ETHEKWINI IRPTN

Geotechnical Investigation Report

Pinetown

PREPARED FOR:

ETHEKWINI MUNICIPALITY

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TITLE

ETHEKWINI IRPTN
Geotechnical Investigation Report

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LIST OF ABBREVIATIONS AND ACRONYMS

ARQ	ARQ Consulting Engineers
CBR	California Bearing Ratio
DCP	Dynamic Cone Penetrometer
GPS	Global Positioning System
IRPTN	Integrated Rapid Public Transport Network
MCA	MSA/Civil Concepts/ARQ
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
TP	Test Pit

1. INTRODUCTION

MSA/Civil Concepts/ARQ (MCA) Joint Venture was requested by the eThekwini Transport Municipality to carry out a field investigation for an Integrated Rapid Public Transport Network development located in Pinetown, KwaZulu-Natal. Phase 1 of the field investigation involved the excavation, profiling and sampling of test pits as well as the completion of Dynamic Cone Penetrometer (DCP) tests along the route. This was carried out by ARQ representatives, Raimondo De Simone and Willie Bronkhorst from 19 to 29 November 2013.

The development under consideration involves expansion to the current pavement layout in order to introduce dedicated lanes for the proposed Bus Rapid Transit system. Seven BRT stations are also envisaged along the route. Currently, long portions of the road are bordered by earth embankments. However, the embankment slopes will steepen with the increasing road width, necessitating retaining walls for large portions of the route.

Excavation of test pits allowed visual inspection of in situ soil conditions and sampling of various soil types. DCP testing provided a cost-effective method of determining the consistency of material underlying the proposed structures. The investigation was aimed at supplying information in terms of:

- foundation conditions and engineering properties of the materials underlying the BRT route,
- excavatability of the material on site,
- presence of groundwater, and
- provision of foundation design recommendations and precautionary measures to ensure the long-term safety and stability of the proposed structures.

2. THE BRT ROUTE

The route is approximately 6.6km long and is divided by the client into four sections, namely A1, A2, B1 and B2. A locality plan of the site, indicating the four sections is provided in the figure on the next page.

For the purpose of this report the route section will be discussed from north to south, hence from Section B2 to A1.

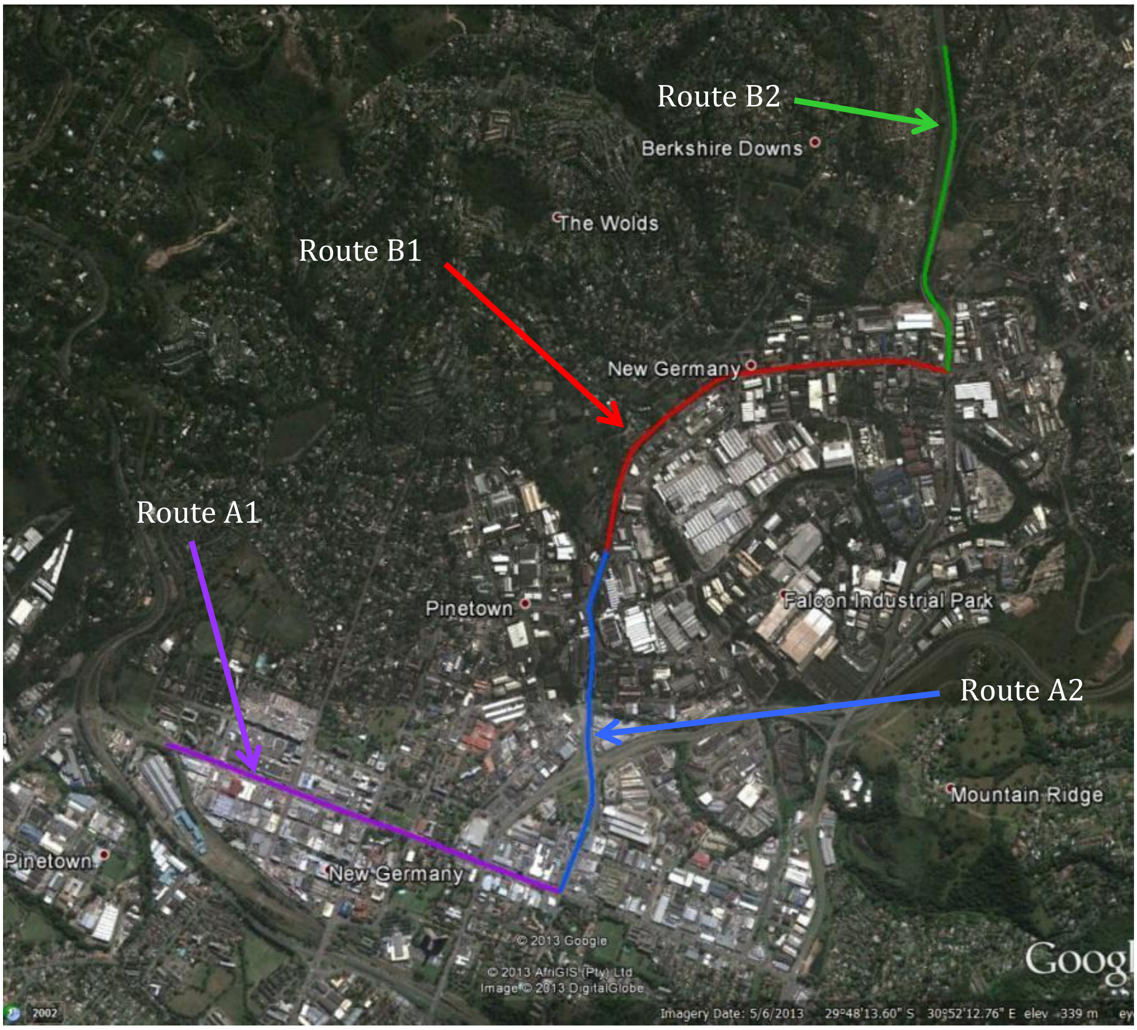


Figure 1: BRT route sections

A total of 7 stations are located along the four route sections, all of which comprise single storey buildings constructed in the median. The stations will be placed between 2 BRT lanes which will accommodate busses travelling in opposite directions. The station locations are described below.

2.1. HARMONY HEIGHTS STATION

Harmony Heights Station is located at the beginning of the IRPTN route on Dinkelman Road. A locality plan of the station is provided in Figure 2 on the next page.

Figure 2: Location plan for Harmony Heights Station.

2.2. HENDRED STATION

Hendred Station will be located on the corner of Dinkelman Road and Qashana Khuzwayo Road (formerly Shepstone Road). Please see Figure 3 for a locality plan of the station.

Figure 3: Location plan for Hendred Station.

2.3. REGENT STATION

Regent Station is located on Qashana Khuzwayo Road, between the intersections of Coventry Avenue and Regent Street. Figure 4 shows an aerial view of the proposed station, obtained from Google Earth.

Figure 4: Location plan for Regent Station.

2.4. HENWOOD STATION

Henwood Station is located in the vicinity of the Qashana Khuzwayo Road and Saint Johns Avenue intersection.

Figure 5: Location plan for Henwood Station.

2.5. BEVISS STATION

Beviss Station will be situated in the vicinity of the intersection of Qashana Khuzwayo Road and Josiah Gumede Road. A locality plan of the station is shown in the figure below.

Figure 6: Location plan for Beviss Station.

2.6. PINETOWN CIVIC STATION

Pinetown Civic Station will be located in the vicinity of the intersection of Josiah Gumede Road and Meller Road. A locality plan of the station is shown in the figure below.

Figure 7: Location plan for Pinetown Civic Station.

2.7. MOODIE STATION

The proposed Moodie Station will be situated in the vicinity of the intersection of Josiah Gumede Road and Glenugie Road. A plan of the station is provided in Figure 8 below.

