ETHEKWINI WATER AND SANITATION

ENGINEERING

WASTEWATER: DESIGN BRANCH

PROJECT NO. Y6652: LANSDOWNE ROAD SEWAGE PUMP STATION

PRELIMINARY DESIGN REPORT

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Checked by:		

Contents

1. Purpose

The purpose of the report is to document the assumptions and sources of data used for the design of the Lansdowne Road sewage pump station.

It should be noted that the following reports were done and can be found in the Appendices:

- *EThekwini Municipality – Water & Sanitation: Wastewater Planning Study: Decommissioning of Pump Stations* (2017) done by Hatch Ltd.
- *Project No. Y6652 Lansdowne Sewage Pump Station Replacement: Prefeasibility for Replacing Existing System with Archimedean Screw Pumps (March 2019)* done by H. Joseph (EWS)
- *Project No. Y6652 Lansdowne Road Sewage Pump Station: Basic Life Cycle Cost Analysis Report (April 2019)* done by H. Joseph (EWS)

This is a design report for upgrading Lansdowne Road Sewage Pump Station by replacing the existing pumps with archimedean screw pumps.

2. Introduction and Background

The existing Lansdowne Road Sewage Pump Station was constructed in the early 1960s and is over 60 years old. Operational and maintenance tasks have become challenging to keep the pump station functioning. Thus eThekwini Water and Sanitation (EWS) is seeking to replace the current pumps and upgrade the pump station to the specified EWS standard with a suitable system that would be effective and efficient in the long-term.

An environmental enquiry for the project was submitted to Environmental Planning and Climate Protection Department (EPCPD), it was concluded on the letter dated the 1st of February 2018 that no Environmental Impact Assessment (EIA) approval or Water Use Licence Application (WULA) as per the National Environmental Management Act no. 107 of 1998 and the National Water Act no. 36 of 1998 respectively is required.

3. Location and Geology

3.1 Location

The pump station site falls under Ward 75 and is located in the Mobeni East Industrial Area, approximately 9km south west of Durban CBD as the crow flies.

The site can be accessed as shown in Figure 1 via Quality Street & Lansdowne Road with co-ordinates 29°55’47.11” S, 30°58’29.99” E at the intersection of Quality Street and 29°55’43.37” S, 30°58’35.68” E at Lansdowne Road entrance to pump station road.

The pump station site is bound by a road embankment to the east and south which supports Quality Street, a factory/warehouse (Roadworthy Mobeni) to the north and the Amanzimyama Canal to the west. The earth embankment along the eastern boundary of the site pinches out towards a concrete retaining wall positioned along the southern boundary of the site – the retaining wall is supporting the section of Quality street.

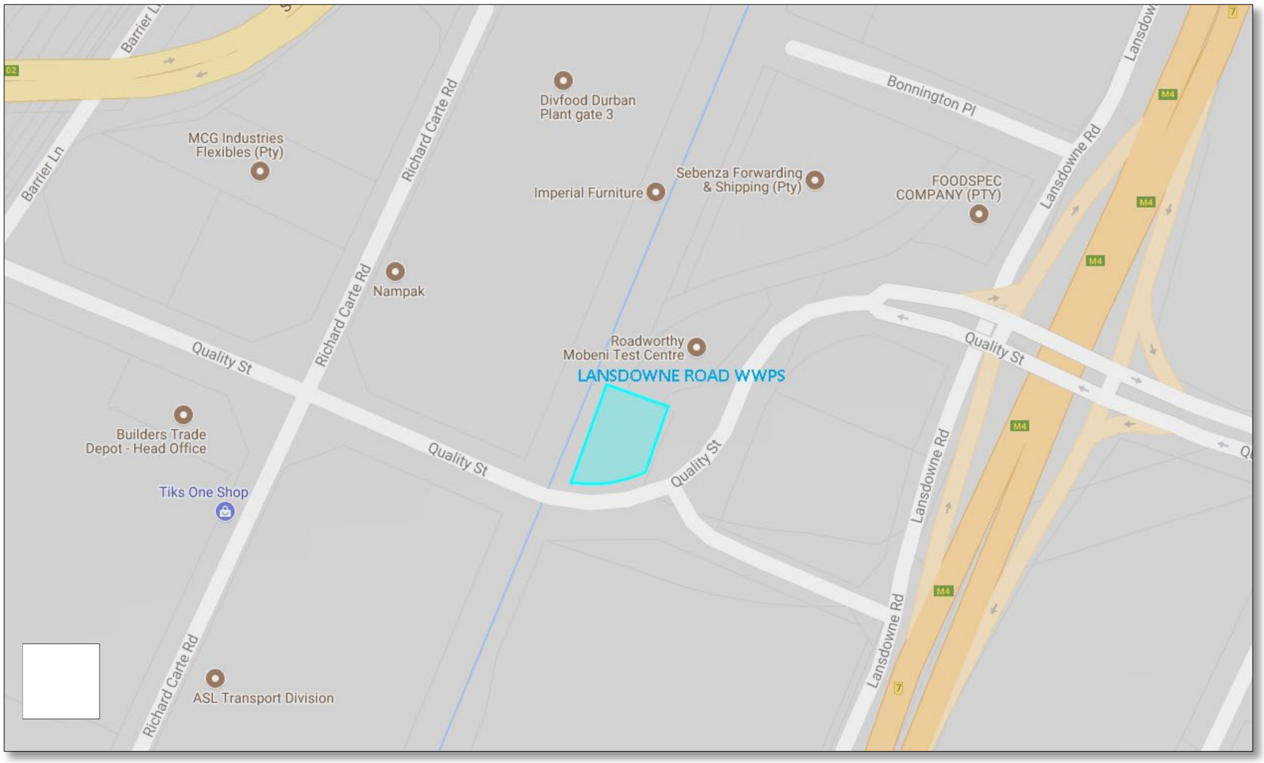


Figure 1: Locality of the Lansdowne Road Sewage Pump Station

3.2 Topography

The pump station site is relatively level across its entire area and is elevated at approximately 10 to 11 meters above mean sea level (amsl).

