flagstaff, E Cape

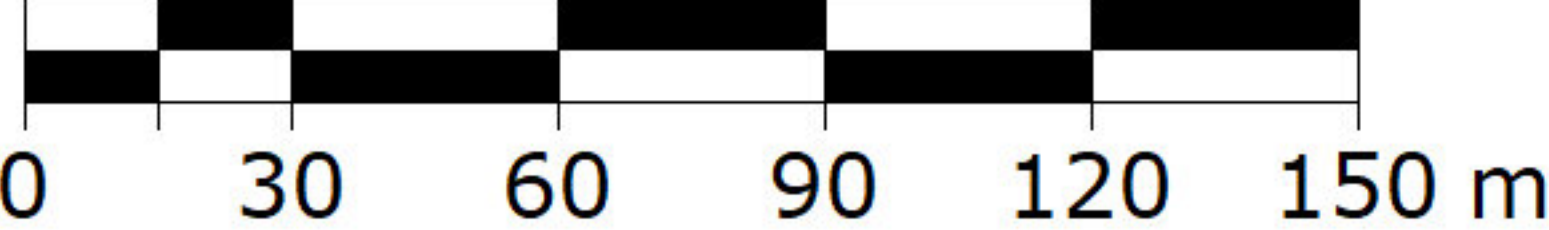
Dwg Name: Drill plan
Date: November 2022
Rev no: 1
Drawn: IP

Client: Ntabankulu LM
Structural/Civil Eng: Uhambiso Consult



18 Clyde St
Knysna
6571
South Africa
+27443820502
info@outeniqua.co.za

Scale 1:3500



Appendix 2

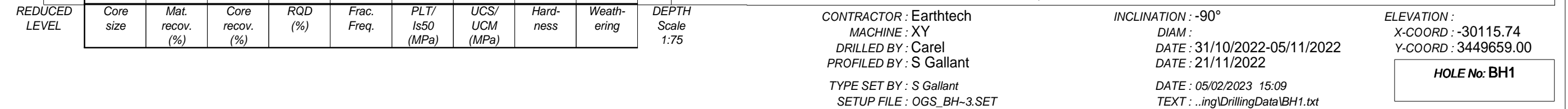
Borehole logs

JOB NUMBER: 000



OUTENIQUA
GEOTECHNICAL SERVICES

JOB NUMBER: 000





PROJECT: DIKIDIKINI BRIDGE

BOREHOLE NO: BH1

CLIENT: UHAMBISO CONSULT



PROJECT: DIKIDIKINI BRIDGE

BOREHOLE NO: BH1

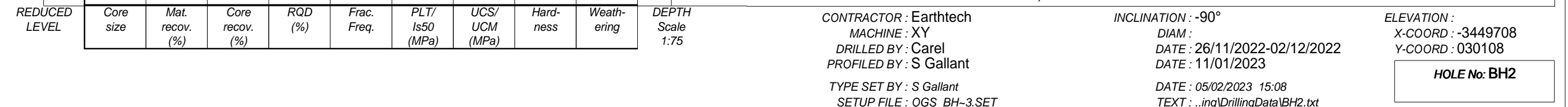
CLIENT: UHAMBISO CONSULT

JOB NUMBER: 000



OUTENIQUA
GEOTECHNICAL SERVICES

JOB NUMBER: 000





PROJECT: DIKIDIKINI BRIDGE

BOREHOLE NO: BH2

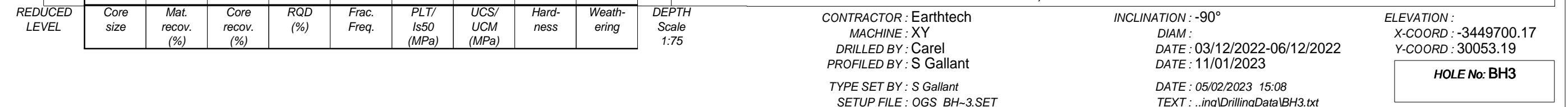
CLIENT: UHAMBISO CONSULT

JOB NUMBER: 000



OUTENIQUA
GEOTECHNICAL SERVICES

JOB NUMBER: 000





PROJECT: DIKIDIKINI BRIDGE

BOREHOLE NO: BH3

CLIENT: UHAMBISO CONSULT

HOLE No: BH4
Sheet 1 of 1

JOB NUMBER: 000

ROCK HARDNESS
VSR-Very soft rock
SR-Soft rock
MHR-Medium hard rock
HR-Hard rock
VHR-Very hard rock

WEATHERING GRAPH
100%-Completely weathered
75%-Highly weathered
50%-Moderately weathered
25%-Slightly weathered
0%-Unweathered
Hatching-Soil/Unconsolidated/NA