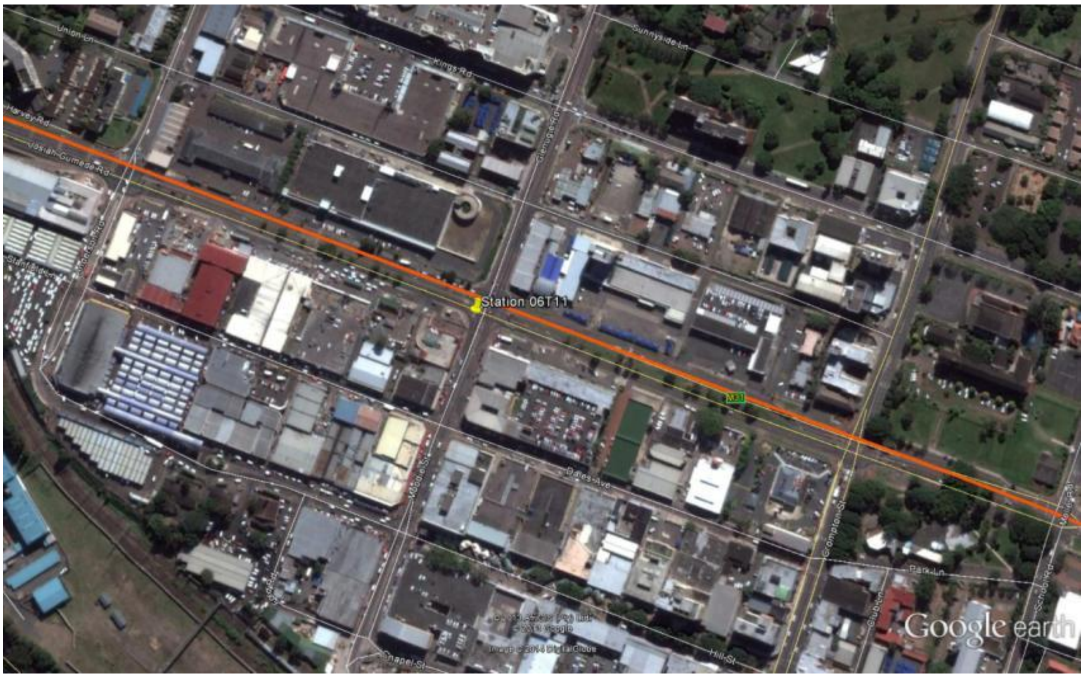


Figure 8: Location plan for Moodie Station.

3. METHOD OF INVESTIGATION

This section discusses the process that was completed during the field investigation, which comprised a test pit and DCP investigation.

A TLB was hired for the excavation, profiling and sampling of 29 test pits. The test pits were excavated to depths ranging from 1.5m to 3.0m, depending on the location of existing services in the road reserve. The test pits were profiled according to Brink and Bruin (2002).

A total of 64 DCP tests were conducted along the route, some of which were performed at the base of test pits. This provided information regarding soil conditions to approximately 1m below the test pit floor. Results from the DCP tests provide an indication of the soil stiffness and strength along the route, which is essential when assessing the founding of structures.

Test pit and DCP test locations were recorded using a hand held GPS accurate to approximately 5m. Due to existing infrastructure along the route, representative placement of test pits and DCPs proved problematic. However when difficulties were encountered, the test points were placed as close as possible to the applicable locations.

Following the conclusion of the fieldwork, disturbed and undisturbed soil samples were submitted to a laboratory for testing. See Section 5.1.

Table 1 on the next page provides the DCP and test pit locations as well as approximate chainages of the tests points (where applicable).

Table 1: Test pit and DCP Locations

Route section	Approximate Chainage	Test Pit Numbers	DCP Numbers	Coordinates	
				South	East
B2		TP B2-1	DCP B2-1	29° 47' 16.3"	30° 53' 15.6"
		TP B2-2		29° 47' 18.9"	30° 53' 15.9"
		TP B2-3	DCP B2-2	29° 47' 19.0"	30° 53' 14.7"
		TP B2-4	DCP B2-3	29° 47' 19.5"	30° 53' 16.5"
		TP B2-5		29° 47' 20.3"	30° 53' 15.6"
		TP B2-6	DCP B2-4	29° 47' 20.9"	30° 53' 15.3"
B1	6 640	TP B1-1	DCP B1-1	29° 47' 53.0"	30° 53' 13.8"
	6 600	TP B1-2	DCP B1-2	29° 47' 53.7"	30° 53' 11.9"
	6 600	TP B1-3		29° 47' 53.6"	30° 53' 11.8"
	6 280		DCP B1-3	29° 47' 52.6"	30° 53' 00.4"
	6 280		DCP B1-4	29° 47' 52.3"	30° 53' 00.3"
	6 260		DCP B1-5	29° 47' 52.2"	30° 52' 59.8"
	6 260		DCP B1-6	29° 47' 53.0"	30° 52' 59.8"
	6 230		DCP B1-7	29° 47' 52.8"	30° 52' 58.7"
	6 220		DCP B1-8	29° 47' 53.1"	30° 52' 58.1"
	6 200		DCP B1-9	29° 47' 52.6"	30° 52' 57.2"
	6 180		DCP B1-10	29° 47' 53.1"	30° 52' 56.3"
	6 040		DCP B1-11	29° 47' 53.4"	30° 52' 51.5"
	6 020	TP B1-4		29° 47' 53.5"	30° 52' 50.7"
	6 000		DCP B1-12	29° 47' 53.3"	30° 52' 50.0"
	5 960		DCP B1-13	29° 47' 53.9"	30° 52' 48.8"
	5 960		DCP B1-14	29° 47' 53.9"	30° 52' 47.5"
	5 920		DCP B1-15	29° 47' 53.9"	30° 52' 46.6"
	5 900		DCP B1-16	29° 47' 53.9"	30° 52' 45.7"
	5 880		DCP B1-17	29° 47' 53.0"	30° 52' 45.6"
	5 720		DCP B1-18	29° 47' 54.8"	30° 52' 39.6"
	5 700		DCP B1-19	29° 47' 55.0"	30° 52' 38.8"
	5 660		DCP B1-20	29° 47' 55.5"	30° 52' 37.6"
	5 620		DCP B1-21	29° 47' 56.4"	30° 52' 36.2"
	5 580		DCP B1-22	29° 47' 55.8"	30° 52' 34.2"
	5 560		DCP B1-23	29° 47' 57.0"	30° 52' 34.9"
	5 540	TP B1-5		29° 47' 57.0"	30° 52' 33.4"
	5 440		DCP B1-24	29° 47' 59.4"	30° 52' 30.7"
	5 440		DCP B1-25	29° 47' 59.1"	30° 52' 30.0"
	5 430	TP B1-6	DCP B1-26	29° 47' 59.1"	30° 52'29.4"
	5 420	TP B1-7	DCP B1-27	29° 47' 59.9"	30° 52' 29.7"
	5 360		DCP B1-28	29° 48' 00.9"	30° 52'28.9"
	5 350		DCP B1-29	29° 48' 00.9"	30° 52'29.4"
	5 340		DCP B1-30	29° 48' 01.4"	30° 52'28.8"
	5 310		DCP B1-31	29° 48' 02.2"	30° 52'27.4"