3.2.1 Site constraints affecting the project

- An existing 8m high retaining wall at the southern boundary of the site
- A wide intersection at the junction of Quality Street
- A 0.5m servitude containing a Sasol methane gas steel pipeline
- A 300mm Ø mPVC watermain runs along the route of the Sasol pipeline
- Two (2) rising mains along Quality Street
- Storm water pipes along Quality Street

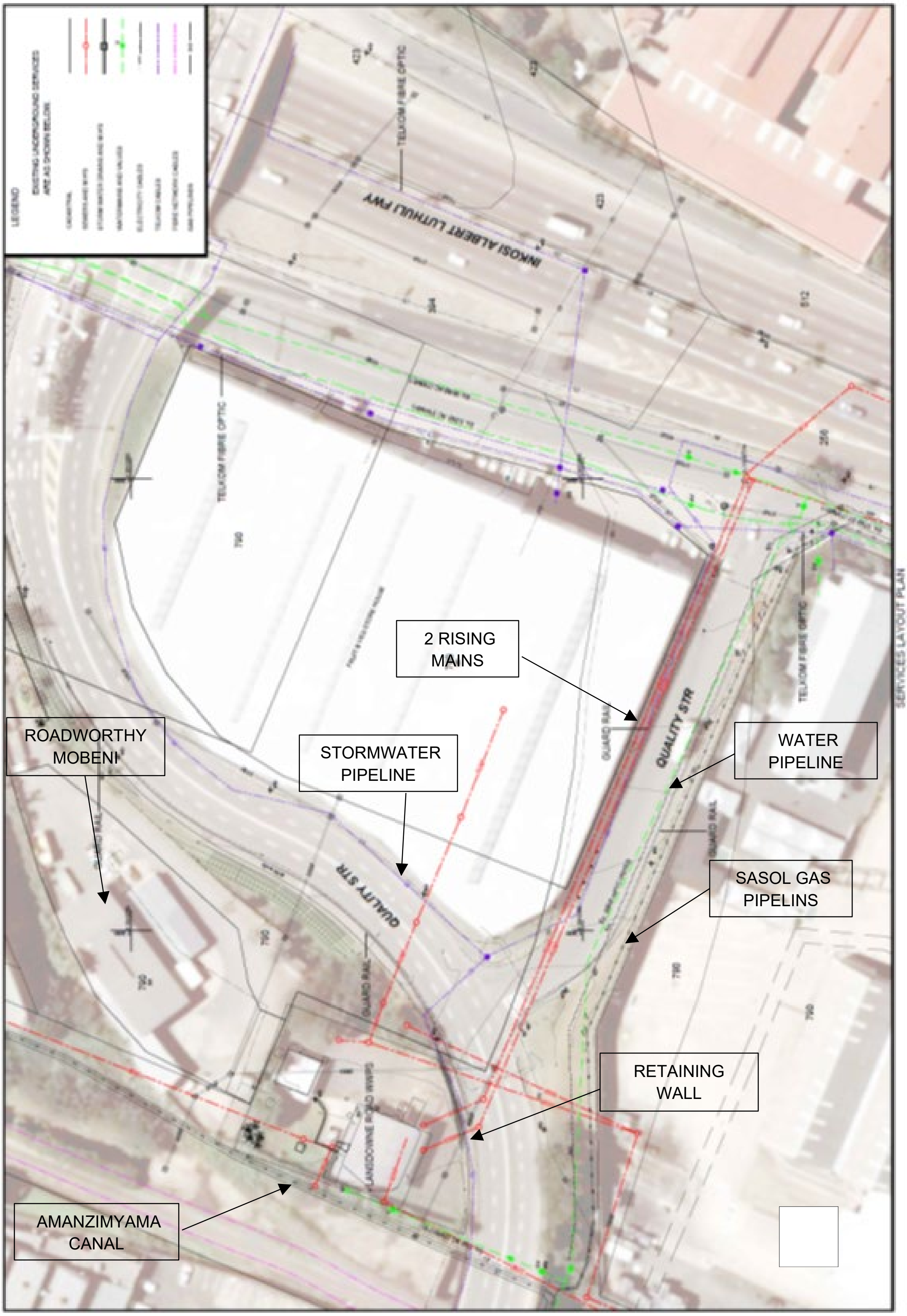


Figure 2: Site Plan

3.3 Geology

A geotechnical investigation was conducted by Drennan Maud (Pty) Ltd in September 2019. The findings of the investigation can be found in the report titled *Ref. 32742 New Pump-Station and Gravity Sewer Pipeline - Lansdowne Road* (refer to Appendix D).

According to the geotechnical investigation, the site and immediate surrounding area is underlain by late Quaternary alluvial and estuarine sediments which form part of the commonly known “Harbour Beds” in the Durban area. Ground water seepage is said to occur at depths ranging between 2.4 – 2.7m below existing ground level across the existing pump station facility.

The geotechnical constraints that were highlighted in the report were problem soils, excavatability/ trenchability, groundwater seepage, trench side wall stability, material suitability and settlement.

It was recommended that interlocking sheet pile shoring is used as lateral support during excavation. The design and supervision of the installation of such shoring should be carried out by an experienced Structural/Geotechnical Engineer. Groundwater seepage management should be considered during construction and post construction. Dewatering of deep excavations during construction will be required. Should piling be required for the foundation construction then additional tests will be required prior to designing the piles.

It was concluded that the proposed development of the new pump station and installation of gravity sewer line is considered feasible provided that the geotechnical considerations and recommendations provided in the geotechnical report are consulted and adhered to during the design and construction of the proposed pump station.

