**DRENNAN MAUD (PTY) LTD**

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**OUR REF:** 31169

25 November 2015

Afzelia Environmental Consultants
P.O Box 37069
Overport
Durban
4067

**Attention : Mr Andrew Batho**

E-mail: andrew@afzelia.co.za

Dear Sirs,

## GEOTECHNICAL AND GEOHYDROLOGICAL REPORT : PTN 151 OF ERF 8 BRIDGE CITY PROPOSED BP SERVICE STATION & SHOPPING COMPLEX

## 1. INTRODUCTION AND TERMS OF REFERENCE

Further to our fee proposal to yourselves referenced ‘91' and dated 19 October 2015, and your subsequent appointment on 29 October 2015, Drennan Maud (Pty) Ltd carried out a geotechnical and geohydrological assessment of the abovementioned proposed development.

The field investigation was undertaken in early November 2015, it comprising the excavation of inspection pits within the footprint of the proposed development and thereafter a drive-over and walk-over of the site proximity to establish topographical and geological/geohydrological conditions in the immediate area.

Recorded hereunder are our findings and recommendations in this matter.

## 2. INFORMATION PROVIDED

Prior to carrying out the field investigation, a sketch site plan was provided courtesy of MAB iKhwezi Architects, it showing the footprint of the proposed development relative to existing Nkuzana Road and Bridge City Boulevard.

## 3. SITE DESCRIPTION AND DEVELOPMENT PROPOSAL

### 3.1 Site Description

The site is located on the immediate western corner of the junction between Nkuzana Road and Bridge City Boulevard.

The existing ground surface is effectively flat at this location, it having been leveled some 20 years ago for Moreland Estates, during the original earthworks for the greater Bridge City development.

At the time of our field investigation being carried out, a newly laid stormwater pipeline along the southern site servitude boundary had recently been back-filled, and the manhole excavations were still open for our observation.

### 3.2 Development Proposal

From a review of the sketch plan provided, it is evident that the site is to be developed as a BP service station, comprising a typical forecourt area and BP Express Shop, in addition to an anchor shop, tanker loading bay and general parking bays.

It is assumed that new underground fuel tanks associated with the proposed development will need to be installed to a typical depth of about 5m below existing ground level.

No detailed drawings (structural or earthworks) or final platform levels have been provided to date.

## 4. FIELDWORK

The approximate positions of all trial excavations and field tests executed as part of the present assessment are indicated on the attached site plan, Figure 1.

Geotechnical works conducted on this site involved the logging of 2No man-made exposures (manhole excavations), the excavation, logging and sampling of 4No inspection pits, conducting 4No dynamic cone penetrometer tests and carrying out 1No percolation test. In addition, representative subsoil samples were obtained for laboratory testing.

### 4.1 Excavation of Inspection Pits: IP1 - 4

Inspection pits were mechanically excavated using the backhoe of a TLB (JCB 3CX) having a maximum reach of about 3m.

The soils and bedrock exposed in the sidewalls of inspection pits were logged by an

Engineering Geologist in accordance with the Guidelines for Soil and Rock Logging in South Africa, edited by A.B.A Brink & R.M.H Bruin, 2nd Impression 2002, with the geotechnical soil logs being included in Appendix 1 herewith.

## 4.2 Dynamic Cone Penetrometer Testing: DCP1 - 5

DCP tests were carried out in order to establish the consistency of subsoils and weathered bedrock material at shallow depths beneath the site.

DCP test results are graphically presented in Appendix 2 herewith, whilst Table 1 provides a guideline for interpreting the tests.

**Table 1: Guideline to Interpreting DCP Test Results**

| **Non Cohesive Soils** | | **Cohesive Soils** | |
|---|---|---|---|
| **No. Of Blows/300mm Penetration** | **Subsoil Consistency** | **No. Of Blows/300mm Penetration** | **Subsoil Consistency** |
| <8 | Very Loose | <4 | Very Soft |
| 8 - 18 | Loose | 4 - 8 | Soft |
| 19 - 54 | Medium Dense | 9 - 15 | Firm |
| 55 - 90 | Dense | 16 - 24 | Stiff |
| >90 | Very Dense | 25 - 54 | Very Stiff |
| | | >54 | Hard |

It is academic to note that all DCP tests encountered refusal on weathered bedrock within 1,2m of existing ground level, with the overburden soils, where encountered, being generally of very stiff consistency.

## 4.3 Logging of Exposure Profiles: EXP1 - 2

Man-made exposures in the form of relatively deep manhole excavations along the southern site servitude boundary were logged by an Engineering Geologist.

The geotechnical profiles thereof are included in Appendix 1 with the inspection pit profiles.

## 4.4 Material Sampling

A total of 3No bulk disturbed samples were obtained of the prevailing substrata, and sent to Thekwini Soils Laboratory in Durban, where they were subjected to Foundation Indicator, Mod. AASHTO moisture-density and CBR testing.

Table 2 provides a schedule of the material samples obtained and laboratory testing conducted thereon;

**Table 2: Schedule of Material Sampling and Laboratory Testing**

| Sample Location | Depth (m) | Material Description | Laboratory Testing | | |
|---|---|---|---|---|---|
| | | | F. Indicator | Mod. | CBR |
| IP2 | 0 - 0.8 | Colluvial Soil | ✓ | ✓ | ✓ |
| IP3 | 0.7 - 1.1 | Moderately weathered SHALE | ✓ | ✓ | ✓ |
| IP4 | 1.1 - 2.9 | Moderately weathered SHALE | ✓ | ✓ | ✓ |

Laboratory test results are included in Appendix 3 herewith and discussed further under Section 6.

### 4.5 Percolation Testing: PT1

A single percolation test was carried out in the central portion of the site, at a depth of 0,7m below existing ground level in the weathered bedrock.

The percolation test pit profile (PT1) was logged by an Engineering Geologist and the profile thereof is included in Appendix 1 herewith.

Following the prescribed soaking period in terms of SABS 0400, the percolation test failed to record any drop in water level over the half hour duration of the test.

## 5. SITE GEOLOGY

### 5.1 General

The site is underlain at generally very shallow depths by cut shale bedrock of the Pietermaritzburg Formation, which is obscured from view at the existing ground surface by a very thin veneer (0,1 to 0,5m) of fill material placed thereon.

In the northwestern corner of the site, as approximately indicated on the attached Figure 1, the platform has not been cut into bedrock, and is underlain by a moderate thickness (≤1,5m) of the very clayey residual and colluvial soils capping the bedrock. The inferred extent of this area is indicated on the attached Figure 1.

### 5.2 Fill Materials

Fill material on this site was encountered to insignificant thicknesses (range 0 to 0,5m, but generally <0,2m). The material is of variable origin (shale to granite), hence its composition varies from a closely packed clayey to sandy gravel, to a coarse grained gravelly sand.

## 5.3 Residual and Colluvial Soils

The residual and colluvial soils encountered in IP2 were both recorded as being very stiff, strongly fissured, slickensided and locally shattered (highly active) clay soils at their relatively dry natural moisture content.

## 5.4 Shale Bedrock

The shale bedrock occurs generally as a moderately weathered, dark grey, medium to widely jointed, laminated soft rock, with this material being locally capped by a thin layer of highly weathered shale, which occurs as a brown, closely to medium jointed, laminated very soft rock.

The bedding planes within the laminated shale, which constitute the most significant and pervasive discontinuity in this rock material, dip at an angle of approximately 16$^o$ toward the southeast (dip direction 170).

## 5.5 Karoo Dolerite

No dolerite bedrock was encountered on this site, and none is expected.

However, a substantial dolerite sill is known to occur approximately 100m east of the site. The dolerite, being a sill, is sandwiched between the shale bedding, hence also likely to dip gently toward the southeast.

The dolerite is expected to occur as a deeply weathered clayey and silty soil to highly fractured soft rock, and its zones of contact with the host-rock shale are expected to behave as groundwater conduits.

# 6. LABORATORY ANALYSIS

## 6.1 Colluvial Soil: IP2, 0 - 0.8m

This material classifies as A-7-6 (22) in terms of the AASHTO Classification and as CL (but very nearly CH) in terms of the Unified Classification. It recorded a clay content of approximately 60%, a Plasticity Index of 23, a Linear Shrinkage of 11% and a Grading Modulus of 0.27.

In terms of the above, the colluvial is a highly plastic, potentially highly active clay that should not be used in any aspect of the proposed development.

The material Mod AASHTO density was recorded at 1680kg/m$^3$ at an optimum moisture content of about 17%, with a calculated CBR value of 1% at 93% Mod AASHTO. A CBR swell of 6% was recorded for the material, which is considered highly unsuitable for use as fill material under any circumstance.

## 6.2 Moderately Weathered SHALE: IP3, 0.7 - 1.1m, IP4, 0 - 0.6m

The samples classified A-2-4 (0) in terms of the AASHTO Classification and as GP-GW to GP-GC in terms of the Unified Classification, being a coarse cobble-gravel material with negligible clay content. The material Plasticity Index is in the order of 6 to 8, Linear Shrinkage 5 to 6% and Grading Modulus approximately 2,6.

In terms of its grading alone, the material would appear to be a good subgrade.