Route	Approximate Chainage	Test Pit Numbers	DCP Numbers	Coordinates	
				South	East
B1	5 310		DCP B1-32	29° 48' 01.7"	30° 52'26.5"
	5 260		DCP B1-33	29° 48' 03.1"	30° 52'26.2"
	5 140		DCP B1-34	29° 48' 06.5"	30° 52' 23.7"
	5 000		DCP B1-35	29° 48' 10.1"	30° 52' 22.3"
	4 960		DCP B1-36	29° 48' 11.8"	30° 52' 21.8"
A2	18 850	TP A2-1		29° 48' 20.9"	30° 52' 19.4"
	18 900	TP A2-2		29° 48' 22.9"	30° 52' 18.8"
	18 950		DCP A2-1	29° 48' 23.5"	30° 52' 17.3"
	18 960		DCP A2-2	29° 48' 24.1"	30° 52' 16.9"
	18 960	TP A2-3	DCP A2-3	29° 48'24.8"	30° 52' 18.1"
	18 960		DCP A2-4	29° 48' 24.8"	30° 52' 16.3"
	19 030	TP A2-4		29° 48' 26.2"	30° 52' 17.5"
	19 180		DCP A2-5	29° 48' 30.2"	30° 52'17.4"
	19 200		DCP A2-6	29° 48' 32.0"	30° 52'17.2"
	19 220	TP A2-5		29° 48' 33.6"	30° 52' 17.9"
	19 220		DCP A2-7	29° 48' 33.9"	30° 52'17.1"
	19 400	TP A2-6		29° 48' 38.2	30° 52' 17.7"
	19 420	TP A2-7		29° 48' 40.2"	30° 52' 17.9"
	19 650		DCP A2-8	29° 48' 46.3"	30° 52'17.7"
	19 650		DCP A2-9	29° 48' 46.7"	30° 52'18.6"
	19 760		DCP A2-10	29° 48' 50.1"	30° 52'16.4"
	19 770		DCP A2-11	29° 48' 50.9"	30° 52'16.6"
	19 780	TP A2-8	DCP A2-12	29° 48' 51.1"	30° 52' 18.3"
	19 920	TP A2-9		29° 48' 55.1"	30° 52' 17.3"
	19 920		DCP A2-13	29° 48' 54.7"	30° 52'16.0"
	20 100		DCP A2-14	29° 49' 02.2"	30° 52'14.0"
	20 100		DCP A2-15	29° 49'02.0"	30° 52'13.1"
	20 200		DCP A2-16	29° 49'03.7"	30° 52'12.6"
A1			DCP A1-1	29° 49' 05.4"	30° 52' 10.2"
		TP A1-1		29° 49' 04.5	30° 52' 09.7"
		TP A1-2	DCP A1-2	29° 49' 03.1"	30° 52' 05.5"
			DCP A1-3	29° 49' 00.6"	30° 51'58.3"
		TP A1-3	DCP A1-4	29° 49' 03.3"	30° 52' 05.3"
		TP A1-4	DCP A1-5	29° 49' 02.7"	30° 52' 05.4"
			DCP A1-6	29° 49' 00.6"	30° 51' 58.3"
		TP A1-5		29° 48' 56.4"	30° 51' 44.8"
		TP A1-6	DCP A1-7	29° 48' 50.4"	30° 51'28.6"
		TP A1-7	DCP A1-8	29° 48' 49.9"	30° 51'25.5"

3.1. ROUTE B2

Route B2 is orientated in a north-south direction, along Dinkelman Road. Six test pits and four DCP tests were conducted in the investigation of the route. All of these were positioned in the vicinity of the off-ramp of Dinkelman Road which bridges over the main road to connect to 11th Avenue.

Figure 9 and Figure 10 show aerial views of the off-ramp bridge and surrounding DCP and test pit locations.

Figure 9: Location of DCP tests conducted for Route B2.

Figure 10: Google Earth image of the Test pits locations completed for Route B2.

3.2. ROUTE B1

Route B1 commences at the intersection of Dinkelman Road and Qashana Khuzwayo Road. It then runs eastwards along Qashana Khuzwayo Road and terminates at the intersection of Qashana Khuzwayo Road and Broadway Street. Thirty six DCP tests and seven test pits were completed along the route. Google Earth images of the test point locations are provided in the figures on the following pages.

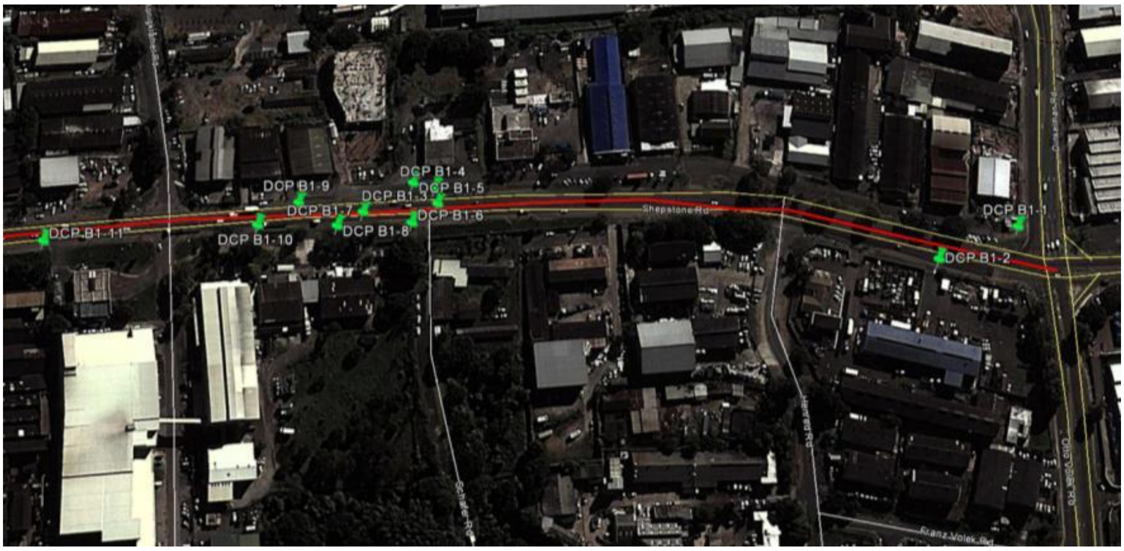


Figure 11: Locations of DCP B1-1 to DCP B1-11 situated along Route B1.

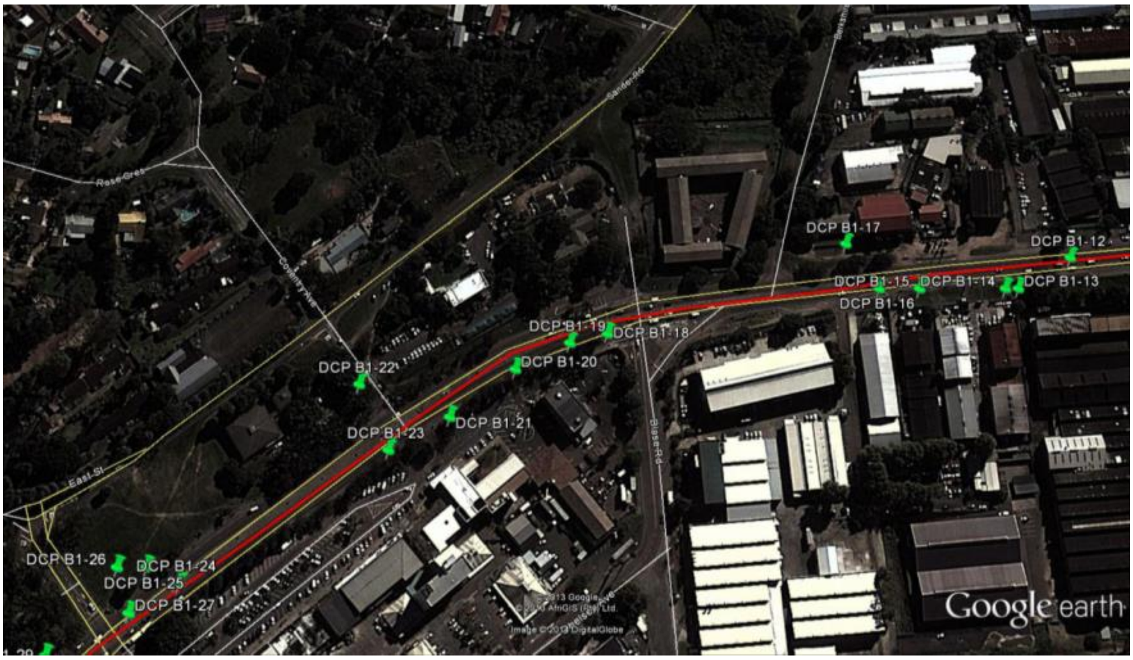


Figure 12: Locations of DCP B1-12 to DCP B1-27 situated along Route B1.

Figure 13: Locations of DCP B1-28 to DCP B1-36 situated along Route B1.