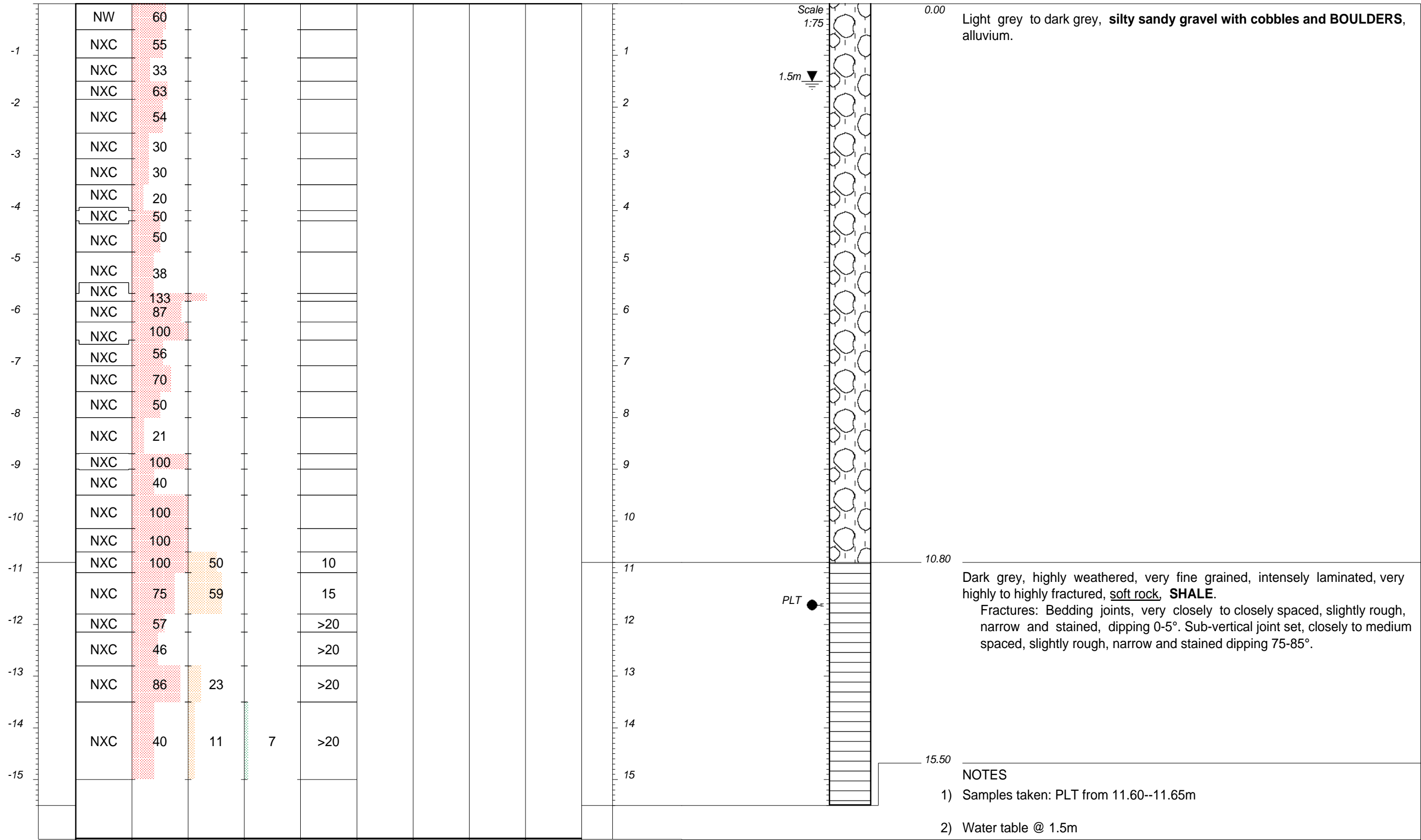


CORE BOREHOLE LOG

CLIENT: Uhambiso
SITE: Dikidikini Bridge

HOLE No: BH4
Sheet 1 of 1

JOB NUMBER: 000



REDUCED LEVEL	Core size	Mat. recov. (%)	Core recov. (%)	RQD (%)	Frac. Freq.	PLT/ Is50 (MPa)	UCS/ UCM (MPa)	Hard-ness	Weath-ering	DEPTH Scale 1:75	CONTRACTOR : Earthtech MACHINE : XY DRILLED BY : Carel PROFILED BY : S Gallant TYPE SET BY : S Gallant SETUP FILE : OGS_BH-3.SET	INCLINATION : -90° DIAM : DATE : 09/12/2022-18/12/2022 DATE : 11/01/2023 DATE : 05/02/2023 15:09 TEXT : ..ing\DrillingData\BH4.txt	ELEVATION : X-COORD : -3449682.95 Y-COORD : 30147.95
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HOLE No: BH4



PROJECT: DIKIDIKINI BRIDGE

BOREHOLE NO: BH4

CLIENT: UHAMBISO CONSULT

Appendix 3
Lab test results

Issued by:

ROCKLAB

(ROCK MECHANICS & EXCAVATION LABORATORIES)

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SOUTH AFRICA

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E-MAIL: CHENJ@ROCKLAB.CO.ZA

RESULTS OF ROCK PROPERTIES TESTS

Sampling Site: Dikidikini Bridge, Eastern Cape

BY

DR J. F. CHEN

Submitted to:

Outeniqua Geotechnical Services

9 DECEMBER 2022

C O N T E N T S

TABLE 1 - RESULTS OF UNIAXIAL COMPRESSION STRENGTH TESTS
WITH ELASTIC MODULUS MEASUREMENTS BY MEANS OF
STRAIN GAUGES

TABLE 2 - RESULTS OF SONIC VELOCITY TESTS

APPENDIX 1 DIAGRAM OF STRESS VIA STRAIN FOR UCM TESTS

APPENDIX 2 FAILURE CODES FOR ROCK COMPRESSION TEST

TABLE 1 RESULTS OF UNIAXIAL COMPRESSION TESTS WITH ELASTIC MODULUS & POISSON RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Client: Outeniqua Geotechnical Services Sampling site: Dikidikini Bridge, Eastern Cape 09-12-2022