4. Sewage Generation

4.1 Catchment

Lansdowne Road Sewage Pump Station is situated in an industrial area. The sewer catchment to Lansdowne Pump Station is made up of 8 pump stations namely Unit Avenue, Seaward Road, Carolina Crescent, Richard Garte, Roberts Grove, River Drive, Edwin Swales Drive and Rossburgh Bus Depot as depicted in Figure 3. The flow from the aforementioned pump stations and attendant gravity flow are conveyed via the Umhlatuzana and Umbilo trunk sewers to Lansdowne Pump Station. The sewage from Lansdowne then pumps up and gravitates to the Southern Wastewater Treatment Plant (WWTP) via the Jacobs line.

Domestic and industrial flow from the Umkumbane, Upper Umbilo, Lower Umbilo, Woodlands Montclair, Upper and Lower Umhlatuzane, Rossburgh Clairwood and Mobeni as depicted in Figure 3 contribute to the total flow into Lansdowne Road Sewage Pump Station.

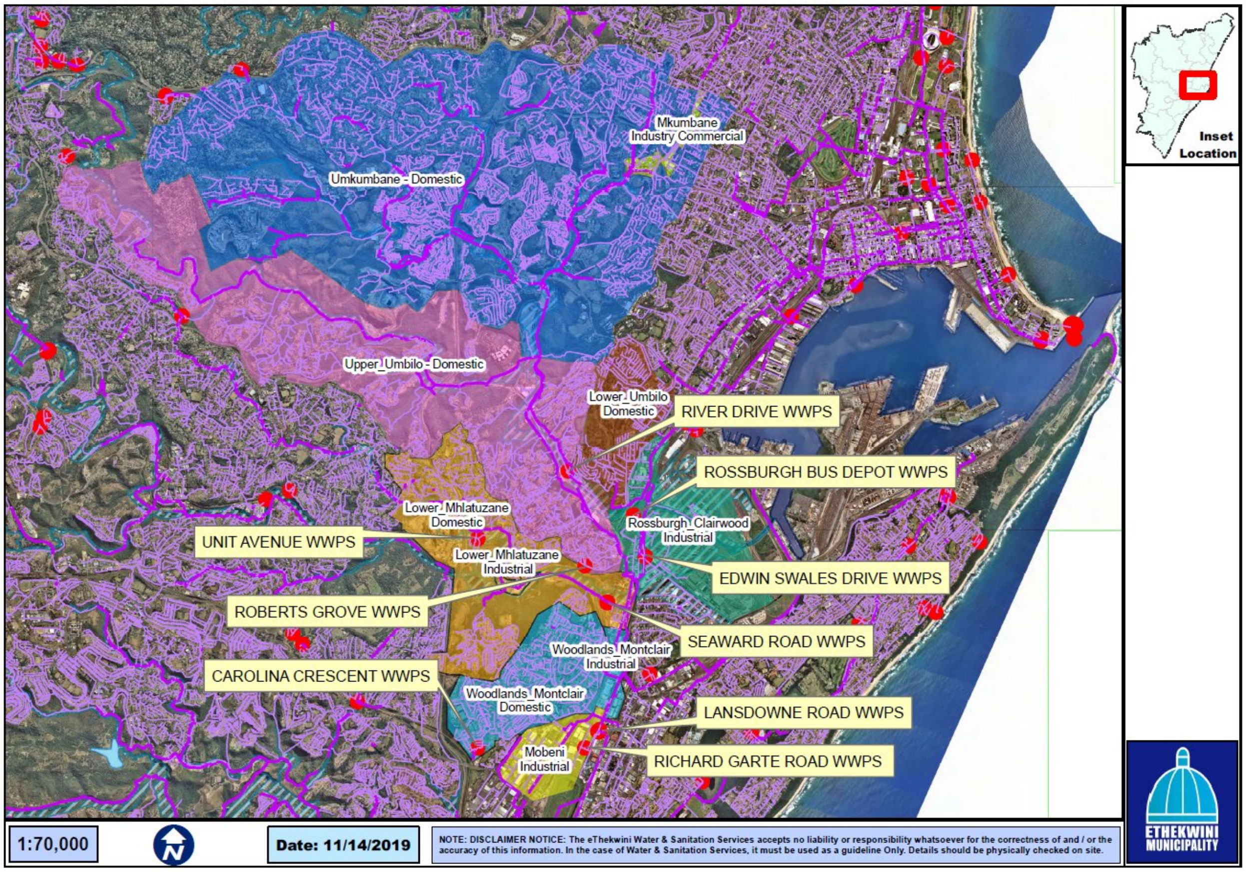


Figure 3: Lansdowne Sewer Catchment

4.2 Sewer Flow

ArcGIS was used to identify the spatial catchment of the pump station. The sewer catchment has been deemed as fully developed with very little room for future development as per EThekwini Spatial Development Framework (SDF) Report 2018/2019. Therefore, a geodatabase that merged ArcGIS and COINS to produce a spatial file with actual meter readings of water consumption was used to calculate the average sewer flow generated from the catchment. Since EWS migrated from COINS to the new RMS (June 2016), the file only had historic data for 2015/2016 therefore the values were raised by a factor of 1.5 (*as per EWS Acting Senior Manager: Planning, 2019*) to calculate the current average sewer flow of 45 Ml/day which equates to 521l/s.

The sump of a centrifugal pump station absorbs the peak flow within its storage capacity (time to fill up) which result in the pumps actually pumping 10%-20% more than the average sewer flow. Archimedean screw pumps do not require a storage sump as dry running is acceptable. The inlet to the screw pumps is generally a feeder channel instead of a sump therefore it is critical that the archimedean screw pumps are adequately sized to cater for pumping the peak flow.