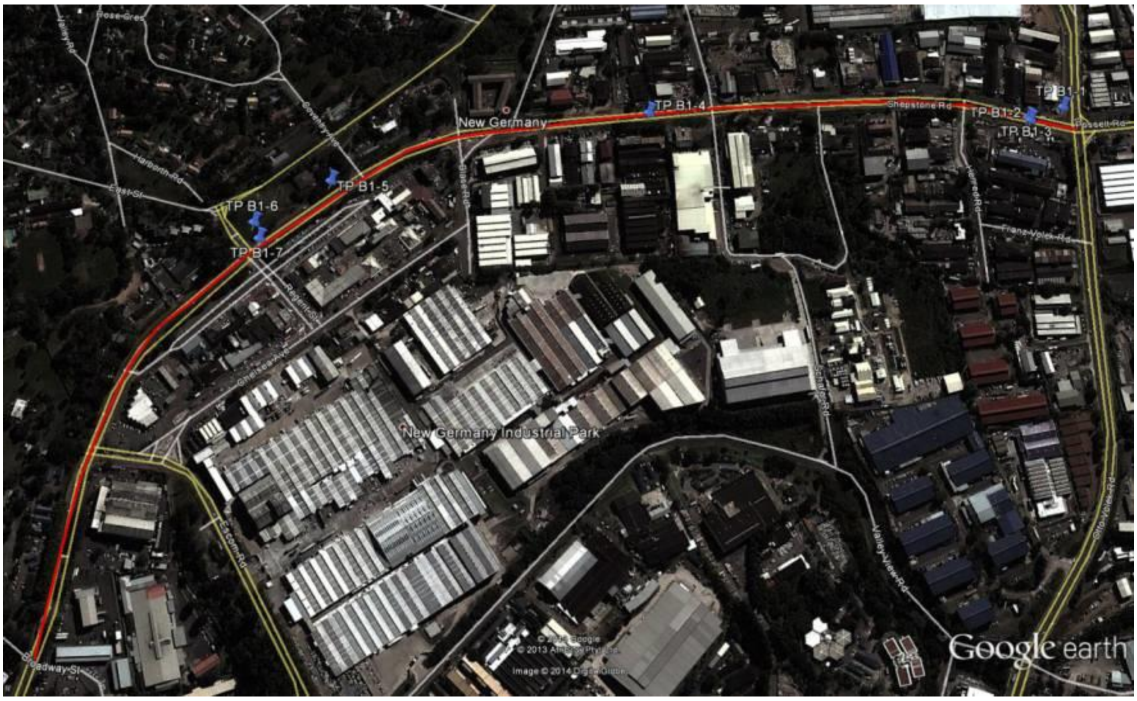


Figure 14: Locations of TP B1-1 to TP B1-7 situated along Route B1.

3.3. ROUTE A2

Route A2 commences at the intersection of Qashana Khuzwayo Road and Broadway Street, and continues along Qashana Khuzwayo Road. The route continues straight over St Johns Avenue and onto Beviss Road, terminating at the intersection of Beviss Road and Josiah Gumede Road. Sixteen DCP tests and nine tests pits were conducted along the route. The test point locations are shown in the figures below.

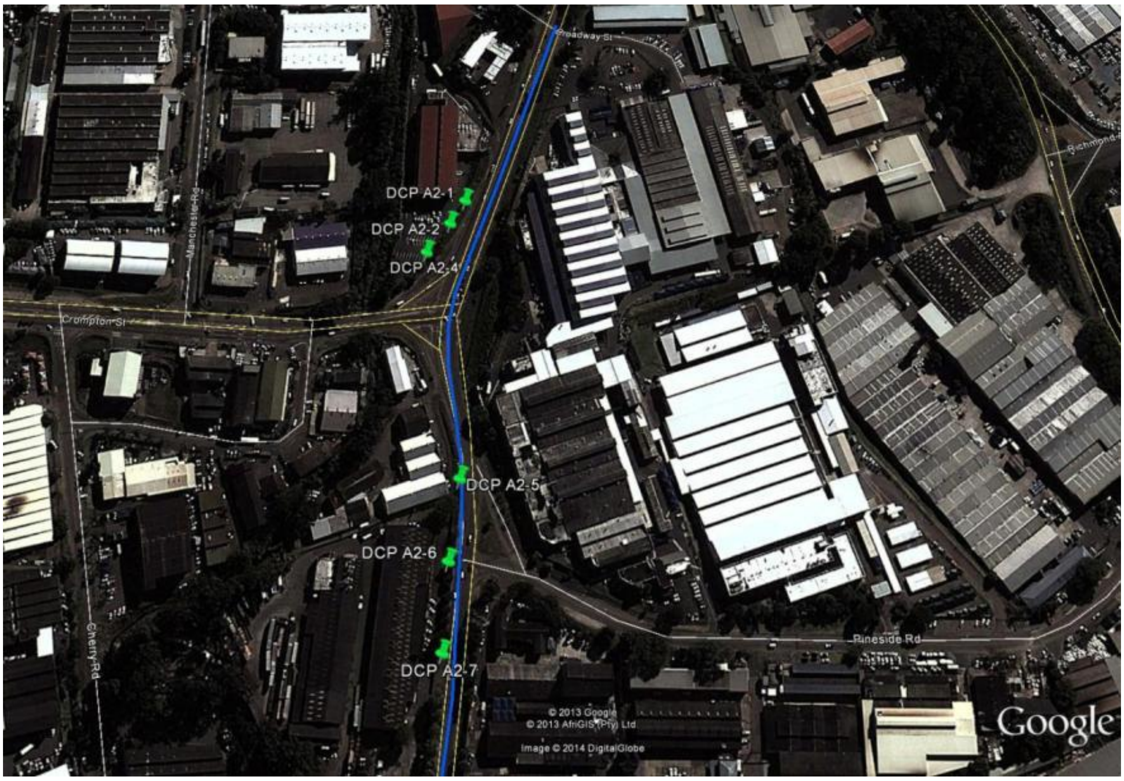


Figure 15: Locations of DCP A2-1 to DP A2-7 completed for Route A2.

Figure 16: Locations of DCP A2-8 to DCP A2-16 completed for Route A2.

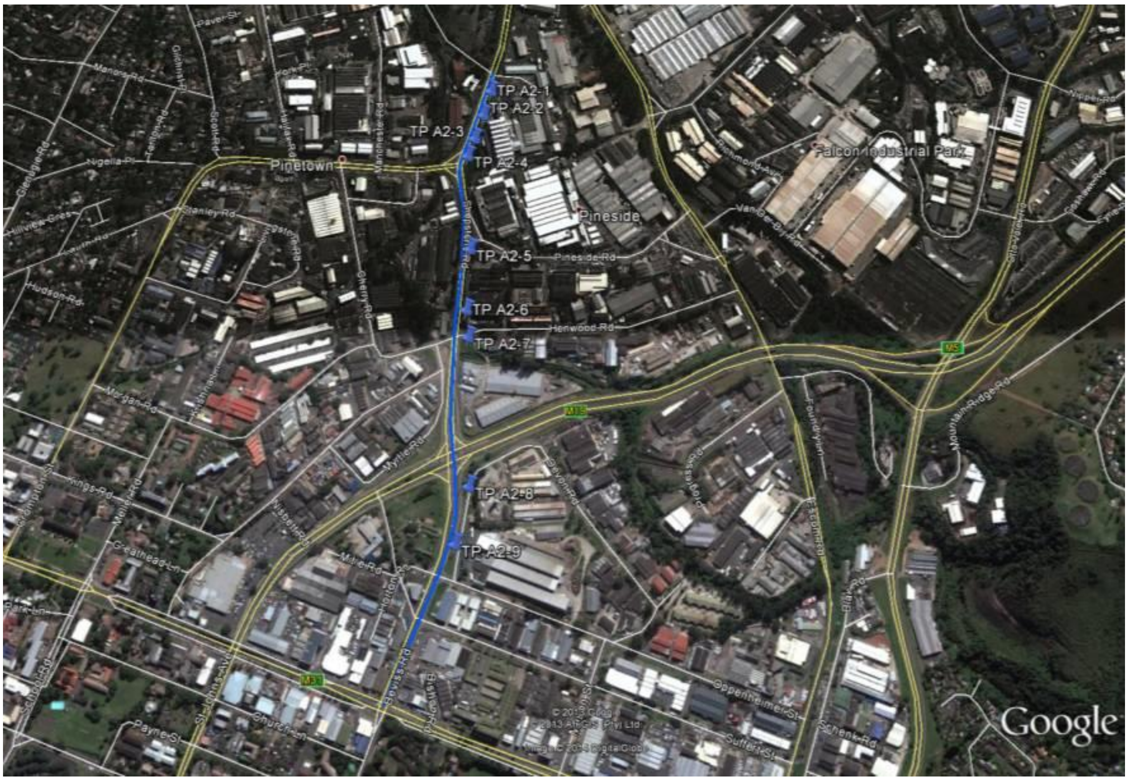


Figure 17: Locations of TP A2-1 to TP A2-9 completed for Route A2.

3.4. ROUTE A1

Route A1 commences at the Beviss Road and Josiah Gumede Road intersection and continues along Josiah Gumede Road for approximately 1.8km. Eight DCP tests and seven test pit excavations were conducted for this section. Their locations are shown in the figures on the next page.