SPECIMEN DETAILS				SPECIMEN DIMENSIONS						SPECIMEN TEST RESULTS								
Rocklab Specimen No	Borehole No	Depth		Description	Diameter	Height	Ratio of Height to diameter	Mass	Density	Failure Load	Strength (UCS)	Tangent Elastic Modulus @ 50% UCS GPa	Secant Elastic Modulus @ 50% UCS GPa	Poisson's Ratio Tangent @ 50% UCS	Poisson's Ratio Secant @ 50% UCS	Linear Axial Strain at Failure mm/mm	Failure Code	Note
		From..	To..															
9129-		m	m		mm	mm		g	g/cm³	kN	MPa							
UCM-01	BH1	8.84	9.05	Shale	60.25	128.24	2.1	977.15	2.67	1228.0	430.7	84.7	84.7	0.20	0.22	0.005091	YA	
UCM-02	BH1	12.00	12.17	Shale	60.12	130.87	2.2	1011.03	2.72	1277.0	449.8	73.5	78.2	0.25	0.25	0.006109	YA	

Note: All tests were conducted according to the ISRM's Specification. Failure codes shown in Appendix 2

TABLE 2 RESULTS OF POINT LOAD STRENGTH INDEX TESTS

Client: Outeniqua Geotechnical Services

Sampling Site: Dikidikini Bridge, Eastern Cape

06-12-2022

SPECIMEN PARTICULARS				SPECIMEN TEST RESULTS							
ROCKLAB Specimen No	Borehole No	Sample depth From.. To.. m	Description	Core Diameter D (mm)	Core Height (mm)	Failue Load P (kN)	Equivalent Core Diameter (mm)	Point Load Strength I _s (MPa)	Corrected I _{s(50)} (MPa)	Test Code	Note
9129-											
PLT-01	BH1	9.42 - 9.51	Dolerite	60.32		28.3	60.32	7.8	8.5	1	A
				60.32	27.62	8.4	46.06	4.0	3.8	2	A
PLT-02	BH1	11.57 - 11.66	Shale	60.37		21.4	60.37	5.9	6.4	1	A
PLT-03	BH1	11.90 - 11.97	Shale	60.15		2.9	60.15	0.8	0.9	1	A
				60.15	30.68	8.2	48.47	3.5	3.4	2	A
				60.15	32.73	22.7	50.07	9.1	9.1	2	A
PLT-04	BH1	13.92 - 14.01	Shale	59.83		2.0	59.83	0.6	0.6	1	A

Note: the tests were conducted according to the ISRM suggested method.

A - specimen was failed on existing crack

Test code: 1 - Diametrial loading, 2 - Axial loading

B - specimen was failed on intact rock

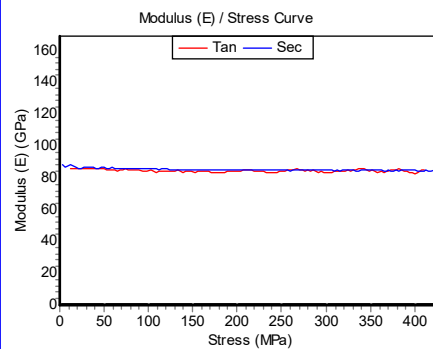
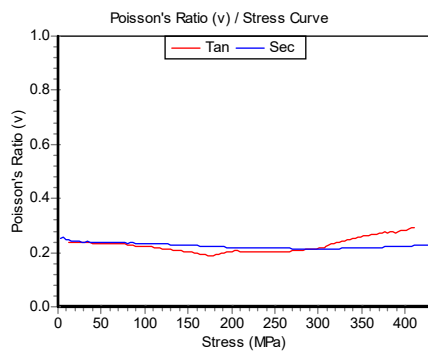
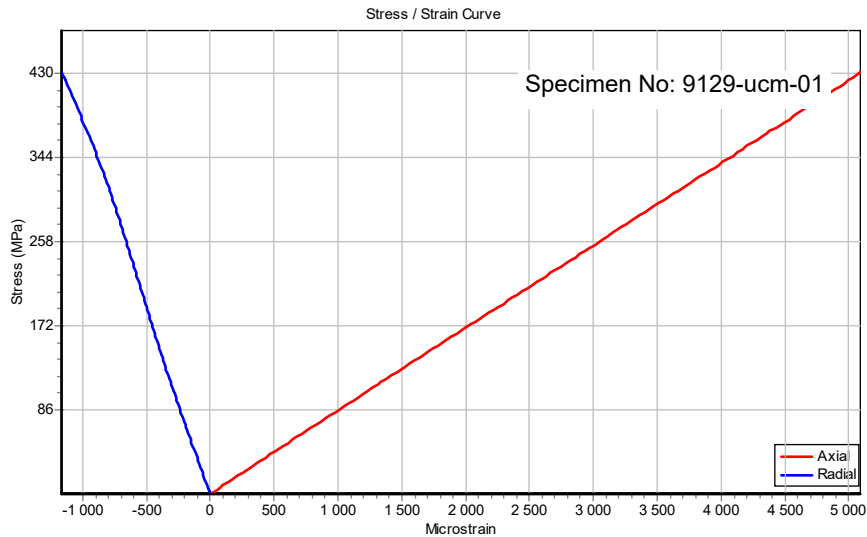
APPENDIX 1