The material Mod AASHTO density was recorded at approximately 1940kg/m$^3$ at an optimum moisture content of about 10%, with CBR values in the order of 3 to 4% at 93 to 95% Mod AASHTO. Although these CBR values are commensurate with a G10 material after TRH 14 (1985), the samples recorded an excessive CBR swell of 2,5 to 3%, hence they preclude classification and will not be suitable for use as subgrade material.

## 7. GEOTECHNICAL ASSESSMENT

This section appraises the site geotechnical conditions relative to the proposed development.

## 7.1 General

The site is generally underlain almost immediately from existing ground level by cut shale bedrock of the Pietermariztburg Formation, over which a negligible thickness (generally <0,3m) of variable fill material has been placed.

Exception to the above is the northwestern corner of the site (Fig. 1 refers), at which location a thickness of up to 1,5m of the shale-derived, active clay soils mantle the weathered bedrock.

In terms of the development proposal, founding conditions for the structures are considered to be favourable, however the presence of cut shale at surface means consideration must be given to the bedrock excavatability for subsurface fuel tanks.

Discussed below are the geotechnical aspects of the site most likely to influence the proposed development;

## 7.2 Excavatability

As stated, cut, moderately weathered, medium to widely jointed soft rock shale occurs almost immediately from existing ground surface across the greater portion of the site. This material is expected to grade into medium hard rock at a general depth of about 3m below existing ground level.

Inspection pits excavated during the present field investigation achieved depths ranging from 1,1 to 1,8m below existing ground level (in trench excavation), averaging about 1,5m, before encountering a dull refusal on soft rock shale at that depth.

It follows that excavation depths in the order of 1,5m below existing ground level will generally be achievable under the “soft excavation” class of SABS 1200D, thereunder, “intermediate excavation” is expected.

From previous work carried out at the northern end of the Bridge City upper platform for the new BRT bus route, it is considered likely that “intermediate excavation” will be achievable to depths of about 5 to 6m below existing ground level before any blasting might be required. This is supported by observing construction activities just 50m east of the site, at which location temporary excavations of up to 6m height have been carried out in very similar shale bedrock using 20ton excavators fitted with rock buckets, as part of the new BRT construction.

### 7.3 Site Drainage

Stormwater will be prone to pond on the site surface if it is not adequately graded, as a consequence of the relatively impermeable shale bedrock.

No groundwater seepage is expected within the scope of normal fuel tank excavations (~5m depth), however, stormwater will be liable to pond in the excavations, which may necessitate a sump and pump in the wet season.

### 7.4 Erosion Potential

None of the soils encountered on site during the present investigation are considered erodible to any significant degree, and the shale bedrock will not be erodible under normal conditions.

### 7.5 Excavation Stability

Excavations into the moderately weathered shale will be inherently stable at steep angles, even vertically, in terms of the overall rock mass strength.

However, unfavourably orientated release joints are sure to occur within the scope of the deep fuel-tank excavation, meaning dislodged blocks sliding or toppling from the excavation faces could pose a risk to any workers therein if not addressed.

### 7.6 Founding Conditions

The moderately weathered shale occurs within 1,5m of existing ground level, and in most cases almost immediately from existing ground level.

It follows that the shallow founding conditions across the site are highly favourable, and a design bearing pressure of 200kPa can confidently be applied for foundations taken into the hard hand-pickable shale.

### 7.7 Materials Suitability

The limited thickness of clay soils on the northwestern portion of this site will not be suitable for use in the proposed development, as they are highly active and will undergo volume changes with fluctuating moisture content. In addition, they will be prone to soften and heave if used in earthworks.

The highly weathered (brown, very soft rock) shale underlying this site will be similarly poor for use in engineering, as it will break down upon compaction to produce a very clayey (active) material with a high plasticity index, that will be prone to heave.

The moderately weathered (dark grey) shale sampled during the present investigation recorded CBR values commensurate with a G10 material in terms of TRH 14, which would provide adequate strength as a subgrade, but the excessive CBR swells recorded make it unsuitable for this purpose, as it will heave during compaction and be susceptible to shrinkage and swell over wetting and drying cycles post-construction.

Another consideration regarding the moderately weathered shale is that it will be recovered as relatively hard platelets that are difficult to compact using lighter plant (rollers <10t).

## 8. GEOHYDROLOGICAL ASSESSMENT

As far as can be ascertained from the Department of Water Affairs borehole database, there are no water abstraction boreholes occurring within a 1km radius of the proposed development. This ties in with the site surrounds being largely dedicated to housing, shopping and industry.

There are however two perennial drainage lines in the vicinity of the proposed development, which are discussed below.

### 8.1 Drainage Line 1, Approximately 550m North of Site

This perennial drainage line has largely been canalized in order to realign it through the Phoenix Industrial development.

Upon leaving the general Phoenix Industrial park, the canalized drainage line opens up and bifurcates into a southerly member, which flows into the Piesang River, itself a tributary of the greater Umgeni River, and a northerly member, which ultimately dissipates in close proximity to Marshall Dam on the Blackburn Estate.

The stream channels have a relatively steady flow and the unconsolidated alluvial soils within the channels are expected to comprise predominantly sandy material.

## 8.2 Drainage Line 2, Approximately 650m South of Site

This perennial drainage line constitutes the main watercourse of the Piesang River, which is joined by a number of smaller tributary streams *en route* to its ultimate discharge into the Umgeni River some 4km from its mouth. Sections of the river within Phoenix Industrial and Springfield/Sea Cow Lake have been canalised.

The unconsolidated alluvial sediment within the river channel is known to comprise predominantly sandy material.

## 8.3 General Aquifer Conditions and Shallow Soil Permeability

### 8.3.1 *Pietermaritzburg Formation Shale and Associated Soils*

The shale bedrock underlying the footprint of the proposed development is a notoriously poor aquifer. The consolidated clay particles making up the intact shale bedrock are effectively impermeable and it is only within zones of highly fractured bedrock that aquifers of the secondary type could potentially occur.

In terms of the present field observations, it is evident that the shale bedrock across the site is generally a moderately weathered, medium to widely jointed, unfractured soft rock. The smooth, planar joint surfaces therein are very tight or otherwise sealed with a thin in-fill of stiff clay gouge.

Where *in-situ* subsoils are developed over the shale bedrock, the permeability of both the residual subsoil and colluvial soil is very low, they being of invariably high clay content and generally classifying as clay/silty clay for the residual shale and as silty/sandy clay for the colluvium. From previous experience, percolation tests in these materials generally fail dismally.

### 8.3.2 *Karoo Dolerite and Associated Soils*

The dolerite sills and dykes occurring on the upper Bridge City platforms are well known to the author, they having been exposed and observed during construction along the new BRT route, as well as pipeline trenches and trial pits across the upper platforms.

The dolerite has been substantially altered by prolonged groundwater seepage therein, now typically occurring as a saturated silty clay to clayey silt. The upper and lower contact zones between the dolerite and the shale bedrock are generally highly fractured, acting as groundwater conduits through which significant seepage may be observed in the wet season.

As stated, the nearest dolerite intrusion appears to be approximately 100m east of the site, but it is a relatively major occurrence, being a thick (est. 20m true thickness) northeast-southwest trending sill that dips at approximately 15 to 20° toward the southeast.

In consideration of the above, the dolerite must be considered a potential aquifer within the shale. Any groundwater reaching the dolerite is likely to be transmitted fairly rapidly along the strike of the sill (in a predominantly south-southwesterly direction), and may feasibly seep into the Piesang River (drainage line 2, approximately 650m south of site).

No geological structures connecting the abovementioned dolerite aquifer to the current site are evident, however caution should be applied as such structures, be they faults, smaller dolerite bodies or simply more pervasive linear zones of fractured bedrock, may not be immediately evident at surface.

## 9. RECOMMENDATIONS FOR DEVELOPMENT

The recommendations provided below are intended as a guideline to assist the responsible Engineer and appointed Contractor in the design and construction of the proposed development. These amount to no more than sound building practice with respect to the site geotechnical conditions.

### 9.1 Site Preparation and Earthworks

No detailed earthworks plan or final levels have been provided to date. Following below are general recommendations for the site preparation and earthworks.

### 9.1.1 *Site Preparation*

Prior to construction, the veneer of fill materials should be scraped off the site and removed to spoil. So too should any clay soils capping the underlying shale bedrock, such that the prepared site exposes shale bedrock at the surface.

Where the thickness of clay soil is found to extend to greater than about 1m below final platform level, as is expected over a limited area in the northwestern corner of the site, the material should be boxed out to a pre-determined depth (about 1m below final platform level) and spoiled, thereafter being replaced with an imported gravel-soil of at least G7 quality, engineered in layers of 200mm loose thickness to at least 95%, but preferably 98% of the material Mod. AASHTO density.

The above general recommendation is made in consideration of the anticipated heavy truck traffic across new road pavement at this location. The actual depth to which the active clay soil is boxed out, and the quality of the imported material with which it is replaced, should be confirmed by the design Engineer.

### 9.1.2 *Cuttings and Deep Excavations*

No permanent cuttings are expected on this site, however deep excavation for the installation of fuel tanks is envisaged.

The deep excavations are likely to be entirely into shale bedrock, the excavatability of which is discussed under Section 7.2.