Design of a pump station is based on the worst case scenario which includes the peak factor, infiltration and future development. Therefore, using a peak factor of 1.8 as per *Guidelines for*

The Design of Foul-Water Sewers, 15% infiltration and 25% for future flow, the calculated design flow yielded 1 346l/s. It was confirmed by a sump draw-down test performed by EWS, Mechanical & Electrical (M&E) branch in 2019, that the current peak flow is being handled by two pumps which pipe 1 164l/s. A 25% for future development within the catchment was added on which yielded 1 455l/s. Therefore, the design flow of 1 460l/s was used for the purposes of this study.

5. Existing Installation

5.1 Pump Station

The pump station is old and is in need of major upgrades. Mechanical rakes are non-operational thus screens have to be manually cleaned and the material lifted using a rope and bucket. No functioning ventilation system exists which is considered a health hazard. Spares for the existing Allen Gwynne pumps cannot be acquired. Thus, it made sense to relook at the design of the existing pump station and find a suitable system that would work best in the long term.

The pump station site, currently accommodates a pump station and guard hut/ablution building on the central to south western and north eastern portions of the site respectively. The fenced site is entirely paved except for the grassed north western corner of the facility.

According to the as-built drawings, the existing pump station is a round caisson having a diameter of 18.6m with a 700mm thick wall containing a section for the dry & wet well. This substructure was constructed in the 1960s.

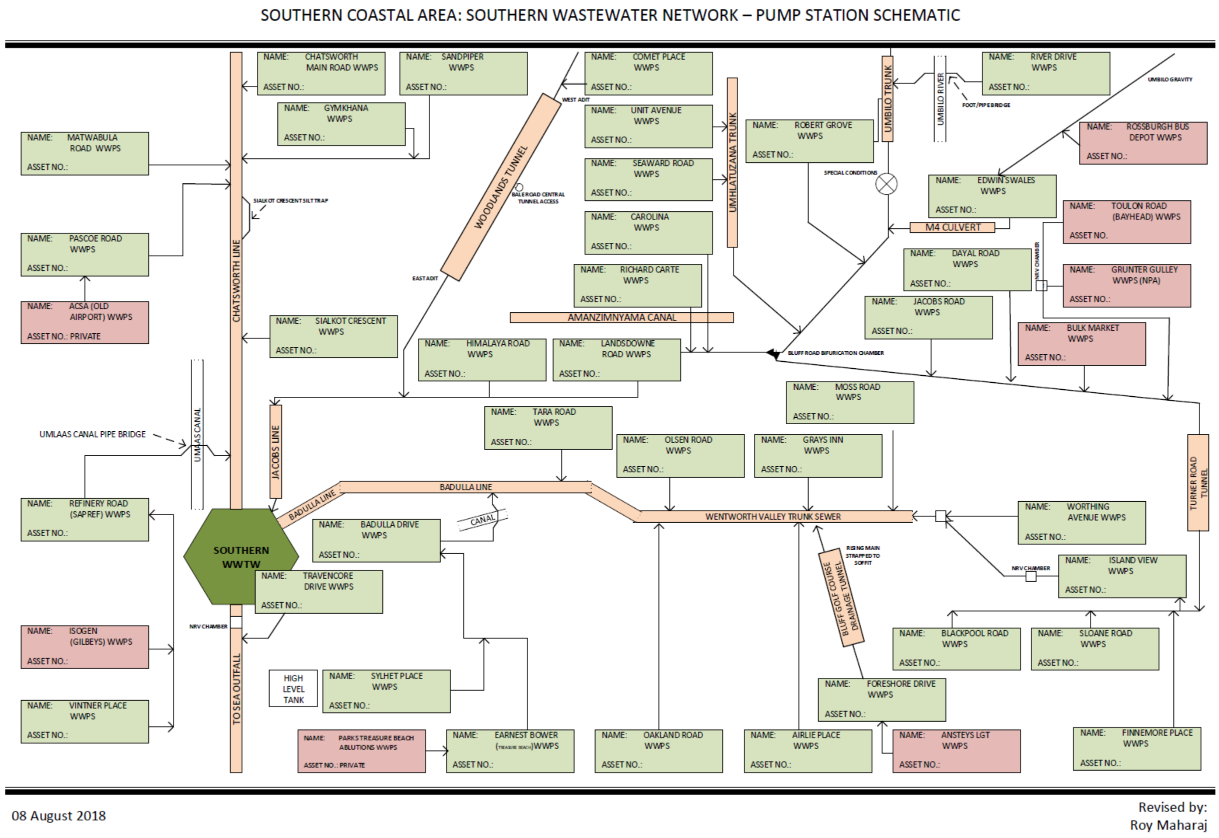
The pump station receives flow via a 57 inch /1 447mm diameter pipe, possibly concrete. The material of the inlet main will need to be verified on site. The flow travels via a channel to three (3) mechanical screens where large debris is removed before entering the wet well. However, the mechanical rakes are not operational and thus screens have to be manually cleaned and material lifted using a rope and bucket. The pump station has two (2) interconnected sumps and a dry well. Of the four (4) pumps installed, three (3) duty and one (1) standby, only two (2) pumps are operational. Each pump installed is a 132kW centrifugal pump with a capacity of 530l/s against existing head. EWS Mechanical & Electrical (M&E) branch confirmed that one (1) pump caters for the average flow and two pumps cater for the peak flow. Two (2) rising mains of cast iron with diameters 31 inch/787mm and 36 inch/914mm respectively exit the pump station and terminate at the connection to the 1 500mm diameter Calcium Aluminate Cement (CAC) trunk sewer on Lansdowne Road as per as built records.