STRESS VIA STRAIN CURVES FOR UCM TESTS

UNIAXIAL COMPRESSION TEST

2022/12/09 09:52:08

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Failure Load: 1228 kN

Peak Strength: 430.7 MPa

Axial Strain at Failure: 5091 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	ν Tan	ν Sec
10	43.1	86.2	85.8	0.233	0.238
20	86.1	85	85.8	0.226	0.236
30	129	84.3	85.3	0.212	0.230
40	172	83.4	85	0.190	0.224
50	215	84.7	84.7	0.203	0.218
60	258	85	84.5	0.205	0.216
70	301	83.2	84.6	0.219	0.215
80	345	84.8	84.5	0.256	0.217
90	388	84	84.5	0.274	0.223

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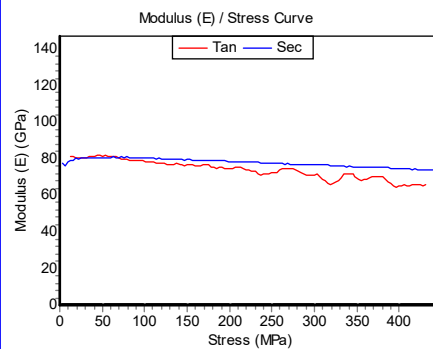
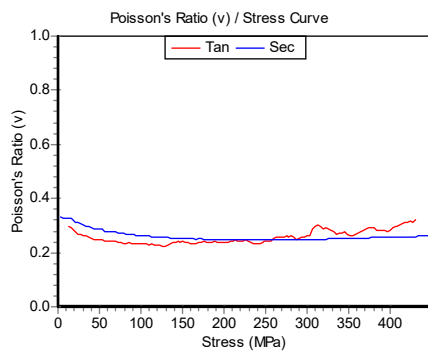
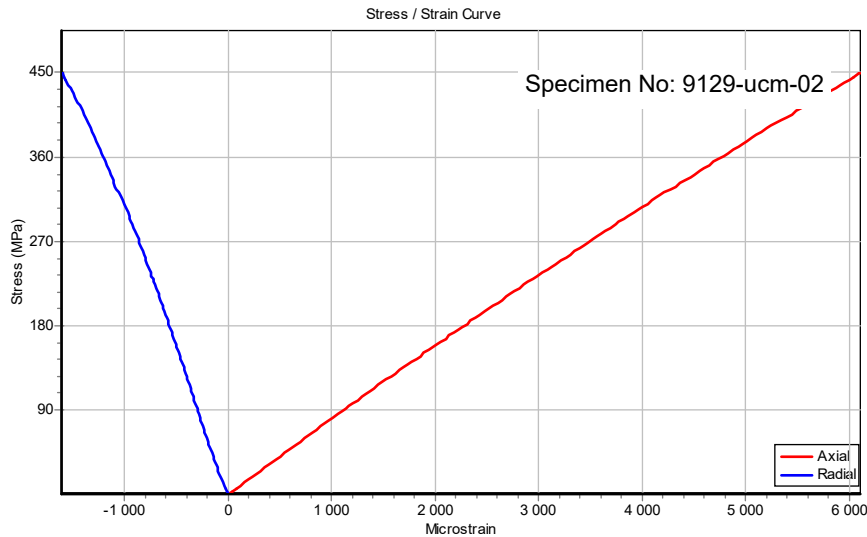
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email: chenj@rocklab.co.za

UNIAXIAL COMPRESSION TEST

2022/12/09 09:52:32

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Failure Load: 1277 kN

Peak Strength: 449.8 MPa

Axial Strain at Failure: 6109 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	ν Tan	ν Sec
10	45	81.7	80.5	0.248	0.289
20	90	79	80.4	0.235	0.265
30	135	77.5	79.7	0.239	0.253
40	180	75.3	78.7	0.237	0.249
50	225	73.5	78.2	0.246	0.249
60	270	74.4	77.1	0.259	0.246
70	315	66.8	76.5	0.297	0.249
80	360	68.8	75.4	0.272	0.253
90	405	65.6	74.5	0.296	0.257

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APPENDIX 2

CLASSIFICATION OF ROCK SPECIMEN FAILURE MODE INFLUENCED / NOT INFLUENCED BY DISCONTINUITIES DURING COMPRESSION TESTING

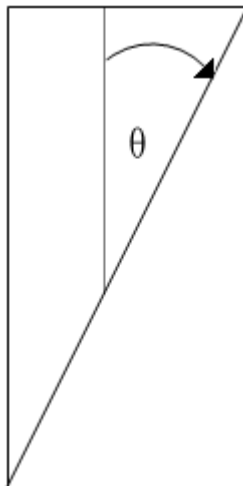
FAILURE NOT INFLUENCED BY DISCONTINUITIES (INTACT)

TYPE CODE	DESCRIPTION OF SUB CODES	
	A	B
X	SINGLE SLIDING SHEAR FAILURE	COMPLETE CONE DEVELOPMENT
Y	SPLITTING	

FAILURE INFLUENCED BY DISCONTINUITIES

TYPE CODE	DESCRIPTION OF SUB CODES	
	A	B
	PARTIAL FAILURE ON DISCONTINUITY	FAILURE COMPLETELY ON DISCONTINUITY
1	AT 0-10° TO AXIS	AT 0-10° TO AXIS
2	AT 11-20° TO AXIS	AT 11-20° TO AXIS
3	AT 21-30° TO AXIS	AT 21-30° TO AXIS
4	AT 31-40° TO AXIS	AT 31-40° TO AXIS
5	AT 41-50° TO AXIS	AT 41-50° TO AXIS
6	AT 51-70° TO AXIS	AT 51-70° TO AXIS
7	AT 71-90° TO AXIS	AT 71-90° TO AXIS
0	Multiple Discontinuities	Multiple Discontinuities

Example: Failure Type3B: Failure completely on a discontinuity with an orientation of between 21° and 30° to the specimen axis.