Regarding the temporary stability of the excavation sidewalls, as stated there will undoubtedly be some adverse release joints in the relatively hard shale bedrock. This could be addressed either by trimming the temporary excavation back to a safe batter of 1:1 (45°) for the duration of construction, or alternatively, incorporating a temporary rock-fall protection system comprising steel mesh bolted to the rock face. The latter option would be preferred as the excavation could then couple as the structural formwork and back-filling behind the tanks could be avoided.

Should any permanent cuttings be created on this site, a final batter of 1:2 (26°) should be applied to the cut slope.

### 9.1.3 *Filling and Back-filling*

Filling on this site is expected to be limited to the engineering of new subgrade and layerworks for access roads and surface beds, but possibly also back-filling behind the new underground tanks.

Prior to construction of new fill, the *in-situ* ground surface should be ripped and re-compacted to 95% Mod AASHTO density.

The compaction requirements for the new fill will vary depending on the material composition and purpose, but in general, a compaction of 93 to 95% Mod AASHTO should be sought for the general fill material, while 95 to 98% Mod AASHTO is preferred for new layerworks, backfill and selected fill.

Should backfilling be required behind the new underground tanks, the backfill material should comprise an imported granular material, as the *in-situ* shale bedrock will not be suitable for this purpose.

## 9.2 Founding of Structures

All structures proposed on this site should be founded into the weathered shale bedrock, which occurs generally within 0,5m of existing ground level, but up to 1,5m depth in the northwestern corner of the site.

Structures could be founded on conventional column bases or nominally-reinforced strip footings excavated by hand into the shale bedrock, wherein an allowable bearing pressure of 200kPa could be applied for design purposes, with negligible settlements to follow. The final founding depth should not be less than 0,5m below final platform level.

Should any localised soft spots (pockets of clay material) be encountered at foundation level, these must be over-excavated and backfilled with a suitable soilcrete material. Prior to casting foundations, a geotechnical professional should be called to inspect the foundation trenches.

### 9.3 Buried Fuel Tanks

The fuel tanks will be buried into a cavity excavated into generally soft to medium hard shale bedrock. These tanks could potentially leak in the future. Although the shale bedrock under normal conditions has an extremely low permeability, it is possible that more fractured zones occur, through which any leaked hydrocarbons could potentially seep until they intersect the nearest dolerite body, which is inferred to be a good aquifer, and could in theory transmit the hydrocarbons down strike (south-southwest) toward the Piesang River.

As a precautionary measure to mitigate the potential for leakage migrating toward an aquifer, the new fuel tank excavations should be thoroughly sealed prior to construction of the new tanks therein.

### 9.4 Site Drainage

As the greater portion of the site will be immediately underlain by shale bedrock, the site grading and drainage must seek to prevent ponding of stormwater following heavy downpours.

Lined interceptor drains connecting to a sand, oil and grease trap should be installed around the site periphery to collect the run-off deriving from the forecourt area.

Areas outside the forecourt should be ensured to adequately grade away from the forecourt, with all runoff therefrom being collected in suitable surface drains and discharged into the Municipal system allowed for.

### 9.5 Road Pavement Design

As laboratory testing shows the ripped and re-compacted shale bedrock to be unsuitable as *in-situ* subgrade, it is recommended that a suitable granular material be imported for this purpose or alternatively, the new pavement layerworks be thickened accordingly.

On the northwestern portion of the site, and any other area where pockets of clay soil might be found to occur at subgrade level, subgrade improvement similar to that recommended under 9.1.1 should be carried out prior to construction of the new layerworks and pavement.

## 10. CONCLUSIONS

The geotechnical and geohydrological conditions across the site are considered relatively favourable and there is no indication of any fatal flaw that might preclude the development of this site as a service station.

Nevertheless, the design and construction of the proposed development should take into account the recommendations set out in this report to ensure the long-term success thereof.

## 11. DISCLAIMER

The ground conditions described in this report refer specifically to those encountered in the trial pits and exposures during the field investigation. It is therefore quite possible that conditions at variance with those observed could be encountered elsewhere on site during construction. The information in this report is given in good faith, as an indication of materials and conditions likely to be encountered during construction. There is no warranty that the information is entirely representative of the whole area and no responsibility will be accepted for any consequences arising from actual conditions differing from those indicated in this document.

We trust that this meets your requirements in this matter however, should you have any further queries please contact the undersigned.

Yours faithfully,
**DRENNAN, MAUD (PTY) LTD**

**M.D. COOPER** **Pr.Sci.Nat**
/mh

**DRENNAN MAUD (PTY) LTD**

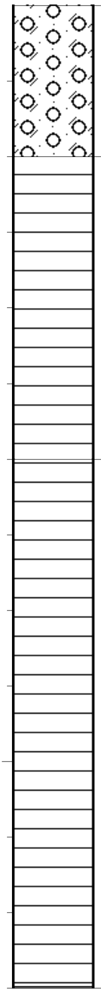
Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* IP1
*Sheet 1 of 1*

*JOB NUMBER:* 31169

*Scale*
*1:10*



| Depth | Description |
|---|---|
| *0.00* | Slightly moist, brown to grey brown, closely packed, relatively loose, coarse, angular shale GRAVEL in a sandy and clayey matrix – (Fill). Highly to moderately weathered shale gravels and few small cobbles. |
| *0.20* | Highly weathered, brown to grey brown, closely to medium jointed, laminated very soft rock SHALE – (Pietermaritzburg Formation). |
| *0.60* | Moderately weathered, dark grey to olive grey, medium to widely jointed, laminated soft rock SHALE – (Pietermaritzburg Formation). |
| *1.30* | |

*1*

NOTES

1) Refusal of machine on shale bedrock

| *Smpl.* | *Depth.* |
|---|---|

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC

*INCLINATION :* NA
*DIAM :* NA
*DATE :* NA
*DATE :* 04.11.2015

*ELEVATION :*
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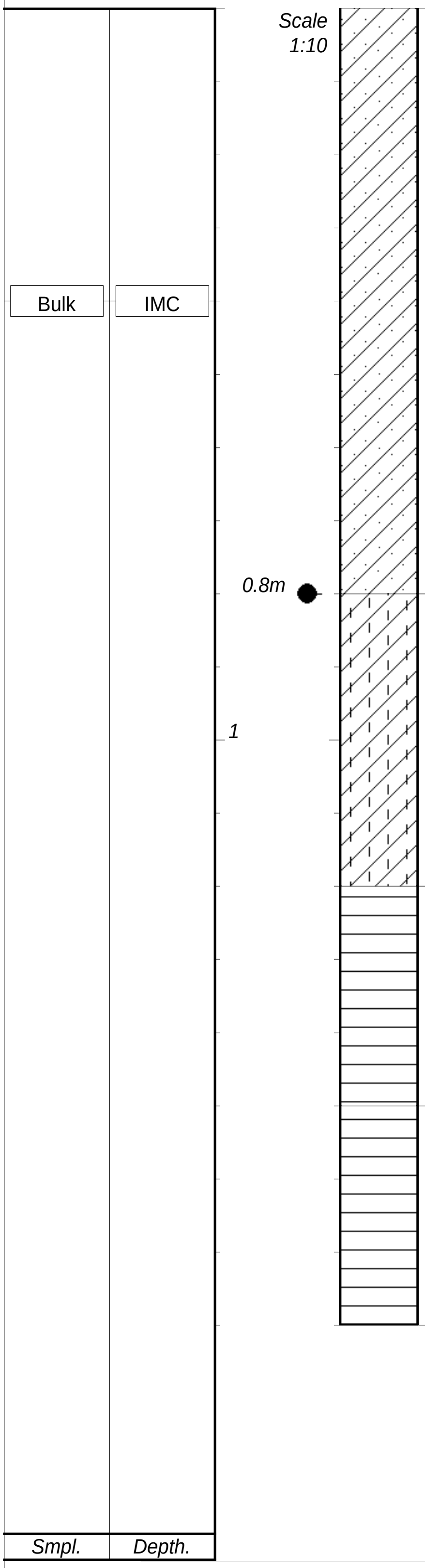
DRENNAN MAUD (PTY) LTD

Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* IP2
*Sheet 1 of 1*

*JOB NUMBER:* 31169



| | |
|---|---|
| 0.00 | Slightly moist, very dark grey brown, very stiff, shattered sandy CLAY – (Colluvium). |
| 0.80 | Slightly moist, olive grey streaked yellow brown, very stiff, fissured and slickensided silty CLAY – (Residual Shale). |
| 1.20 | Highly weathered, brown to grey brown, very closely to closely jointed, laminated very soft rock SHALE – (Pietermaritzburg Formation). |
| 1.50 | Moderately weathered, grey to dark grey stained orange on joints, medium jointed, laminated soft rock SHALE – (Pietermaritzburg Formation). |
| 1.80 | |

NOTES

1) Refusal of machine on shale bedrock

2) Bulk disturbed sample obtained from 0 to 0.8m

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC
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*HOLE No:* IP2

DRENNAN MAUD (PTY) LTD

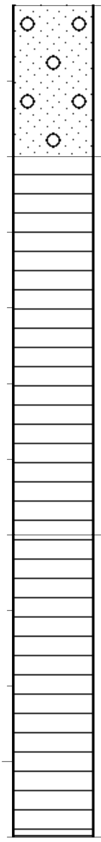
Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* IP3
*Sheet 1 of 1*

*JOB NUMBER:* 31169

*Scale*
*1:10*



*0.00*

Dry, light orange brown, medium dense, coarse grained <u>gravelly SAND</u> – (Fill). Granite derived fill.