Figure 18: Locations of DCP A1-1 to DCP A1-6 completed for Route A1.

Figure 19: Locations of DCP A1-7 to DCP A1-8 completed for Route A1.

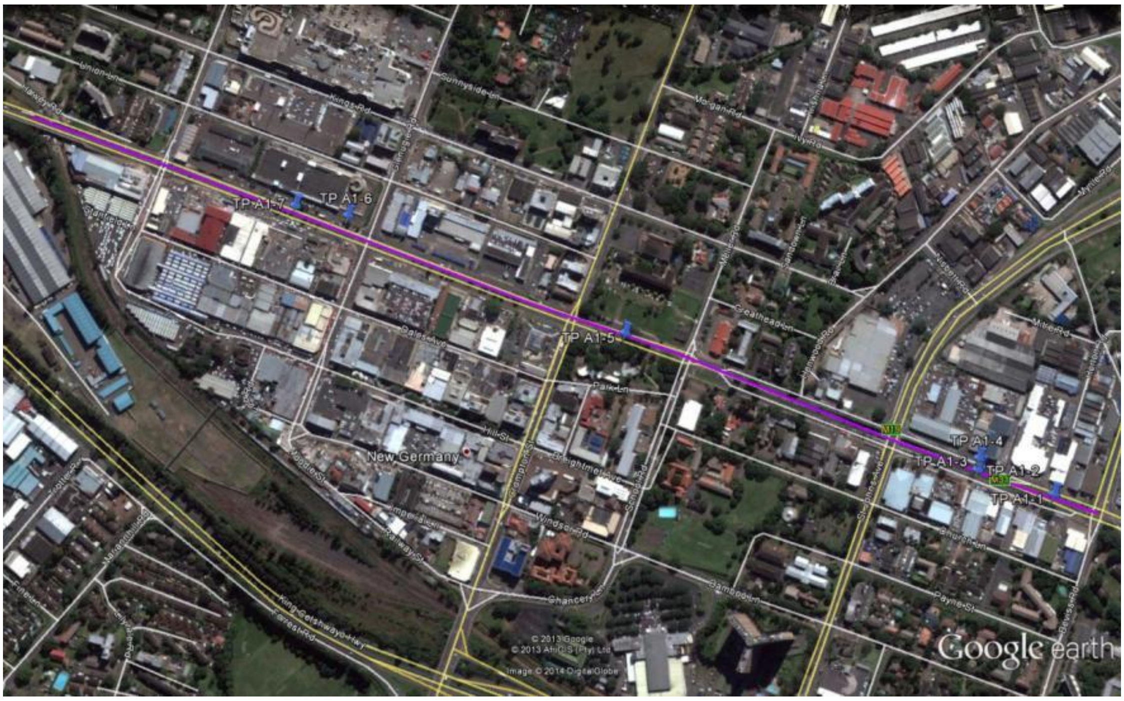
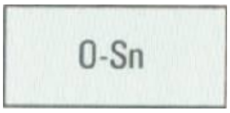


Figure 20: Locations of TP A1-1 to TP A1-7 completed for Route A1.

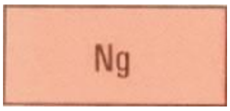
4. GEOLOGY AND TYPICAL SOIL PROFILE

The 1:250 000 geological map 2930 DURBAN was overlain in Google Earth, and using the route co-ordinates the geology underlying the route was established. Figure 21 shows the Google Earth image with the geological map and BRT route superimposed.

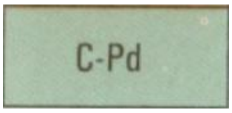
As seen in Figure 21 the route is directly underlain by O-Sn with Ng and C-Pd encountered in the immediate vicinity. These geological formations are described as:



Red-brown coarse-grained arkosic to sub arkosic sandstone; quartz arenite; micaceous sandstone; small pebble conglomerate; subordinate siltstone; and mudstone of the Natal Group.



Megacrystic biotite granite of the Natal Structural and Metamorphic Province (Structural Succession).



Diamictite; subordinate varved shale and boulder shale of the Dwyka Formation of the Karoo Sequence.

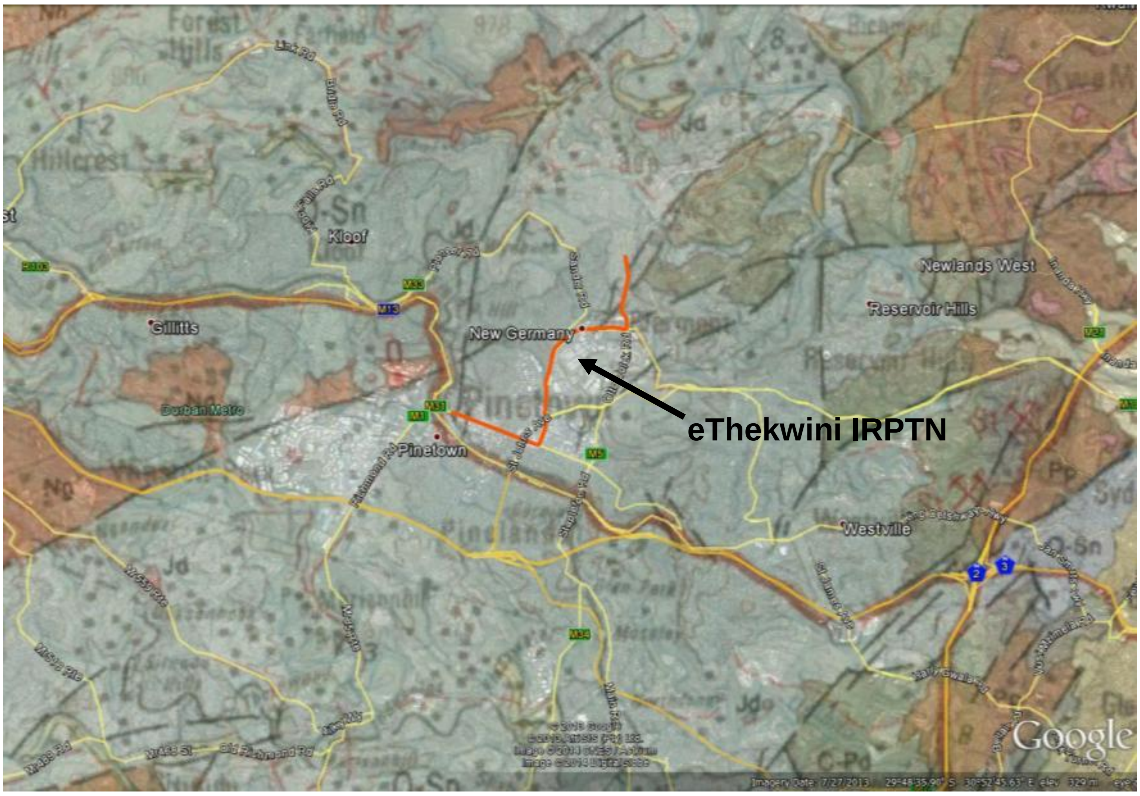


Figure 21: Google Earth image of the site, with the 1: 250 000 geological map 2930 Durban overlain.

4.1. SITE SPECIFIC GEOLOGY

Table 2 contains a summary of the findings gained from the test pit investigation. Appendix A contains additional information regarding each test pit.

Table 2: Test pit summary.