Issued by:

ROCKLAB

(ROCK MECHANICS & EXCAVATION LABORATORIES)

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E-MAIL: CHENJ@ROCKLAB.CO.ZA

RESULTS OF ROCK PROPERTIES TESTS

Sampling Site: Dikidikini Bridge, Eastern Cape

BY

DR J. F. CHEN

Submitted to:

Outeniqua Geotechnical Services

2 FEBRUARY 2023

C O N T E N T S

TABLE 1 - RESULTS OF UNIAXIAL COMPRESSION STRENGTH TESTS
WITH ELASTIC MODULUS MEASUREMENTS BY MEANS OF
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TABLE 1 RESULTS OF UNIAXIAL COMPRESSION TESTS WITH ELASTIC MODULUS & POISSON RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES

Client: Outeniqua Geotechnical Services

Sampling site: Dikidikini Bridge

02-02-2023

SPECIMEN DETAILS					SPECIMEN DIMENSIONS						SPECIMEN TEST RESULTS								
Rocklab Specimen No	Borehole ID	Sample No	Depth		Rock Type	Diameter mm	Height mm	Ratio of Height to diameter	Mass g	Density g/cm³	Failure Load kN	Strength (UCS) MPa	Tangent Elastic Modulus @ 50% UCS GPa	Secant Elastic Modulus @ 50% UCS GPa	Poisson's Ratio Tangent @ 50% UCS	Poisson's Ratio Secant @ 50% UCS	Linear Axial Strain at Failure mm/mm	Failure Code	Note
			From.. m	To.. m															
9152-																			
UCM-01	BH2	BH2A	10.00	10.12	Shale	54.48	94.64	1.7	606.03	2.75	471.3	202.2	70.1	73.1	0.23	0.24	0.002819	2B	
UCM-04	BH2	BH2D	10.68	10.86	Shale	54.50	152.00	2.8	953.00	2.69	275.8	118.2	73.2	73.5	0.24	0.24	0.001605	2B	
UCM-09	BH2	BH3C	8.60	8.77	Shale	54.52	114.98	2.1	730.19	2.72	1091.0	467.3	71.4	76.2	0.28	0.25	0.006667	YA	
UCM-11	BH2	BH3F	10.80	11.00	Shale	54.52	147.55	2.7	941.65	2.73	1134.0	485.7	71.5	74.5	0.29	0.25	0.007045	YA	

Note: All tests were conducted according to the ISRM's Specification.

Failure codes shown in Appendix 2

TABLE 2 RESULTS OF POINT LOAD STRENGTH INDEX TESTS

Client: Quteniqua Geotechnical Services

Sampling Site: Dikidikini Bridge, Eastern Cape

26-01-2023

SPECIMEN PARTICULARS					SPECIMEN TEST RESULTS									
ROCKLAB Specimen No	Borehole ID	Sample No	Depth		Rock Type	Core Diameter D (mm)	Core Height (mm)	Failue Load P (kN)	Equivalent Core Diameter (mm)	Point Load Strength I _s (MPa)	Corrected I _{S(50)} (MPa)	Test Code	Note	
			From.. m	To.. m										
9152-PLT-02	BH2	BH2C	10.45	10.50	Shale	54.64 54.64	25.76	25.2 22.2	54.64 42.33	8.4 12.4	8.8 11.5	1 2	B B	
PLT-03	BH2	BH2B	10.25	10.40	Shale	54.49	25.64	12.0	54.49	4.0	4.2	1	A	
						54.49		23.3	54.49	7.8	8.2	1	B	
						54.49		9.6	42.18	5.4	5.0	2	A	
						54.49		25.95	6.9	42.43	3.8	3.6	2	A
						54.49		28.77	19.6	44.68	9.8	9.3	2	B
PLT-05	BH2	BH2E	10.86	10.95	Shale	54.51	28.72	25.5	54.51	8.6	8.9	1	B	
						54.51		10.9	44.65	5.5	5.2	2	A	
						54.51		5.8	41.53	3.4	3.1	2	A	
PLT-06	BH2	BH2F	11.09	11.15	Shale	54.59	28.50	19.1	54.59	6.4	6.7	1	A	
						54.59		16.2	44.51	8.2	7.8	2	A	
						54.59		27.95	11.5	44.08	5.9	5.6	2	A
PLT-07	BH3	BH3A	8.10	8.15	Shale	54.45	26.41	32.2	54.45	10.8	11.3	1	B	
						54.45		22.0	42.79	12.0	11.2	2	B	
						54.45		24.10	17.4	40.88	10.4	9.5	2	A
PLT-08	BH3	BH3B	8.40	8.45	Shale	54.53	27.78	33.0	54.53	11.1	11.5	1	B	
						54.53		12.2	43.92	6.3	6.0	2	A	
						54.53		23.03	5.2	39.99	3.3	2.9	2	A
PLT-10	BH3	BH3D	9.63	9.70	Shale	54.65	27.30	25.8	54.65	8.6	9.0	1	B	
						54.65		22.7	43.58	11.9	11.2	2	B	
						54.65		29.41	6.6	45.24	3.2	3.1	2	A
PLT-12	BH3	BH3E	11.71	11.80	Shale	54.63	28.43	28.6	54.63	9.6	10.0	1	B	
						54.63		6.0	44.47	3.0	2.9	2	A	
						54.63		25.92	5.7	42.46	3.2	2.9	2	A
PLT-13	BH4	BH4A	11.60	11.65	Shale	54.65	25.31	29.4	54.65	9.8	10.2	1	B	
						54.65		17.9	41.97	10.2	9.4	2	B	