5.2 Diversion Possibilities

It should be noted that the Amanzimyama Canal that runs behind the Lansdowne Road Pump Station is very eco sensitive. Overflow from Landsdowne Road and Richard Carte pump stations flow into the Amanzimyama Canal. Sewage flow from Richard Garte and Carolina pump stations need to be tankered during the tie-in process as flow cannot be diverted. It is possible to divert the flow from River Drive, Roberts Grove, Unit Avenue, Seaward Road, Edwin Swales, Rossburgh Bus Depot pump stations and attendant gravity flow from Umhlatuzana and Umbilo trunk sewer via the Bluff Road Bifurcation Chamber which flows through Turner Road Tunnel to the Island View pump station as shown in Figure 4.

The Bifurcation Chamber is located near the railway line behind the container terminal at the corner of South Coast Road and Bluff Road. The headgear from the chamber has been stolen however the gate is still in position.

It should be noted that localised flow from the Lansdowne Road Pump station will need to be tankered. Contributors between Lansdowne Road pump station and the Bifurcation Chamber must be considered during the shutdown.


SOUTHERN COASTAL AREA: SOUTHERN WASTEWATER NETWORK – PUMP STATION SCHEMATIC

08 August 2018

Revised by:
Roy Maharaj

Figure 4: Schematic

As per EWS M&E branch (2019), the rising main from the Island View pump station has a capacity of 400l/s. Therefore, this is the limiting factor. Four (4) pumps were initially installed at the Island View pump station however two have been removed for other uses. Each pump has a capacity of 300l/s therefore two pumps are adequate to cater for the maximum flow of 380l/s.

In light of the above diversion of flow is not possible due to the limiting capacity at the Island View Pump Station.

6. Design Inputs & Assumptions

The financial study report titled *Project No. Y6652 Lansdowne Road Sewage Pump Station: Basic Life Cycle Cost Analysis Report (April 2019)* concluded that the installation of archimedean screw pumps are financially favourable in the long term.

The invert level of the inlet is 6.00m and the invert level of the connection manhole at the end of the gravity sewer in Lansdowne Road in 10.99m as per survey done in March 2018. The peak flow of 1 460l/s will be used for the design. Archimedean screw pumps are displacement pumps thus a new gravity main would be required from the discharge outlet of the pumps. The gravity flow will be designed in accordance to Manning's formula keeping it at subcritical level. It is proposed that the concrete class of the pipe will be a 100D and will be in the order of 1 250mm to 1 500mm diameter with a 19mm sacrificial layer.

7. Proposed Installation

7.1 Pump Station

The proposed system will require a 90° take-off from the existing inlet main to a new trash rack where large objects will be screened and removed. Sewage will then be directed to the proposed storage sump downstream of the trash rack where the archimedean screw pumps will then pump the sewage for a head of approximately 6m into an outlet channel. The outlet channel will be connected to the proposed gravity main which will run along Quality street and connect into the manhole at the intersection of Quality Street and Lansdowne Road.

Table 1: Proposed Specifications for Archimedean Screw Pumps

Proposed Specifications		
Diameter of screw	2 200	mm
No of screws incl. standby	2	
Q (1 screw – 1550l/s)	1550	l/s
Pumping Head (max 7.9m)	6	m
Angle of inclination	30	°

7.2 Gravity Main

Approximately 30m of the proposed 180m long concrete gravity main will be jacked under Quality Street without the use of a sleeve. The gravity main will discharge into a manhole on the 1 500mm diameter trunk main on Lansdowne Road. The trunk main terminates at Southern Wastewater Treatment Works. The two (2) existing rising mains will be decommissioned.

7.3 Structural and Civil Works

Structural and civil works for the pump station. It was identified in the preliminary design that the Motor Control Centre (MCC) will be located within the pump station building and the trash rack and bypass will require civil works of its own. The inlet channel need to be adequately sized to accommodate the operating philosophy of the pump station. The possibility of using the existing pump station as an emergency overflow/storage needs to be investigated. Due to the location of the pump station, the existing access in all probability may not work therefore the design of a new access may be required. These items have come out of the preliminary design which must be addressed.

7.4 Mechanical Design

It was identified during the preliminary design that proper ventilation is critical for the successful operation of the pump station. Therefore, this aspect must be given adequate consideration during the design. Mechanical trash rack must be carefully selected taking into account the velocity of the flow. Odour control is crucial as this is a raw sewage pump station thus enclosing of equipment should be considered for the design. Using a cover lid over the screws will protect from heat and assist in controlling odours escaping. Proper measures need to be taken to ensure that backflow is prevented. Fire protection for equipment must be catered for in the design. Hazop study is required as per EWS M&E branch.