Route	Test pit	Layer									Refusal
		Loose to Medium dense, silty sand	Asphalt	Dense to very dense, (silty/clayey) sand	Soft to firm, (sandy/ clayey) silt	Firm to stiff, sandy clayey silt	Dense, sandy gravel	Loose (clayey/ silty) sand	Soft to firm, sandy clay	Soft rock to medium hard rock, Sandstone	
B2	TP B2-1	0.0 - 0.5m			0.5 - 2.5m						
	TP B2-2	0.0 - 0.3m								0.3 - 1.2m	1.2m
	TP B2-3				1.5 - 3.0m	0.0 - 1.2m				1.2 - 1.5m	
	TP B2-4	0.0 - 0.2m; 2.2 - 3.0m			0.2 - 2.2m						
	TP B2-5	0.0 - 0.2m									0.2m
	TP B2-6	0.0 - 0.5m				0.5 - 3.0m					
B1	TP B1-1	0.0 - 0.6m						0.6 - 2.0m			
	TP B1-2	0.0 - 2.5m									
	TP B1-3	0.0 - 2.5m									
	TP B1-4	0.0 - 1.2m									
	TP B1-5	0.0 - 2.0m			2.0 - 2.5m						
	TP B1-6	0.0 - 1.7m							1.7 - 2.5m		
	TP B1-7	0.0 - 1.5m							1.5 - 2.0m		
A2	TP A2-1	0.0 - 0.6m							0.6 - 2.5m		
	TP A2-2	0.0 - 0.5m				0.5 - 1.6m					

Route	Test pit	Layer									Refusal
		Loose to Medium dense, silty sand	**Asphalt**	**Dense to very dense, (silty/clayey) sand**	**Soft to firm, (sandy/ clayey) silt**	**Firm to stiff, sandy clayey silt**	**Dense, sandy gravel**	**Loose (clayey/ silty) sand**	**Soft to firm, sandy clay**	**Soft rock to medium hard rock, Sandstone**	
A2	TP A2-3	0.0 - 1.5m		1.5 - 2.5m							
	TP A2-4	0.0 - 1.5m							1.5 - 2.5m		
	TP A2-5	0.0 - 1.6m		1.6 - 2.0m							
	TP A2-6	0.0 - 1.0m		1.0m - 2.0m							
	TP A2-7				0.0 - 1.6m **						
	TP A2-8	0.0 - 1.0m		1.0 - 2.0m							
	TP A2-9	0.0 - 2.5m									
A1	TP A1-1	0.0 - 0.2m *						0.2 - 2.2m			
	TP A1-2	0.0m - 0.3m; 0.65 - 1.50m	0.3 - 0.35m		1.5 - 2.5m		0.35 - 0.65m				
	TP A1-3	0.0 - 1.8m									
	TP A1-4	0.0 - 1.2m							1.2 - 2.5m		
	TP A1-5	0.0 - 1.8m									
	TP A1-6	0.0 - 1.5m									
	TP A1-7	0.0m - 0.5m					0.5 - 0.8m	0.8 - 2.0m			

* with tree roots
** with sandstone boulders (500m)

5. MATERIAL PROPERTIES

5.1. LABORATORY TESTING

A total of 14 disturbed and 1 undisturbed block sample were retrieved during the investigation. These soil samples were submitted to Soilco Materials Investigations (Pty) Ltd, an independent soil testing laboratory in Durban for the following tests:

- Foundation indicator,
- pH and Conductivity,
- MOD and CBR,
- Moisture content, and
- Collapse Potential.

The samples were submitted to Soilco on 2 December 2013 and the bulk of the results received on 31 January 2014. The collapse potential test on the undisturbed sample is still in progress. The results are discussed in the following sub-sections, with a complete summary included in Table 3. See Appendix B for the complete set of results.

5.1.1. GRADING AND ATTERBERG LIMITS

A total of 14 samples were submitted for grading analyses including hydrometer, and Atterberg Limits. From the results it is clear to see that 8 of the 14 samples may be classified as cohesionless, with some 70 - 80% of the material particles tested within the sand-size range (0.075 – 2.0mm). For these materials no problems with regard to heaving are expected. However, collapse of the sandy structure may well be a problem – this is discussed further in Section 5.1.4.

The 6 remaining samples indicated PI values in the range of 5 – 11, which is indicative of some form of plasticity, but not any cause for problems for the founding of any structure.

Furthermore it is noteworthy that the sample from TP B2-1 exhibited slightly worse characteristics than the rest of the samples. The percentage passing the 0.075mm sieve was significantly higher the linear shrinkage, swell, MDD and OMC also confirm that this material differs from the other samples. The fairly low PI of 11 is however in slight contrast to the other parameters. It is therefore recommended that special care with this material be taken and should structures be intended for construction on it, an engineer must be asked to inspect the material once exposed on site for any signs of heaving or other problematic behaviour.

5.1.2. MOISTURE-DENSITY RELATIONSHIP

Moisture-density and CBR tests were conducted on 11 samples according to the methodology described in TMH1 (1986). The material generally possesses low strength characteristics, with many of the samples having tested for CBR values < 15 at 93% of the Mod AASHTO derived density (MAD). A CBR = 15 at 93% MAD is the minimum accepted value to be classified as a G7 quality material as per COLTO (1998). This means that the majority of the material on site will in all likelihood not be suitable for use as layerworks, but will be adequate for the founding of light structures.

A number of the samples did however indicate good results, i.e. G6 or G7 quality material. The samples from TP's A1-7, B1-4 and B2-6 will thus be suitable for use in layerworks.

5.1.3. PH AND CONDUCTIVITY

The soil exhibited a range of pH values ranging from 4.69 – 8.57. The average pH value is 5.98 which is slightly acidic.

The electrical conductivity of all samples were also tested. The results indicated that the soil on site in ranges from 'generally not corrosive' to 'highly corrosive'. The corrosivity at each location will need to be addressed accordingly.

5.1.4. COLLAPSE POTENTIAL

Unfortunately the collapse potential test was still in progress at the soil laboratory at the time of writing. It is however pertinent to take note that a pinholed structured was identified in the silty sand material in a number of the test pits. This must be duly noted by the engineer who will conduct the foundation inspections during construction.

Table 3: Summary of laboratory results

Sample No.	A1-1	A1-2	A1-4	A1-7	A2-2	A2-6	A2-9	B1-1	B1-4	B1-5	B1-6	B2-1	B2-2	B2-6
Depth (m)	2.0 - 2.2	1.5 - 2.5	1.2 - 2.5	0.8 - 2.0	0.5 - 1.6	1.0 - 2.0	0.4 - 2.5	0.6 - 2.0	0 - 1.2	0 - 2.0	1.7 - 2.5	0.5 - 2.5	0.3 - 1.2	0.5 - 3.0
Liquid Limit	-	28	29	-	27	28	-	-	-	-	-	34	23	-
Plasticity Index	NP	7	10	NP	6	9	NP	NP	NP	NP	SP	11	5	NP
Linear shrinkage	0.0	3.5	5.0	0.0	3.0	4.5	0.0	0.0	0.0	0.0	1.0	5.5	3.0	0.0
Grading modulus	0.97	0.84	0.77	1.01	0.77	0.83	1.03	0.97	1.01	0.95	0.84	0.51	1.54	1.12
% passing 0.425mm	88	87	90	81	95	85	77	83	77	88	93	90	62	67
% passing 0.075mm	17	31	33	19	30	33	21	22	23	18	24	65	21	22
Expansiveness rating	Low	Low	Low	Low	Low	Low	Low	Low	Low	Low	Low	Low	Low	Low
AASHTO Classification	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-2-4 (0)	A-6 (6)	A-2-4 (0)	A-2-4 (0)
pH values	7.49	4.69	4.76	5.09	6.38	5.14	6.35	7.92	5.04	5.94	4.85	6.30	8.57	5.23
Electrical Conductivity (S/m)	0.0154	0.0855	0.0135	0.0230	0.0188	0.0398	0.0079	0.0101	0.0064	0.0068	0.0115	0.0704	0.0272	0.0105
Corrosivity	Mildly corrosive	Very corrosive	Mildly corrosive	Corrosive	Mildly corrosive	Corrosive	Generally not corrosive	Mildly corrosive	Generally not corrosive	Generally not corrosive	Mildly corrosive	Very corrosive	Corrosive	Mildly corrosive
OMC (%)	9.7	11.6	12.6	7.5		13.8	7.0	7.3	7.4		11.6	18.5		7.1
MDD (kg/m^3)	1865	1930	1876	1914		1936	2052	2035	2084		1942	1616		2117
CBR @ 93% Mod AASHTO (%)	9	1	1	16		3	14	13	21		3	1		32
Max swell @ 100% Mod AASHTO (%)	0	4.09	8.02	0		2.36	0	0	0		1.97	7.09		0
Classification (COLTO)	G9	<G10	<G10	G7		<G10	G8	G8	G7		<G10	<G10		G6
In situ moisture content (%)		19.6		7.5			8.7	10.0			29.3	34.5		6.7

5.2. IN-SITU TESTING

Dynamic Cone Penetrometer (DCP) tests were the only in-situ tests completed during the investigation.

A summary of the DCP results can be seen in Table 4 on the next page. The allowable bearing capacity of each soil layer was calculated using a factor of safety of 4, to ensure that the expected settlement is controlled. The material consistency was categorised according to the DCP penetration rate (mm/blow). DCP test results commencing at depths other than zero indicate tests that were done from below current ground level, i.e. at the base of a test pit. Comprehensive results of the DCP investigation can be seen in Appendix C.