Note: the tests were conducted according to the ISRM suggested method.

A - specimen was failed on existing crack

Test code: 1 - Diametral loading, 2 - Axial loading

B - specimen was failed on intact rock

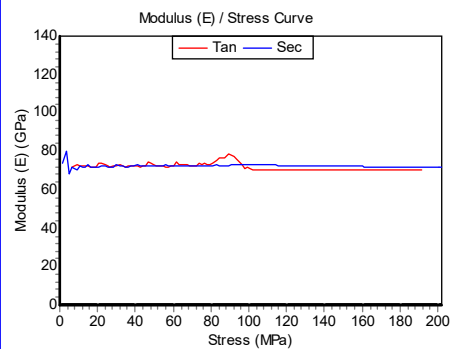
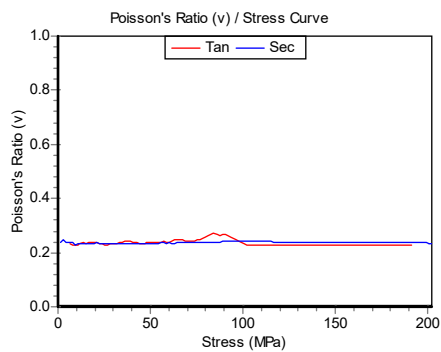
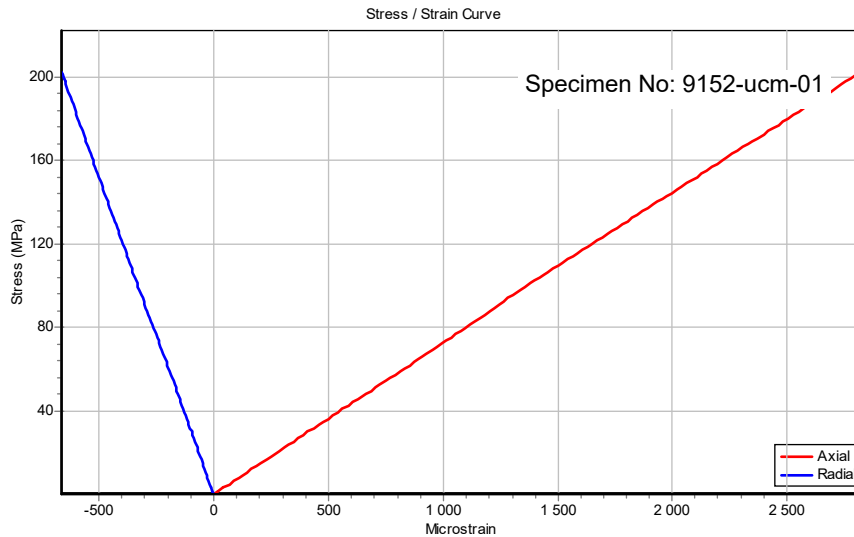
APPENDIX 1

STRESS VIA STRAIN CURVES FOR UCM TESTS

UNIAXIAL COMPRESSION TEST

2023/02/02 08:19:57

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Failure Load: 471.3 kN

Peak Strength: 202.2 MPa

Axial Strain at Failure: 2819 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	ν Tan	ν Sec
10	20.2	73.5	71.9	0.236	0.236
20	40.4	72.2	72.9	0.239	0.235
30	60.7	74.3	72.3	0.242	0.235
40	80.9	73.9	72.5	0.261	0.237
50	101	70.1	73.1	0.228	0.242
60	121	70.3	72.7	0.228	0.240
70	142	70.4	72.3	0.229	0.238
80	162	70.4	72.1	0.229	0.237
90	182	70.4	71.9	0.229	0.236