*0.20*

Moderately weathered, olive grey to grey, closely to medium jointed, laminated, very soft rock <u>SHALE</u> – (Pietermaritzburg Formation).

*0.7* ●

*0.70*

Moderately weathered, dark grey stained orange on joints, medium to widely jointed, laminated soft rock <u>SHALE</u> – (Pietermaritzburg Formation).

Bulk IMC

1

*1.1m* ●

*1.10*

NOTES

1) Refusal of machine on soft rock shale

2) Bulk disturbed sample obtained from 0.7 – 1.1m

*Smpl.* *Depth.*

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC

*TYPE SET BY : M.C*
*SETUP FILE : DMPSP.SET*

*INCLINATION :*
*DIAM :* NA
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*DATE :* 04.11.2015

*DATE : 23/11/15 09:48*
*TEXT : ..C:\DOTIN\SPMASTER.DOC*

*ELEVATION :*
*X-COORD :*
*Y-COORD :*

*HOLE No:* IP3

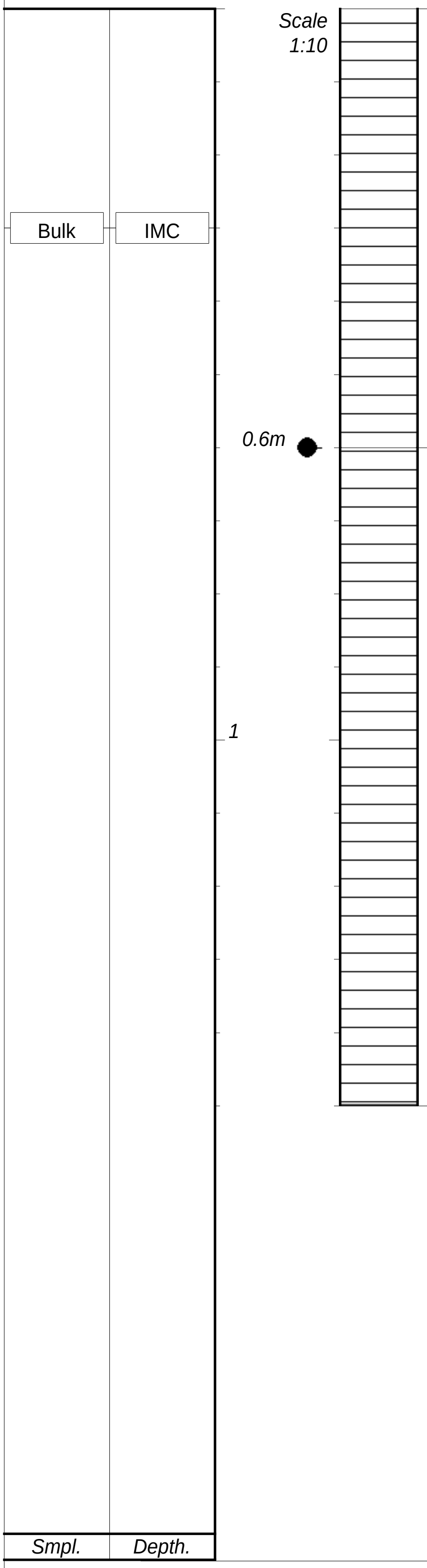
DRENNAN MAUD (PTY) LTD

Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* IP4
*Sheet 1 of 1*

*JOB NUMBER:* 31169



*0.00*

Moderately weathered, dark grey, closely to medium jointed, laminated very soft rock SHALE – (Pietermaritzburg Formation).

*0.60*

Moderately weathered, dark grey stained yellow brown to orange on joints, medium to widely jointed, laminated soft rock SHALE – (Pietermaritzburg Formation).

*1.50*

NOTES

1) Refusal of machine on soft rock shale

2) Bulk disturbed sample obtained from 0 – 0.6m

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC

*TYPE SET BY : M.C*
*SETUP FILE : DMPSP.SET*

*INCLINATION :*
*DIAM :* NA
*DATE :* NA
*DATE :* 04.11.2015

*DATE : 23/11/15 09:48*
*TEXT : ..C:\DOTIN\SPMASTER.DOC*

*ELEVATION :*
*X-COORD :*
*Y-COORD :*

*HOLE No:* IP4

DRENNAN MAUD (PTY) LTD

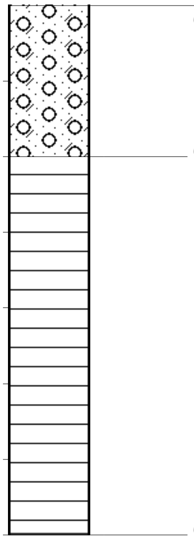
Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* PT1
*Sheet 1 of 1*

*JOB NUMBER:* 31169

*Scale*
*1:10*



| | |
|---|---|
| *0.00* | Dry, grey brown, closely packed, subangular, fine to medium shale GRAVEL in a stiff clayey SAND matrix – (Fill). |
| *0.20* | Moderately weathered, grey, closely to medium jointed, laminated, very soft rock SHALE – (Pietermaritzburg Formation). |
| *0.70* | |

NOTES

1) Percolation test conducted at 0,7m depth

| *Smpl.* | *Depth.* |
|---|---|

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC

*TYPE SET BY : M.C*
*SETUP FILE : DMPSP.SET*

*INCLINATION :*
*DIAM :* NA
*DATE :* NA
*DATE :* 04.11.2015

*DATE : 23/11/15 09:48*
*TEXT : ..C:\DOTIN\SPMASTER.DOC*

*ELEVATION :*
*X-COORD :*
*Y-COORD :*

*HOLE No:* PT1

DRENNAN MAUD (PTY) LTD

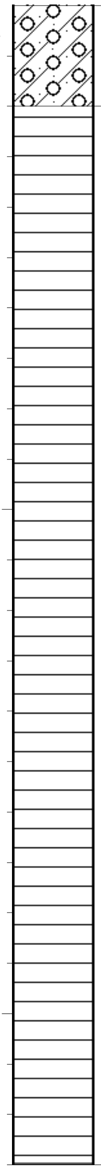
Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* EXP1
*Sheet 1 of 1*

*JOB NUMBER:* 31169

*Scale*
*1:15*



*0.00*

Slightly moist, grey brown, closely packed, relatively dense, coarse, angular shale GRAVEL in a sandy CLAY matrix – (Fill).

*0.20*

Moderately weathered, dark grey stained light orange brown on joints, medium to widely jointed, laminated soft rock SHALE – (Pietermaritzburg Formation). Appears to have been removed from pipe trench using excavator- no indication of hard breaking (chiselling) having been required.

*2.30*

NOTES

1) Man-made exposure excavated for installing manhole to storm-water line

2) Shale bedding dip recorded at 16deg (dip) dipping toward 170 (dip direction)

3) Shale joints are oblique to highly inclined (subvertical) relative to bedding. Joints are generally tight and locally in-filled with light grey, stiff clay to about 5mm thickness.

*Smpl.* | *Depth.*

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC

*TYPE SET BY : M.C*
*SETUP FILE : DMPSP.SET*

*INCLINATION :*
*DIAM :* NA
*DATE :* NA
*DATE :* 04.11.2015

*DATE : 23/11/15 09:48*
*TEXT : ..C:\DOTIN\SPMASTER.DOC*

*ELEVATION :*
*X-COORD :*
*Y-COORD :*

*HOLE No:* EXP1

DRENNAN MAUD (PTY) LTD

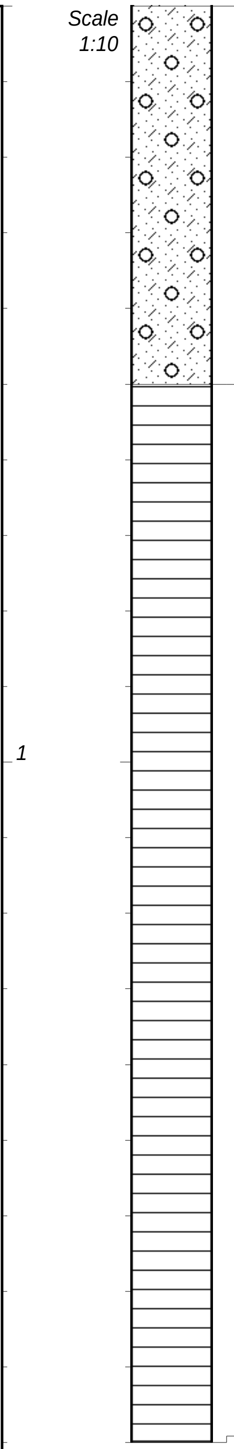
Geotechnical Engineers & Engineering Geologists

AFZELIA ENVIRONMENTAL
PTN 151 OF ERF 8, BRIDGE CITY

*HOLE No:* EXP2
*Sheet 1 of 1*

*JOB NUMBER:* 31169

*Scale*
*1:10*



| | |
|---|---|
| *0.00* | Dry to slightly moist, light grey brown, medium dense, coarse grained gravelly and clayey SAND – (Fill). |
| *0.50* | Moderately weathered, dark grey, closely to medium jointed, locally widely jointed, soft rock SHALE – (Pietermaritzburg Formation). |
| *1.90* | |