Material along all routes exhibited highly varying penetration rates. This indicates varying founding conditions which should be taken into account when designing the foundations of all structures. It is recommended that an experienced engineer be on site to assess the founding conditions of each structure as the foundation excavation commences.

In the analysis of the DCP results, empirical methods as described by Paige-Green and Du Plessis (2009) were used to interpret CBR and in turn allowable bearing capacities for the strata. These methods are only indicative and should be used as such. Experience has shown that more often than not these interpretations are reliable for feasibility level design.

Along the course of Route B2, all DCP tests were completed at the bottom of test pits along the route. Low CBR values at the start of the DCP test is in all likelihood due to soil disturbance caused by the TLB during test pit excavation and may not be representative.

The allowable bearing capacities approximated for the material underlying Route B2 generally increased with depth. The majority of the bearing capacities remained well below 100kPa, with some exceptions such as DCPs B1-6, B1-26, B1-27 and B1-28. In the occurrence of a seemingly high strength material present for a shallow depth (such as in the results of DCP B1-29) the DCP cone was likely passing through a pebble layer - this would produce unrealistic results and should thus not be used.

Routes A2 and A1 produced results very similar to Route B2.

The allowable bearing capacity for the soil strata was plotted against depth for each DCP. These graphs are included in Appendix D.

Table 4: DCP Summary

Route	DCP	Depth below ground level (mm)	CBR (%)	Allowable bearing capacity (FOS = 4) (kPa)	Material Consistency
B2	DCP B2-1	2 500-2 660	5	22	Firm
		2 660-2 800	22	75	Very Stiff
		2 800-2 850	120	312	Hard
	DCP B2-2	3 000-3 180	5	20	Firm

Route	DCP	Depth below ground level (mm)	CBR (%)	Allowable bearing capacity (FOS = 4) (kPa)	Material Consistency
		3 180-3 570	7	27	Stiff
		3 570-3 840	10	40	Stiff
	DCP B2-3	3 000-3 120	120	412	Very dense
	DCP B2-4	3 000-3 360	2	10	Soft
		3 360-3 550	8	37	Stiff
		3 550-3 730	45	127	Hard
B1	DCP B1-1	2 000-2 380	2	7	Very loose
		2 380-2 730	8	32	Medium dense
		2 730-2 760	30	85	Dense
	DCP B1-2	2 500-2 830	2	12	Loose
		2 830-3 050	7	30	Medium dense
	DCP B1-3	0-240	8	32	Medium dense
		240-920	3	15	Loose
	DCP B1-4	0-160	5	22	Loose
		160-820	2	10	Loose
		820-920	9	35	Medium dense
	DCP B1-5	0-640	4	17	Loose
	DCP B1-6	0-680	45	125	Very dense
	DCP B1-7	0-200	4	17	Loose
		200-560	10	40	Medium dense
		560-900	4	17	Loose
	DCP B1-8	0-120	7	30	Medium dense
		120-320	17	55	Medium dense
		320-930	7	30	Medium dense
	DCP B1-9	0-440	8	27	Medium dense
		440-850	2	7	Very loose
	DCP B1-10	0-240	17	60	Dense
		240-860	8	32	Medium dense
	DCP B1-11	0-570	4	17	Medium dense
		570-860	2	12	Loose
	DCP B1-12	0-730	2	10	Very loose
		730-860	7	30	Medium dense
B1	DCP B1-13	0-340	5	20	Loose
		340-870	1	5	Very loose
	DCP B1-14	0-870	5	22	Loose
	DCP B1-15	0-630	5	22	Loose
		630-810	11	42	Medium dense
	DCP B1-16	0-290	23	70	Dense
		290-870	6	27	Medium dense

Route	DCP	Depth below ground level (mm)	CBR (%)	Allowable bearing capacity (FOS = 4) (kPa)	Material Consistency
	DCP B1-17	0-360	15	52	Medium dense
		360-820	4	17	Loose
	DCP B1-18	0-880	5	20	Loose
	DCP B1-19	0-200	9	37	Stiff
		200-510	5	22	Firm
		510-870	2	10	Soft
	DCP B1-20	0-380	2	10	Soft
		380-860	5	20	Firm
	DCP B1-21	0-880	2	10	Soft
	DCP B1-22	0-210	3	15	Firm
		210-420	8	35	Stiff
		420-880	3	15	Firm
	DCP B1-23	0-860	2	12	Soft
	DCP B1-24	0-840	5	22	Firm
	DCP B1-25	0-260	6	30	Stiff
		260-810	2.5	12	Soft
	DCP B1-26	2 500-3 000	3	15	Firm
		3 000-3 300	40	120	Hard
	DCP B1-27	2 000-2 340	5	22	Firm
		2 340-2 720	60	150	Hard
	DCP B1-28	0-300	2.5	12	Soft
		300-540	12	50	Stiff
		540-640	60	155	Hard
		640-800	45	122	Hard
	DCP B1-29	0-640	2	10	Soft
		640-680	120	202.5	Hard
		680-820	8	27.5	Stiff
	DCP B1-30	0-280	7	30	Stiff
		280-700	3	15	Firm
		700-800	10	40	Stiff
B1	DCP B1-31	0-280	15	55	Very Stiff
		280-830	4	17	Firm
	DCP B1-32	0-700	<1	5	Soft
		700-750	17	62	Very Stiff
	DCP B1-33	0-580	6	25	Stiff
		580-800	17	65	Very Stiff
	DCP B1-34	0-860	1.5	12	Soft
	DCP B1-35	0-360	5	22	Stiff
		360-820	3	15	Firm

Route	DCP	Depth below ground level (mm)	CBR (%)	Allowable bearing capacity (FOS = 4) (kPa)	Material Consistency
	DCP B1-36	0-920	<1	5	Soft
A2	DCP A2-1	0-840	2	7	Soft
	DCP A2-2	0-860	4	17	Firm
	DCP A2-3	0-290	75	192	Hard
	DCP A2-4	0-180	4	17	Firm
		180-530	12	45	Stiff
		530-800	17	57	Very Stiff
	DCP A2-5	0-580	2.5	12	Loose
		580-1 400	8	35	Medium dense
		1 400-1 620	12	42	Medium dense
	DCP A2-6	0-670	<1	5	Very loose
		670-820	50	125	Very dense
		820-1 890	7	31	Medium dense
	DCP A2-7	0-440	2	7	Very loose
		440-1 680	3	15	Loose
	DCP A2-8	0-300	2	12	Loose
		300-680	30	107	Dense
		680-830	13	50	Medium dense
	DCP A2-9	0-230	4	18	Medium dense
		230-550	55	157	Very dense
	DCP A2-10	0-300	2	12	Loose
		300-780	45	100	Dense
	DCP A2-11	0-220	3	15	Loose
		220-360	95	225	Very dense
	DCP A2-12	0-200	160	365	Very dense
	DCP A2-13	0-420	2	7	Very loose
		420-690	6	27	Medium dense
		690-1 440	3	15	Loose
		1 440-1 830	6	27	Medium dense
	DCP A2-14	0-320	5	22	Loose
		320-680	10	42	Medium dense
	DCP A2-15	0-1010	3	15	Loose
	DCP A2-16	600-780	35	92	Dense
		780-1 400	120	177	Very dense
A1	DCP A1-1	2 200-2 380	5	20	Loose
		2 380-2 650	11	42	Medium dense
		2 650-2 890	22	80	Dense
	DCP A1-2	2 500-2 930	6	27	Stiff
		2 930-3 230	16	57	Very Stiff

Route	DCP	Depth below ground level (mm)	CBR (%)	Allowable bearing capacity (FOS = 4) (kPa)	Material Consistency
		3 230-3 390	35	110	Hard
	DCP A1-3	0-310	12	50	Medium dense
		310-830	5	17	Loose
		830-1 210	22	65	Dense
		1 210-1 900	3	15	Loose
	DCP A1-4	1 800-2 540	4.5	20	Loose
		2 540-2810	13	52	Medium dense
		2 810-3 250	27	80	Dense
		3 250-3 400	30	110	Dense
	DCP A1-5	2 500-2 820	5	22	Loose
		2 820-3 160	16	57	Medium dense
	DCP A1-6	1 100-1 220	6	30	Medium dense
		1 220-1 560	15	55	Medium dense
		1 560-1 900	20	70	Dense
	DCP A1-7	1 500-2 660	5	20	Loose
		2 660-2 780	45	122	Dense
	DCP A1-8	2 000-2 480	5	25	Medium dense
		2 480-2 720	60	125	Very dense

6. GEOTECHNICAL EVALUATION

6.1. GROUNDWATER

No groundwater was encountered during the excavation of any test pits except TP B2-2. No major problems are therefore expected for groundwater.