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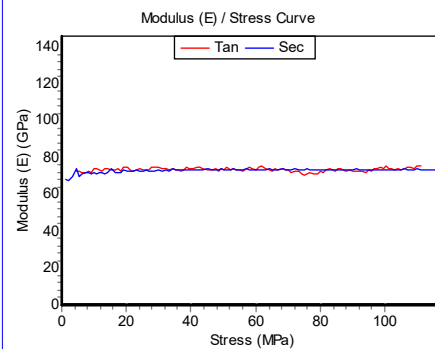
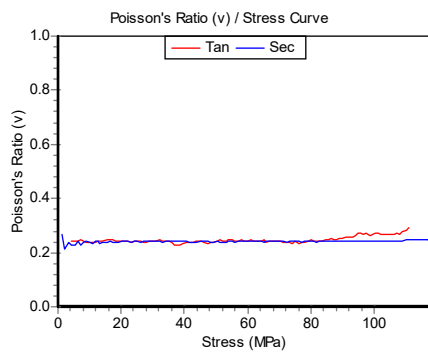
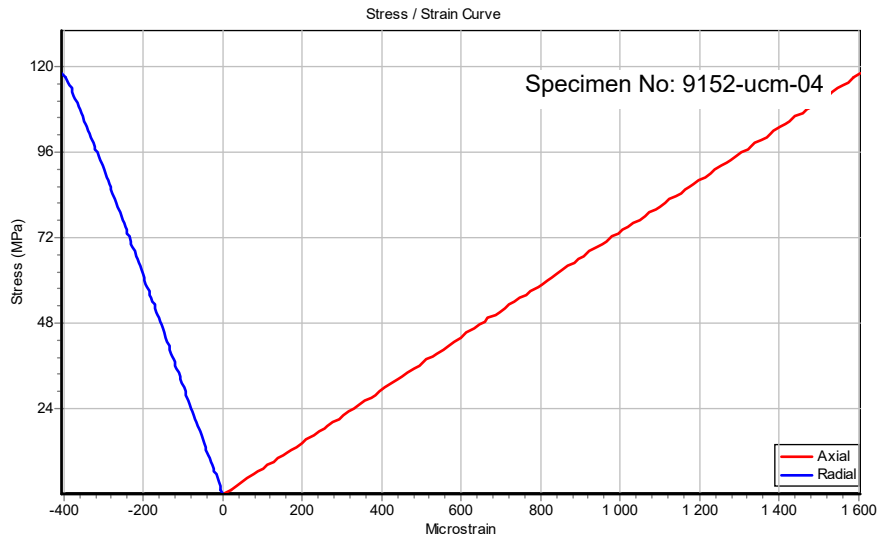
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UNIAXIAL COMPRESSION TEST

2023/02/02 08:14:51

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Failure Load: 275.8 kN

Peak Strength: 118.2 MPa

Axial Strain at Failure: 1605 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	ν Tan	ν Sec
10	11.8	72.8	72.2	0.241	0.241
20	23.6	74.4	72.9	0.241	0.242
30	35.5	73.5	73.2	0.236	0.242
40	47.3	74.1	73.4	0.234	0.241
50	59.1	73.2	73.5	0.244	0.241
60	70.9	72	73.5	0.239	0.242
70	82.7	74	73.6	0.244	0.242
80	94.6	73.3	73.6	0.271	0.242
90	106	74.5	73.3	0.272	0.245

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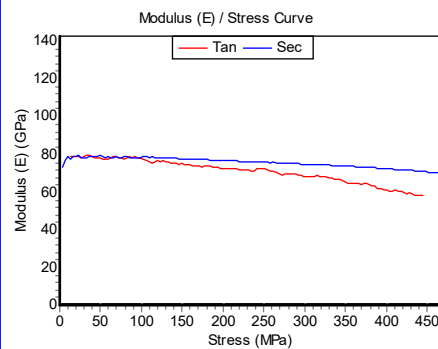
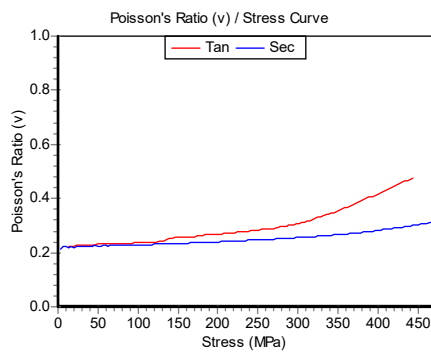
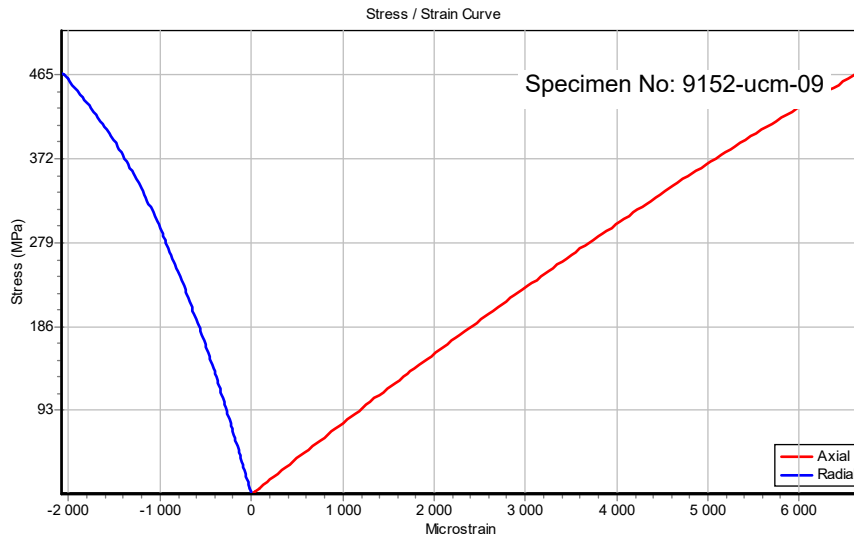
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UNIAXIAL COMPRESSION TEST