NOTES

1) Man-made exposure excavated for installing manhole to storm-water line

*Smpl.* *Depth.*

*CONTRACTOR :* NA
*MACHINE :* TLB (JCB 3CX)
*DRILLED BY :* NA
*PROFILED BY :* MC

*TYPE SET BY : M.C*
*SETUP FILE : DMPSP.SET*

*INCLINATION :*
*DIAM :* NA
*DATE :* NA
*DATE :* 04.11.2015

*DATE : 23/11/15 09:48*
*TEXT : ..C:\DOTIN\SPMASTER.DOC*

*ELEVATION :*
*X-COORD :*
*Y-COORD :*

*HOLE No:* EXP2

# Dynamic Cone Penetrometer

## Test No. : 1

**Project :** **B.P Service Station - Ptn. 151 of Erf 8 Bridge City**
**Client :** **Afzelia Environmental Consultants**
Date: 0.00
Remarks: -
Test Location: -
-
Date of Test: 04.11.2015
Depth Interval (m) : 0.3

| **Depth**<br>(m) | **Count**<br>Blows/0.3m |
|---|---|
| 0 | 0 |
| -0.3 | 79 |
| - | |
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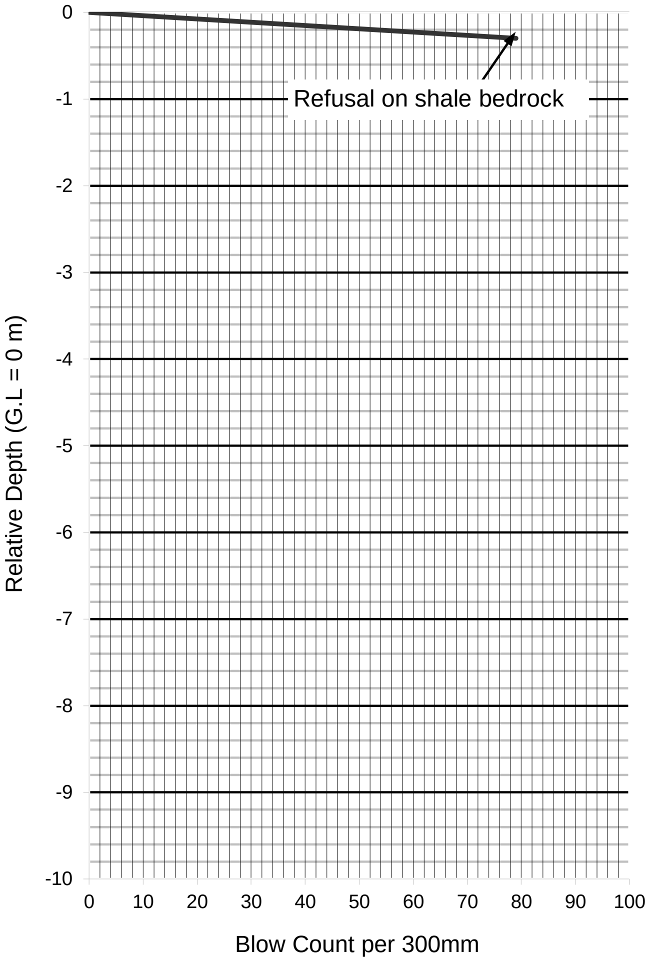
## Blow Count vs Depth



**Reference No. : 31169**

**Drennan Maud (Pty) Ltd.**

Fig. No. 2

# Dynamic Cone Penetrometer

## Test No. : 2

**Project :** **B.P Service Station - Ptn. 151 of Erf 8 Bridge City**
**Client :** **Afzelia Environmental Consultants**
Date: 0.00 Remarks: -
Test Location: - -
Date of Test: 04.11.2015 Depth Interval (m) : 0.3

| **Depth**<br>(m) | **Count**<br>Blows/0.3m |
|---|---|
| 0 | 0 |
| -0.3 | 56 |
| -0.6 | 38 |
| -0.9 | 53 |
| -1.2 | 78 |
| - | |
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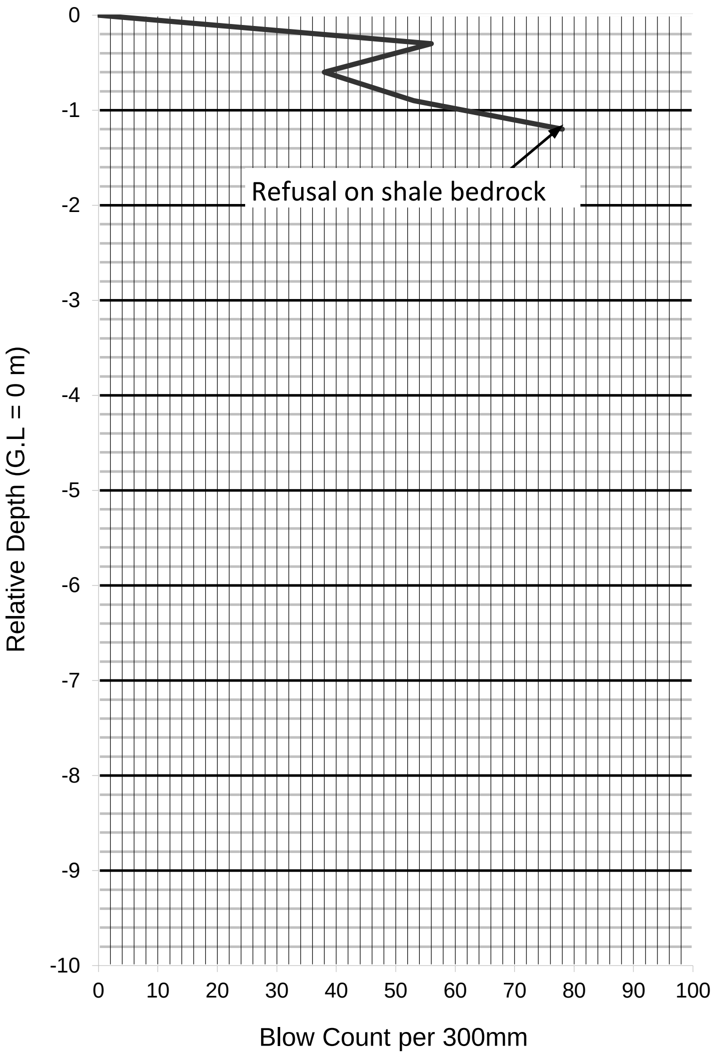
## Blow Count vs Depth



**Reference No. :** **31169**

**Drennan Maud (Pty) Ltd.**

Fig. No. 3

# Dynamic Cone Penetrometer

## Test No. : 3

**Project :** **B.P Service Station - Ptn. 151 of Erf 8 Bridge City**
**Client :** **Afzelia Environmental Consultants**

|  |  |  |  |
|---|---|---|---|
| Date: | 0.00 | Remarks: | - |
| Test Location: | - |  | - |
| Date of Test: | 04.11.2015 | Depth Interval (m) : | 0.3 |

| **Depth**<br>(m) | **Count**<br>Blows/0.3m |
|---|---|
| 0 | 0 |
| -0.3 | 68 |
| - |  |
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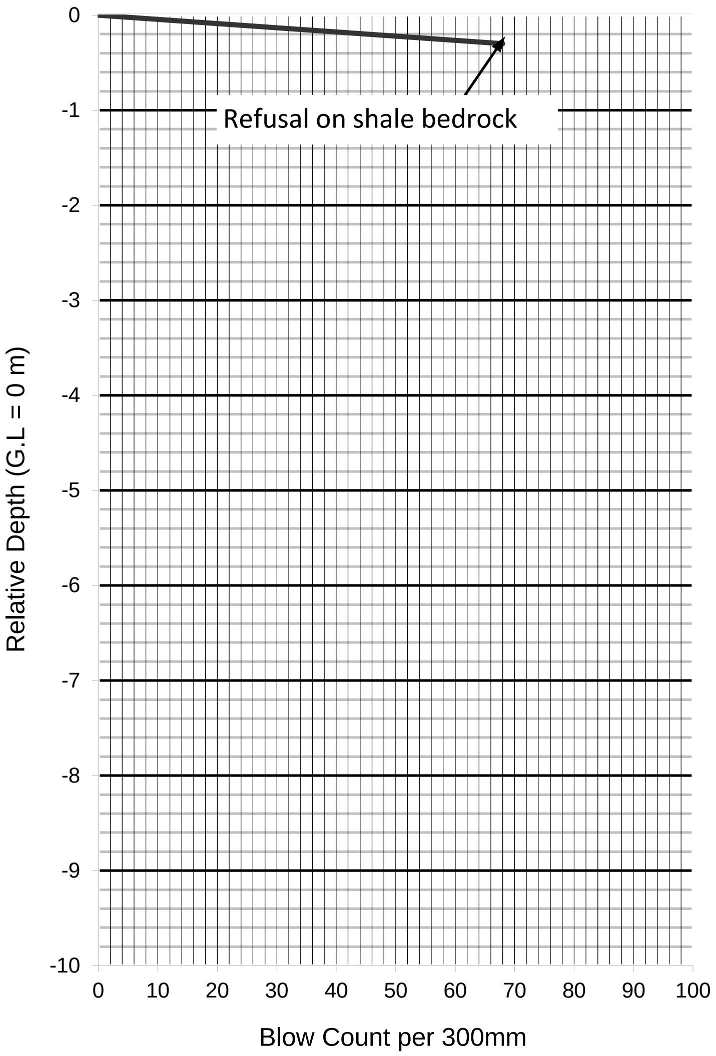
## Blow Count vs Depth