Groundwater was encountered in TP B2-2 at a depth of 0.5m below ground level. This test pit was situated just east of Dinkelman Road near to the off-ramp bridge leading to 11th Avenue.

6.2. EXCAVATION CONDITIONS

Based on SABS 1200, “soft” excavation conditions can be expected for the entire site area down to depths of 2.0m to 2.5m.

Refusal with the TLB was only encountered at 2 test pits, TP B2-2 and TP B2-5, both at the Dinkelman Road and 11th Avenue bridge site. Further investigation in the form of rotary core drilling has been commissioned at this bridge. The drilling is set to commence in February 2014 pending final approval by the client.

6.3. IRPTN STATIONS

The geotechnical evaluation of each station was completed separately, according to the soil conditions in the locality of each station. The founding recommendations for the stations will be discussed in Section 7.2.

6.3.1. HARMONY HEIGHTS STATION

It is assumed that the soil conditions underlying Harmony Heights Station will be similar to those encountered in TP B2-1 and DCP B2-1. It is therefore interpreted that the station will be underlain by medium dense silty sand underlain by firm clayey silt.

6.3.2. HENDRED STATION

Results from TP B1-1, TP B1-2, TP B1-3, DCP B1-1 and DCP B1-2 were used in the evaluation of the geology underlying Hendred Station. From this information it is interpreted that the station will be underlain by loose to medium dense silty sand.

6.3.3. REGENT STATION

Soil conditions encountered in TP B1-6, TP B1-7, DCP B1-24, DCP B1-25, DCP B1-26 and DCP B1-27 were used in the evaluation of the geology underlying Regent Station. From these sources it is inferred that the station will be underlain by loose silty sand followed by soft sandy clay.

6.3.4. HENWOOD STATION

Findings from TP A2-8, DCP A2-8, DCP A2-9 DCP A2-10, DCP A2-11 and DCP A2-12 will be used in the geotechnical consideration of Henwood Station. It was hence inferred that the station will be underlain by medium dense to dense silty sand. The allowable bearing capacities obtained from the in-situ testing increased consistently from DCP A2-8 to DCP A2-12.

6.3.5. BEVISS STATION

Findings from TP A1-1, DCP A1-1, DCP A2-14, DCP A2-15 and DCP A2-16 will be considered during the recommendation regarding Beviss Station. The station will accordingly be underlain by loose silty sand.

6.3.6. PINETOWN CIVIC STATION

TP A1-5 was the only test pit excavated in the locality of Pinetown Civic Station and will therefore be used in the geotechnical evaluation of Pinetown Civic Station. According to the test pit profile the station will be underlain by loose silty sand.

6.3.7. MOODIE STATION

The findings retrieved from TP A1-6, TP A1-7, DCP A1-7 and DCP A1-8 were used in the geotechnical considerations regarding Moodie Station. It is therefore inferred that the station will be underlain by loose silty sand.

7. RECOMMENDATIONS

The recommendations in this report have been created using the information gathered during the site visit. Visual observations of the in situ material, as well as DCP test results, have been used to provide meaningful recommendations.

7.1. RETAINING WALLS

Approximately 3.8km of retaining walls will be required along all four of the route sections. None of the retaining walls will exceed a height of 3m. It is suggested that the test pit investigation results be used to estimate the shear strength parameters of in situ material, while DCP test results are used to estimate allowable bearing capacities. The retaining walls bordering the underpass of Josiah Gumede Road and St John's Avenue are not discussed here, but in Section 7.4.

From the test pit profiles it is clear that the majority of the route is underlain by silty sand material. It is therefore expected that the majority of the retaining walls will be founded on this material type. From a visual appraisal this material does not appear to be problematic. Its loose to medium dense consistency does however need to be addressed prior to placing the foundations of the walls. Small hand-operated compaction equipment will suffice for this purpose. The DCP results appended in this report should be used as estimates for bearing capacity. It is however recommended that the engineer on site inspect each foundation prior to approving it for the placement of the foundation concrete. This inspection should be aimed at identifying problematic materials in the form of either collapsible sandy or soft clayey type materials, and also at inspecting the consistency of the founding so as to ensure that no excessive settlements will take place.

7.2. IRPTN STATIONS

From the individual station discussions in Section 6.3 it is clear to note that all stations will be underlain by fairly similar soil conditions, i.e. layers of silty sand overlying sandy clay material of varying consistency. For this reason it is concluded that one general founding methodology applied for all stations will suffice. This is discussed in the following sub-sections.

7.2.1. SITE COMPACTION

Apply the following preparation and compaction for the stations:

- Remove topsoil (approximately 300mm) and all organic material,
- Using a smooth drum roller operating in vibratory mode, compact the entire extent of the station site prior to any construction activities for 20 passes. A BOMAG 219D or similar approved, single drum roller with VARIOCONTROL is recommended,
- Ensure that the in-situ material is at or slightly wet (+2%) of its optimum moisture content prior to the compaction. This is necessary to ensure that the material is sufficiently densified,
- Found as shallow as possible within compacted horizon using conventional pad or strip footings. See Section 7.2.2.

7.2.2. POSSIBLE FOUNDING SOLUTIONS

Conventional strip footings can be considered for light loads (up to 150kPa). Differential settlement may result if footings belonging to the same structure are placed in different strata (e.g.: differential settlement will occur between footing "A" and "B", if A is placed in the soil profile and B is placed on rock). If soft rock is encountered in the foundation excavations it is recommended that excavation is continued so that all footings are placed on a similar strength material.

7.2.3. FLOOR SLABS

Following the compaction rolling, only light preparation work will be necessary prior to floor slab construction. Backfill to the correct level using G7 quality material if necessary and compact to 93% of the Mod AASHTO density at 0 to +2% of the optimum moisture content.

7.3. BRIDGE

Five test pits were excavated in the vicinity of the bridge where 11th Avenue passes over Dinkelman Road. The TLB excavations refused at TP B2-2 and TP B2-5 on soft to medium hard rock sandstone. Depth to bedrock at the other 3 test pit locations is unknown.

At this preliminary stage it is expected that a combination of piled and shallow pad footings will be incorporated for the founding of the bridge structure. This will however be confirmed by the results of the core drilling anticipated to commence in February 2014.

7.4. UNDERPASS BRIDGE

At the underpass bridge at the intersection of St John's Avenue and Josiah Gumede Road, a total of 4 test pits and 5 DCP tests were conducted. TP A1-1 to TP A1-4 were all done on the eastern side of the bridge, with the aim of being placed as close as possible to the centreline underpass. Due to the layout of the bridge it was unfortunately only possible to place the test pits some 140m away from the centreline. DCP A1-2 to A1-6 were done on the eastern and western sides of the bridge; all were also placed a distance away from the centreline due to practical reasons.

The test pit profiles indicate that the area is underlain by silty sand transported material to an approximate depth of 1.2m, followed by a sandy clay residual mudstone layer continuing down to at least 2.5m below current ground level. The depth to bedrock is unknown. These test pits were however done at grade, while the bridge and accompanying retaining walls will be founded some 3-4m below grade. It is therefore expected that the residual mudstone layer will stiffen with depth, as was suggested by the DCP results, and that conventional pad or strip footings will in all likelihood suffice for the intended structures with allowable bearing capacity ranging from 50kPa to 120kPa according to the DCP results.

8. GENERAL

The comments and recommendations contained within this report are based on a limited number of test pits excavated and DCP tests conducted which we believe are representative. Therefore, conditions at variance with what is described herein should not be overlooked.

9. REFERENCES

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APPENDIX A
SOIL PROFILES

APPENDIX B
LABORATORY RESULTS

APPENDIX C
DCP RESULTS

APPENDIX D
BEARING CAPACITY GRAPHS

BRIDGES

DAMS & HYDRO

GEOTECH

STRUCTURES

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