2023/02/02 08:15:30

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Failure Load: 1091 kN

Peak Strength: 467.3 MPa

Axial Strain at Failure: 6667 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	ν Tan	ν Sec
10	46.7	77.9	79.6	0.231	0.224
20	93.5	78.4	78.4	0.236	0.229
30	140	75.5	78.1	0.254	0.233
40	187	73.4	77	0.266	0.239
50	234	71.4	76.2	0.278	0.245
60	280	70	75.4	0.295	0.252
70	327	67.7	74.4	0.334	0.261
80	374	64.5	73.2	0.385	0.273
90	421	60	71.8	0.446	0.290

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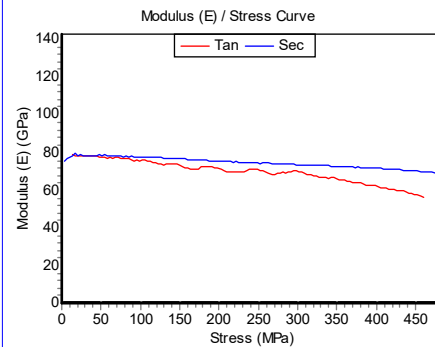
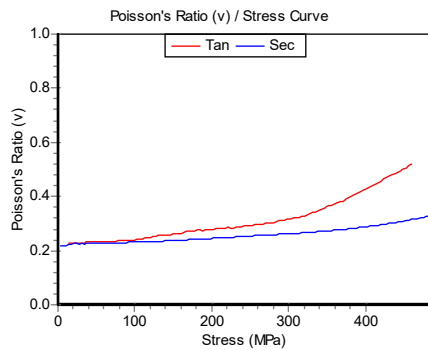
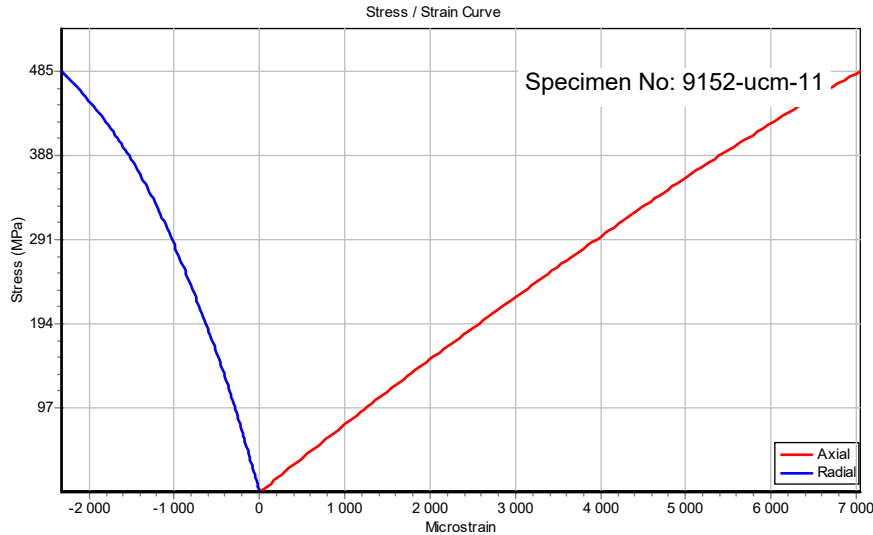
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UNIAXIAL COMPRESSION TEST

2023/02/02 08:15:47

WITH ELASTIC MODULUS AND POISSON'S RATIO MEASUREMENTS BY MEANS OF STRAIN GAUGES



Failure Load: 1134 kN

Peak Strength: 485.7 MPa

Axial Strain at Failure: 7045 microstrain

% Strength	Strength (MPa)	E Tan (GPa)	E Sec (GPa)	ν Tan	ν Sec
10	48.6	77.5	78.1	0.232	0.227
20	97.1	75.7	77.7	0.240	0.231
30	146	73.7	76.7	0.259	0.237
40	194	71.9	75.6	0.277	0.245
50	243	71.5	74.5	0.290	0.253
60	291	70.1	73.7	0.313	0.261
70	340	66.6	72.9	0.352	0.270
80	389	62.4	71.8	0.417	0.285
90	437	58.7	70.6	0.485	0.304

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APPENDIX 2

CLASSIFICATION OF ROCK SPECIMEN FAILURE MODE INFLUENCED / NOT INFLUENCED BY DISCONTINUITIES DURING COMPRESSION TESTING

FAILURE NOT INFLUENCED BY DISCONTINUITIES (INTACT)

TYPE CODE	DESCRIPTION OF SUB CODES	
	A	B
X	SINGLE SLIDING SHEAR FAILURE	COMPLETE CONE DEVELOPMENT
Y	SPLITTING	

FAILURE INFLUENCED BY DISCONTINUITIES

TYPE CODE	DESCRIPTION OF SUB CODES	
	A	B
	PARTIAL FAILURE ON DISCONTINUITY	FAILURE COMPLETELY ON DISCONTINUITY
1	AT 0-10° TO AXIS	AT 0-10° TO AXIS
2	AT 11-20° TO AXIS	AT 11-20° TO AXIS
3	AT 21-30° TO AXIS	AT 21-30° TO AXIS
4	AT 31-40° TO AXIS	AT 31-40° TO AXIS
5	AT 41-50° TO AXIS	AT 41-50° TO AXIS
6	AT 51-70° TO AXIS	AT 51-70° TO AXIS
7	AT 71-90° TO AXIS	AT 71-90° TO AXIS
0	Multiple Discontinuities	Multiple Discontinuities

Example: Failure Type3B: Failure completely on a discontinuity with an orientation of between 21° and 30° to the specimen axis.