**Reference No. :** **31169**

**Drennan Maud (Pty) Ltd.**

Fig. No. 4

# Dynamic Cone Penetrometer

## Test No. : 4

**Project :** **B.P Service Station - Ptn. 151 of Erf 8 Bridge City**
**Client :** **Afzelia Environmental Consultants**
Date: 0.00 Remarks: -
Test Location: - -
Date of Test: 04.11.2015 Depth Interval (m) : 0.3

| **Depth**<br>(m) | **Count**<br>Blows/0.3m |
|---|---|
| 0 | 0 |
| -0.3 | 100 |
| - | |
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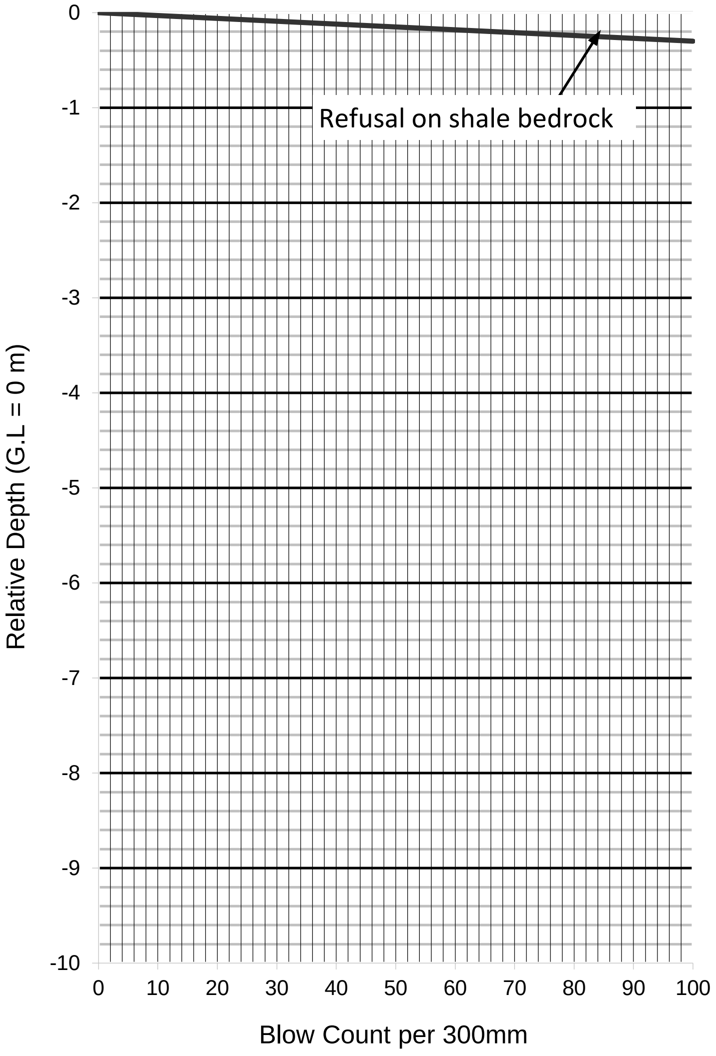
## Blow Count vs Depth



**Reference No. :** **31169**

**Drennan Maud (Pty) Ltd.**

Fig. No. 5

# Laboratory Test Summary

**THEKWINI SOILS LAB. CC**

**Job Description:** Bridge City - Ref. 31169
**Job no.:** 7935
**Date:** 24-11-2015

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Lab no. | | | 11039 | 11040 | 11041 | | | | | | | | |
| Location | | | IP 2 | IP 3 | IP 4 | | | | | | | | |
| Depth | | | 0.0 - 0.80 | 0.70 - 1.10 | 0.0 - 0.60 | | | | | | | | |
| Description | | | - | - | - | | | | | | | | |
| | | | - | - | - | | | | | | | | |
| Binder Material | | | - | - | - | | | | | | | | |
| Particle Size (mm) | 75 | Cumulative % Passing | | 73 | 79 | | | | | | | | |
| | 53 | | | 61 | 67 | | | | | | | | |
| | 37.5 | | | 50 | 54 | | | | | | | | |
| | 26.5 | | | 41 | 38 | | | | | | | | |
| | 19 | | | 33 | 32 | | | | | | | | |
| | 13.2 | | 100 | 32 | 32 | | | | | | | | |
| | 9.5 | | 99 | 29 | 29 | | | | | | | | |
| | 4.75 | | 98 | 21 | 24 | | | | | | | | |
| | 2 | | 96 | 15 | 18 | | | | | | | | |
| | 0.425 | | 91 | 9 | 13 | | | | | | | | |
| | 0.25 | | 89 | 8 | 12 | | | | | | | | |
| | 0.15 | | 88 | 8 | 11 | | | | | | | | |
| | 0.075 | | 86 | 7 | 9 | | | | | | | | |
| Hydrometer | 0.05 | % Passing | 84 | 7 | 9 | | | | | | | | |
| | 0.02 | | 78 | 5 | 7 | | | | | | | | |
| | 0.005 | | 67 | 4 | 5 | | | | | | | | |
| | 0.002 | | 59 | 3 | 3 | | | | | | | | |
| Soil Mortar | Coarse Sand <2.0 >0.425mm | % Passing | 5.9 | 38.0 | 29.8 | | | | | | | | |
| | Fine Sand <0.425>0.05mm | | 14.9 | 57.7 | 63.9 | | | | | | | | |
| | Silt <0.05 >0.005 | | 15.9 | 1.9 | 3.0 | | | | | | | | |
| | Clay <0.005 | | 63.4 | 2.4 | 3.4 | | | | | | | | |
| Atterberg Limits | Liquid Limit % (m/m) | | 49 | 24.2 | 25.1 | | | | | | | | |
| | Plasticity Index | | 22.7 | 6.8 | 8.2 | | | | | | | | |
| | Linear Shrinkage % | | 10.7 | 4.7 | 6 | | | | | | | | |
| | Natural MC % | | - | - | - | | | | | | | | |
| Mod AASHTO Density | Dry Density kg/m$^3$ | | 1680 | 1940 | 1943 | | | | | | | | |
| | OMC % | | 16.9 | 10.2 | 10.1 | | | | | | | | |
| CBR | 100% MDD | | 2 | 4.4 | 4.6 | | | | | | | | |
| | 98% | | 1.6 | 4.2 | 4.2 | | | | | | | | |
| | 95% | | 1.2 | 3.8 | 3.4 | | | | | | | | |
| | 93% (Inferred) * | | 1 | 3 | 3 | | | | | | | | |
| | 90% | | 0.5 | 1.8 | 2.5 | | | | | | | | |
| | CBR Swell (%) | | 5.93 | 2.50 | 3.07 | | | | | | | | |
| AASHTO Soil Classification * | | | A - 7 - 6 (22) | A - 2 - 4 (0) | A - 2 - 4 (0) | | | | | | | | |
| Grading Modulus | | | 0.27 | 2.69 | 2.60 | | | | | | | | |
| TRH 14 (1985) * | | | >G10 | >G10 | >G10 | | | | | | | | |

Signature: ...........................
Title: ...........................

# TEST REPORT

## MATERIALS ANALYSIS

**THEKWINI SOILS LAB. CC**

**Project:** Bridge City - Ref. 31169

**Ref no.:** 7935 **Lab no.:** 11039 **Borehole/Pit no.:** IP 2 **Fig no.:** -

**Description:** -

**Depth:** 0.0 - 0.80 -

Test Methods: TMH1 METHOD A1(a), A2, A3 & A4, ASTMD422

**Grading Analysis**

| Grain Size (mm) | %Passing |
|---|---|
| 75 | 100.0 |
| 53 | 100.0 |
| 37.5 | 100.0 |
| 26.5 | 100.0 |
| 19 | 100.0 |
| 13.2 | 100.0 |
| 9.5 | 99.1 |
| 4.75 | 98.1 |
| 2 | 96.3 |
| 0.425 | 90.6 |
| 0.25 | 89.2 |
| 0.15 | 88.0 |
| 0.075 | 86.3 |
| 0.05 | 84.2 |
| 0.02 | 77.9 |
| 0.005 | 67.3 |
| 0.002 | 58.9 |

**M.I.T SIZE CLASSIFICATION** *

| | | |
|---|---|---|
| Cobble% | | 0.0 |
| Gravel% | 3.7 | |
| Coarse | | 0.0 |
| Medium | | 1.6 |
| Fine | | 2.1 |
| Sand% | 11.3 | |
| Coarse | | 5.0 |
| Medium | | 2.7 |
| Fine | | 3.5 |
| Silt% | 26.1 | |
| Coarse | | 7.2 |
| Medium | | 9.8 |
| Fine | | 9.1 |
| Clay% | 58.9 | |