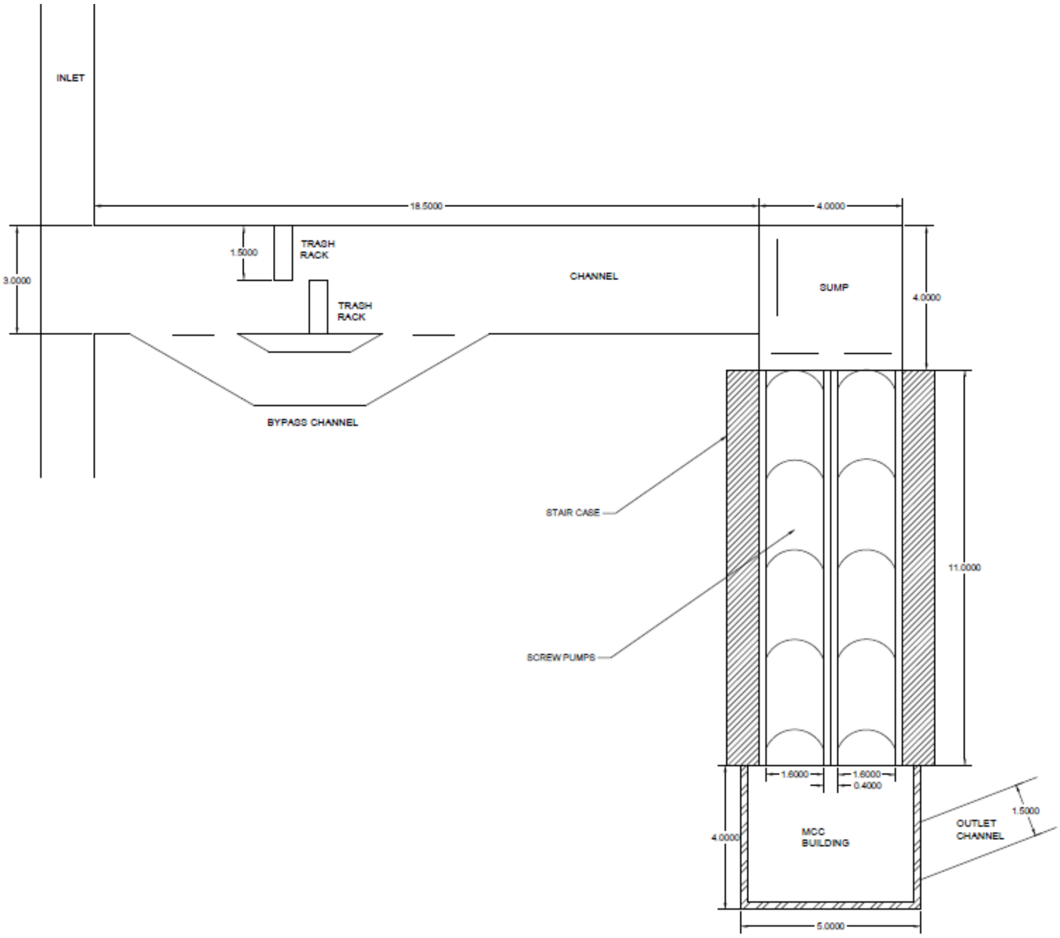
7.4 Electrical Design

The existing main power supply will be investigated for the proposed system. In all probability it should be adequate as the electrical consumption for the proposed installation will be the same or less than the consumption of the existing system. However, this will be checked in the design. Fully automated system to include installation of flow meters at the inlet and outlet which must be connected to telemetry to allow for online monitoring. Allowance for a generator and fuel storage must be considered in the design as a backup power supply.

7.5 Other General Considerations for Design

Method statement on how to do the switch over from the existing installation to the proposed installation need to be addressed during the design process otherwise it will become a problem at construction stage. Boundary wall of adjacent property need to be removed to allow for encroachment into property. Relocation of the existing transformer building is required. It should be noted that normal operation of the existing pumps is necessary during construction.

8. Proposed Arrangement



9. Financial Implications

Costs used were obtained from suppliers (2019) and a report titled *EThekwini Municipality – Water & Sanitation: Wastewater Planning Study: Decommissioning of Pump Stations* (2017) done by Hatch Ltd were inflated by 30% to relate cost to today (2019).

Table 2: Cost Estimate

DESCRIPTION	***ESTIMATED COST***
Civil	R 6 708 000.00
Mechanical	R 4 800 000.00
Electrical	R 9 139 000.00
Ventilation & Odour Control	R 1 500 000.00
Pipe Jacking & New Gravity Main	R 1 200 000.00
Sub Total	**R 23 347 000.00**
Preliminary & General (15%)	R 3 502 050.00
Engineering Fees (10%)	R 2 334 700.00

Escalation (6%)	R 2 042 862.50
TOTAL	**R31 226 612.50**

10. Programme

A summary or the proposed project showing keys aspects and their proposed dates can be seen below (see Appendix 1 for full programme). The programme will see the commencement of construction in March 2021.

Table 3. Project program

ACTIVITIES	MONTHS	START DATE	COMPLETION TIME
Survey	2	Dec-18	Feb-19
Geotechnical Investigation	4	May-19	Sep-19
Design, Drawings and Tender Document	8	Oct-19	Jun-20
Tender Process	8	Jul-20	Feb-21
Construction	30	Mar-21	Sep-23
As Built Survey	1	Oct-23	Nov-23
As Built Drawing	1	Nov-23	Dec-23

11. Conclusion and Recommendation

Appendix A: Exert from Hatch Report

Exert from Hatch Report:

5.13 Lansdowne Road

5.13.1 Aerial of site

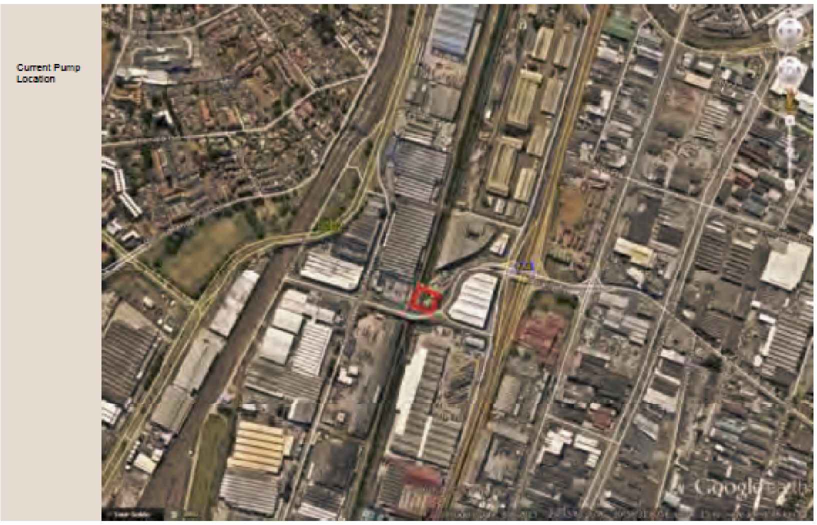


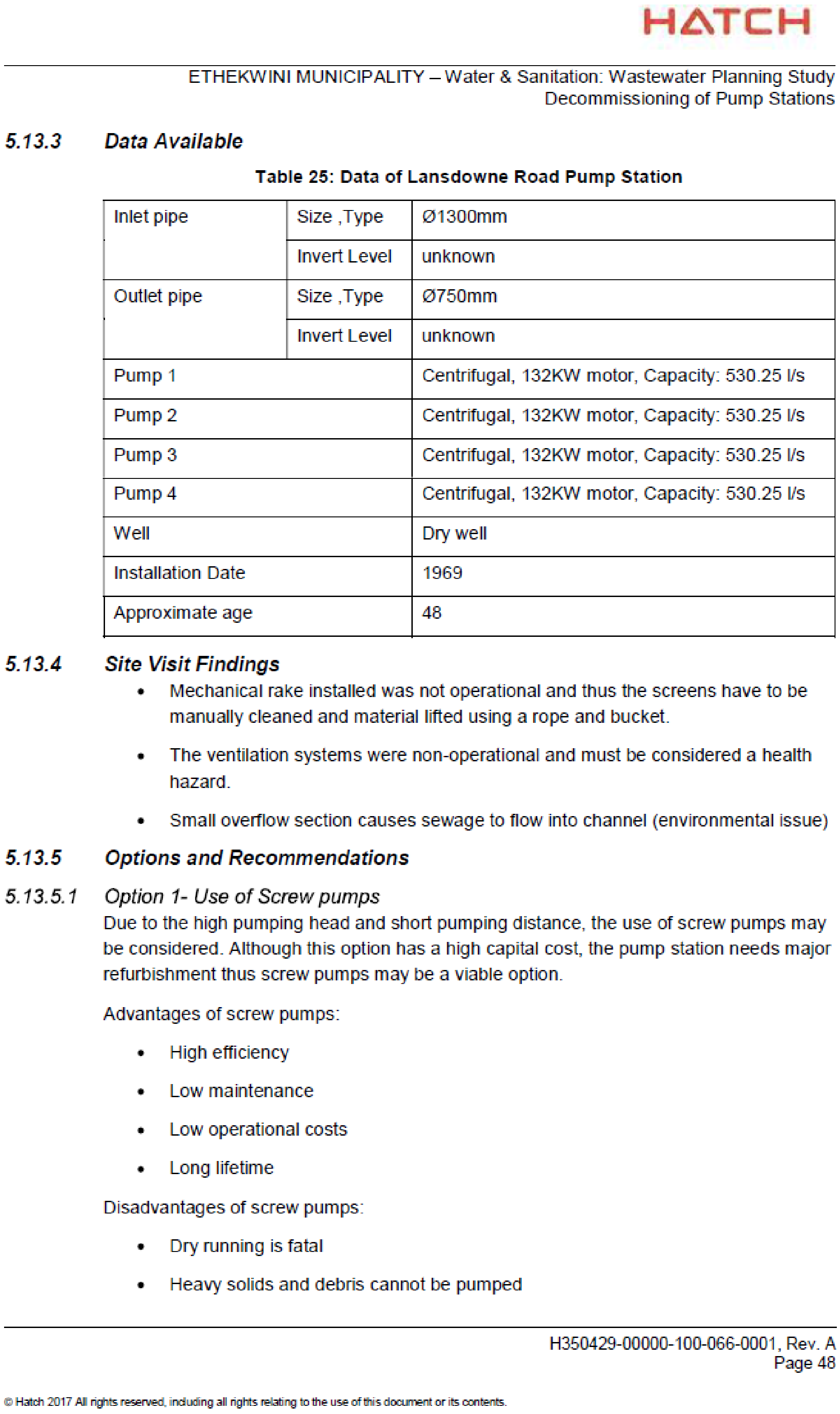
Figure 5-33: Aerial of Lansdowne Road Pump Station

5.13.2 Status Quo

The pump station is situated approximately 15mMSL and pumps into a trunk sewer which is in close proximity of the station. The trunk sewer ultimately terminates at Durban Southern WWTW.

The pump station's maintenance and operational costs are considered high. It is old and is in need of major upgrades.

The current sump is approximately 6m below ground level and the pumping head required is approximately 15m above inlet level.



5.13.3 *Data Available*

Table 25: Data of Lansdowne Road Pump Station

Inlet pipe	Size ,Type	Ø1300mm
	Invert Level	unknown
Outlet pipe	Size ,Type	Ø750mm
	Invert Level	unknown
Pump 1		Centrifugal, 132KW motor, Capacity: 530.25 l/s
Pump 2		Centrifugal, 132KW motor, Capacity: 530.25 l/s
Pump 3		Centrifugal, 132KW motor, Capacity: 530.25 l/s
Pump 4		Centrifugal, 132KW motor, Capacity: 530.25 l/s
Well		Dry well
Installation Date		1969
Approximate age		48

5.13.4 *Site Visit Findings*

- Mechanical rake installed was not operational and thus the screens have to be manually cleaned and material lifted using a rope and bucket.
- The ventilation systems were non-operational and must be considered a health hazard.
- Small overflow section causes sewage to flow into channel (environmental issue)

5.13.5 *Options and Recommendations*

5.13.5.1 *Option 1- Use of Screw pumps*

Due to the high pumping head and short pumping distance, the use of screw pumps may be considered. Although this option has a high capital cost, the pump station needs major refurbishment thus screw pumps may be a viable option.