**PLASTICITY**

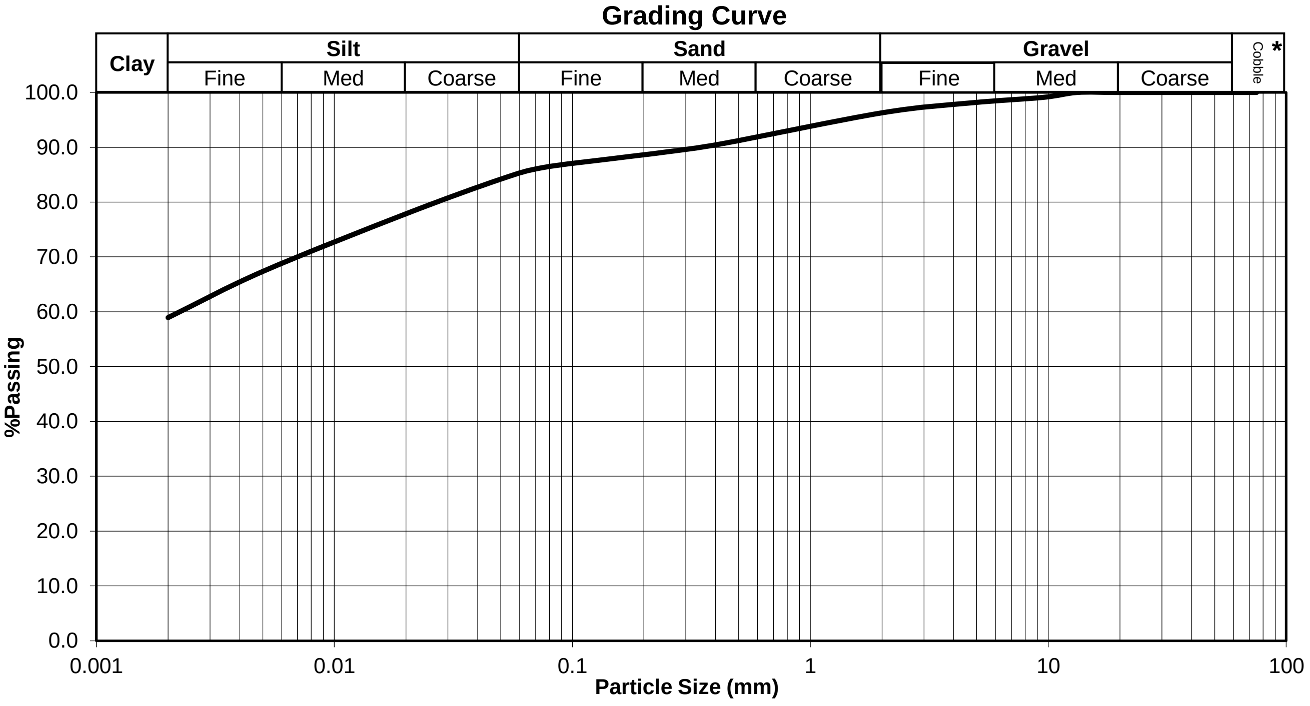
| | |
|---|---|
| Liquid Limit, % | 49 |
| Plasticity Index | 22.7 |
| Linear Shrinkage, % (L/L) | 10.7 |

**GRADING**

| | |
|---|---|
| D10 Size (mm) | <0.002 |
| Uniformity Coefficient | * |
| Grading Modulus | 0.27 |

**CLASSIFICATION** *

| | |
|---|---|
| Potential Expansiveness | Low |
| Group Index | 22 |
| AASHTO Soil Classification | A - 7 - 6 |
| Unified Classification | CL or OL |



Ref no.: 7935 Fig no.: -

* Information marked with an asterisk is outside the scope of Accreditation.
The results only relate to the samples tested.
The report may not be reproduced except in full.

# TEST REPORT

# MATERIALS ANALYSIS

**THEKWINI SOILS LAB. CC**

**Project:** Bridge City - Ref. 31169

**Ref no.:** 7935 **Lab no.:** 11040 **Borehole/Pit no.:** IP 3 **Fig no.:** -

**Description:** -

-

**Depth:** 0.70 - 1.10

Test Methods: TMH1 METHOD A1(a), A2, A3 & A4, ASTMD422

| **Grading Analysis** | |
|---|---|
| Grain Size (mm) | %Passing |
| 75 | 72.7 |
| 53 | 60.9 |
| 37.5 | 49.6 |
| 26.5 | 40.5 |
| 19 | 33.2 |
| 13.2 | 32.0 |
| 9.5 | 28.9 |
| 4.75 | 21.3 |
| 2 | 14.5 |
| 0.425 | 9.0 |
| 0.25 | 8.3 |
| 0.15 | 7.9 |
| 0.075 | 7.4 |
| 0.05 | 6.9 |
| 0.02 | 5.3 |
| 0.005 | 3.9 |
| 0.002 | 2.5 |

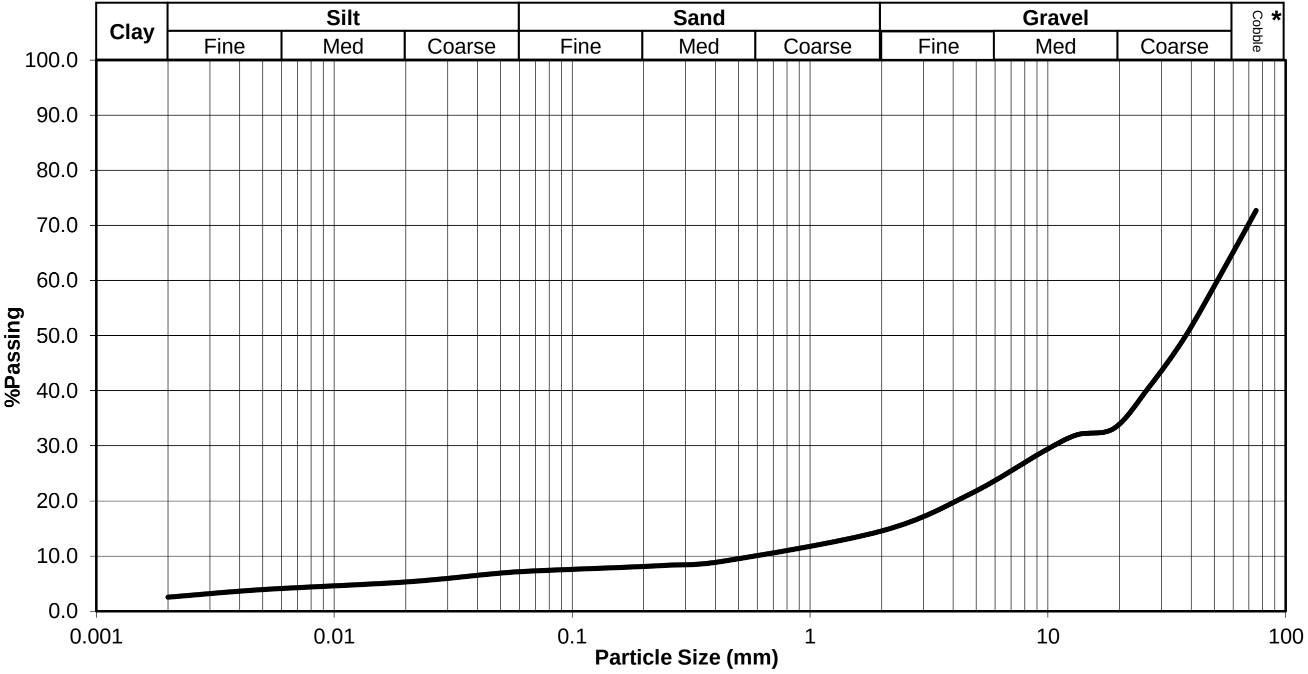
| **M.I.T SIZE CLASSIFICATION** | | * |
|---|---|---|
| Cobble% | | 35.3 |
| Gravel% | 50.1 | |
| Coarse | | 30.5 |
| Medium | | 10.8 |
| Fine | | 8.8 |
| Sand% | 7.5 | |
| Coarse | | 4.9 |
| Medium | | 1.5 |
| Fine | | 1.0 |
| Silt% | 4.6 | |
| Coarse | | 1.8 |
| Medium | | 1.3 |
| Fine | | 1.5 |
| Clay% | 2.5 | |

| **PLASTICITY** | |
|---|---|
| Liquid Limit | 24.2 |
| Plasticity Index | 6.8 |
| Linear Shrinkage | 4.7 |

| **GRADING** | |
|---|---|
| D10 Size (mm) | 0.56 |
| Uniformity Coefficient | 92.12 |
| Grading Modulus | 2.69 |

| **CLASSIFICATION** | * |
|---|---|
| Potential Expansiveness | Low |
| Group Index | 0 |
| AASHTO Soil Classification | A - 2 - 4 |
| Unified Classification | (GP-GW) |

**Grading Curve**



Ref no.: 7935 Fig no.: -

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# TEST REPORT

## MATERIALS ANALYSIS

**THEKWINI SOILS LAB. CC**

**Project:** Bridge City - Ref. 31169

**Ref no.:** 7935 **Lab no.:** 11041 **Borehole/Pit no.:** IP 4 **Fig no.:** -

**Description:** -

**Depth:** 0.0 - 0.60 -

Test Methods: TMH1 METHOD A1(a), A2, A3 & A4, ASTMD422

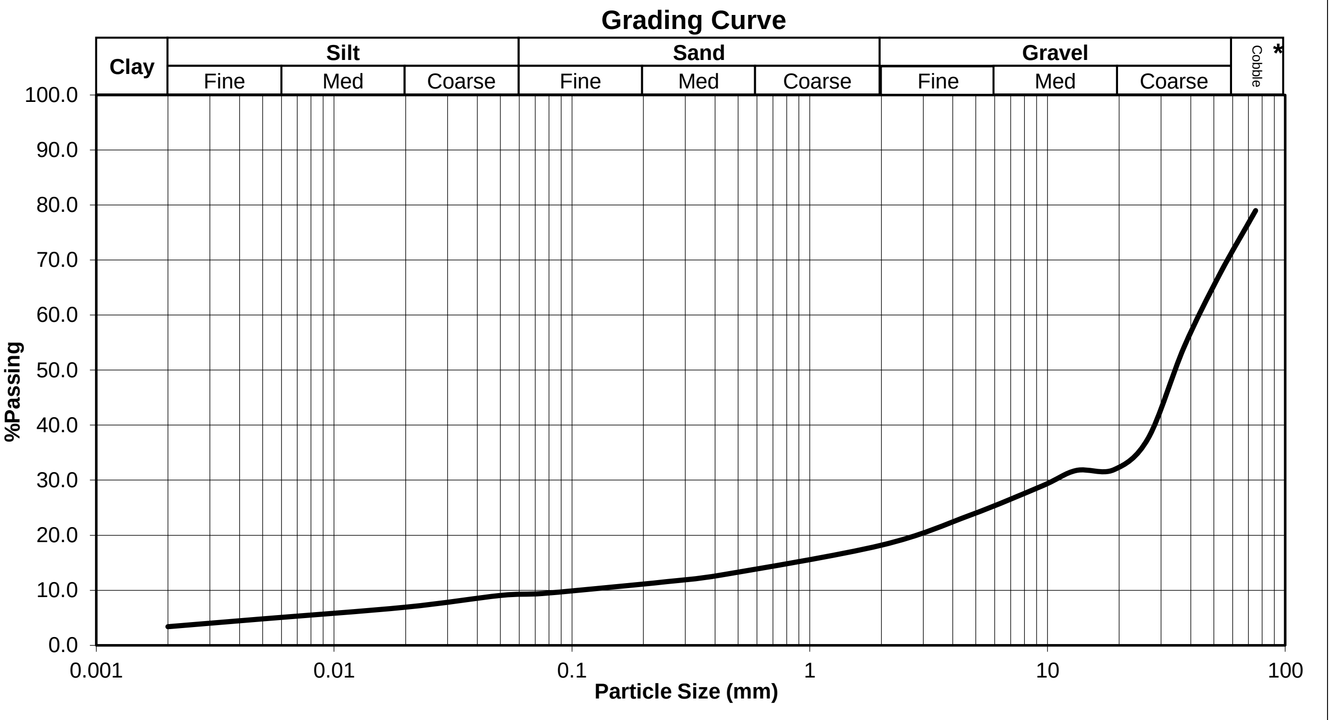
| **Grading Analysis** | |
|---|---|
| Grain Size (mm) | %Passing |
| 75 | 79.0 |
| 53 | 67.4 |
| 37.5 | 54.2 |
| 26.5 | 37.6 |
| 19 | 31.9 |
| 13.2 | 31.8 |
| 9.5 | 29.0 |
| 4.75 | 23.7 |
| 2 | 18.2 |
| 0.425 | 12.8 |
| 0.25 | 11.6 |
| 0.15 | 10.6 |
| 0.075 | 9.4 |
| 0.05 | 9.0 |
| 0.02 | 6.9 |
| 0.005 | 4.8 |
| 0.002 | 3.4 |

| **M.I.T SIZE CLASSIFICATION** | | * |
|---|---|---|
| Cobble% | | 28.9 |
| Gravel% | 52.9 | |
| Coarse | | 38.5 |
| Medium | | 7.6 |
| Fine | | 6.9 |
| Sand% | 9.0 | |
| Coarse | | 4.8 |
| Medium | | 2.3 |
| Fine | | 1.9 |
| Silt% | 5.8 | |
| Coarse | | 2.3 |
| Medium | | 2.0 |
| Fine | | 1.6 |
| Clay% | 3.4 | |

| **PLASTICITY** | |
|---|---|
| Liquid Limit | 25.1 |
| Plasticity Index | 8.2 |
| Linear Shrinkage | 6 |

| **GRADING** | |
|---|---|
| D10 Size (mm) | 0.11 |
| Uniformity Coefficient | >99 |
| Grading Modulus | 2.60 |

| **CLASSIFICATION** | * |
|---|---|
| Potential Expansiveness | Low |
| Group Index | 0 |
| AASHTO Soil Classification | A - 2 - 4 |
| Unified Classification | GP - GC |



Ref no.: 7935 Fig no.: -

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The results only relate to the samples tested.
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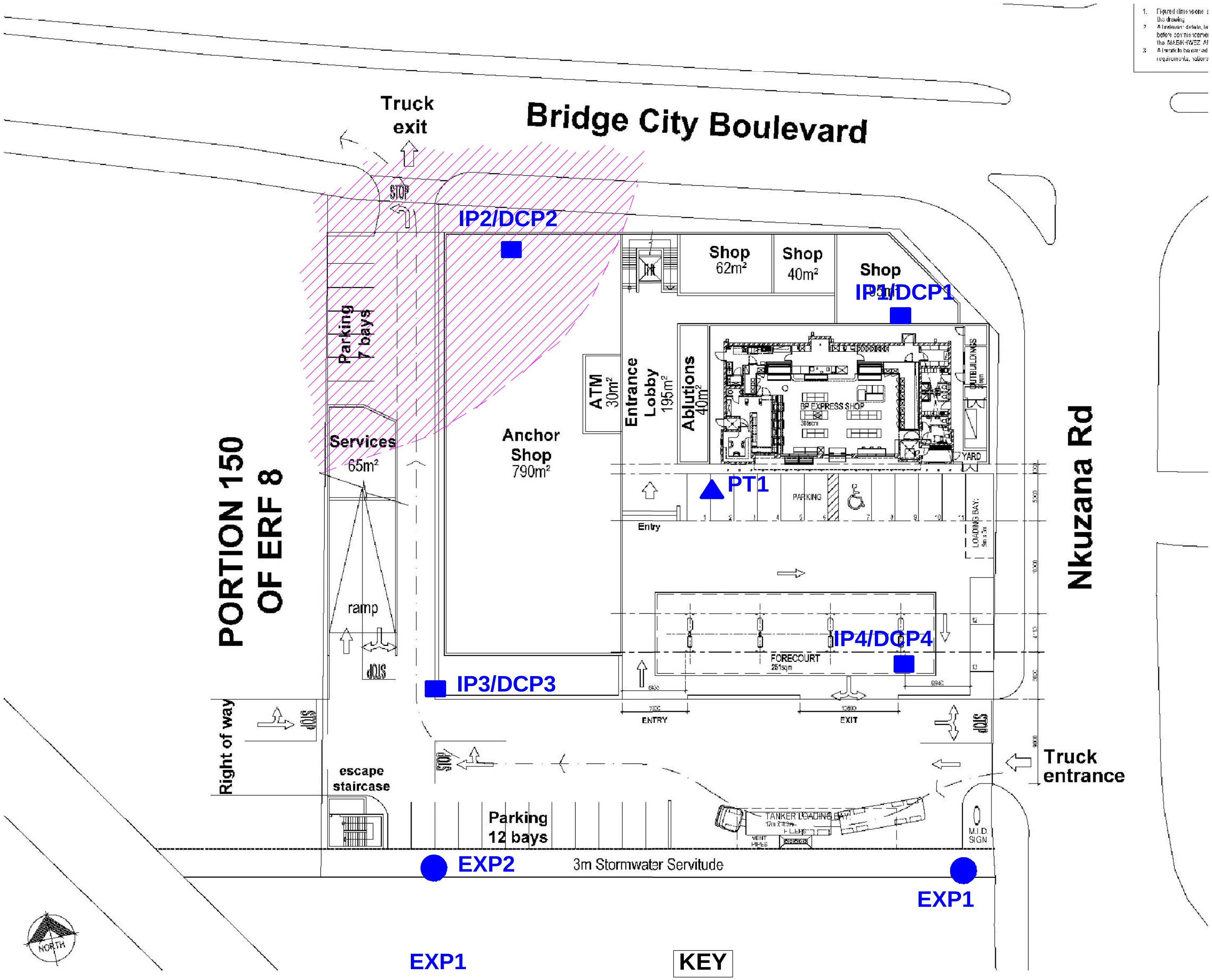

**IP1/DCP1**

■ **APPROX. POSITION OF INSPECTION PITS/ DYNAMIC CONE PENETROMETER TESTS**

**EXP1**

● **APPROX. POSITION OF EXPOSURES**

**PT1**

▲ **APPROX. POSITION OF PERCOLATION TEST**

**KEY**

▨ **BEDROCK OVERLAIN BY UP TO 1,5m STIFF CLAY SUBSOIL**

**DRENNAN MAUD (PTY) LTD.**

Geotechnical Engineers and
Engineering Geologists

DESIGNED : M.C.
DRAWN : S.P.
DATE : 12/11/2015
SCALE : N.T.S.
CHECKED :

**GEOTECHNICAL INVESTIGATION**
**BP PORTION 151 OF ERF 8 BRIDGE CITY**

REF. NO. 31169

FIG. NO. 1