Advantages of screw pumps:

- High efficiency
- Low maintenance
- Low operational costs
- Long lifetime

Disadvantages of screw pumps:

- Dry running is fatal
- Heavy solids and debris cannot be pumped

Current Pump Configuration

The current configuration of Lansdowne Road pump station is shown in Figure 5-34: Current configuration of Lansdowne Road Pump Station

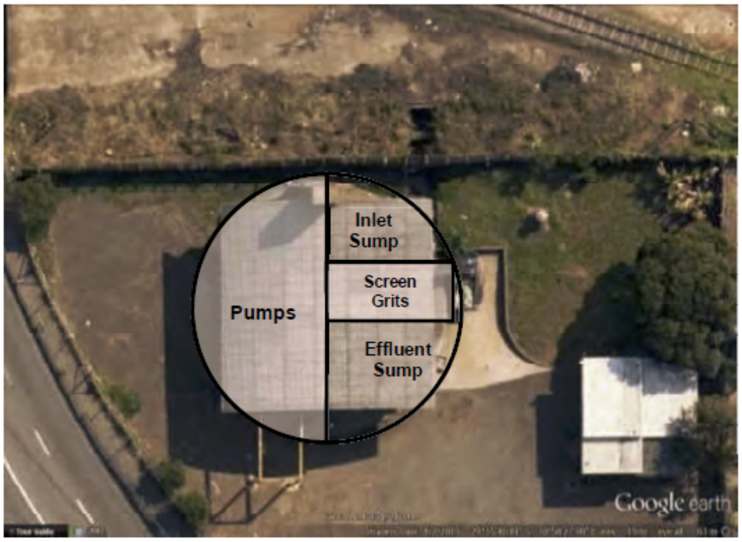


Figure 5-34: Current configuration of Lansdowne Road Pump Station

Screw Pump Configuration

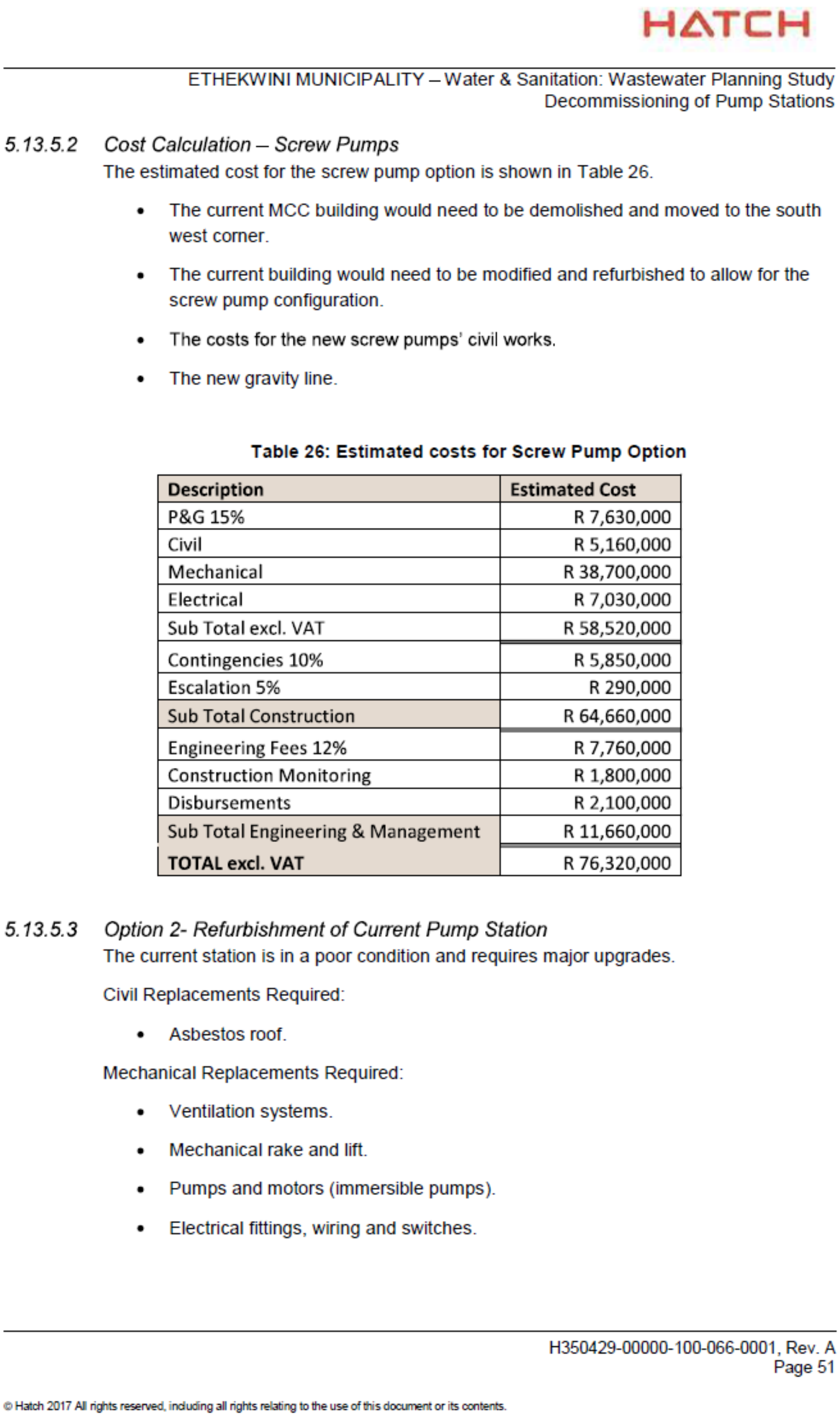
The screw pump site configuration would require the electrical building to be demolished and rebuilt in the south west corner of the site. Due to the size, configuration and depth of the current sump it is estimated that at least two screws will be required.

Screw 1 would pump from sump level to ground level and screw 2 would pump from ground level to the required height of approximately 10m above ground level. Refer to Figure 5-35: Proposed configuration of screw pumps

From screw pump 2 a new gravity line would need to be constructed to tie into the trunk sewer. This line would be approximately 160m in length as indicated in Figure 5-36: Proposed new gravity main.

Figure 5-35: Proposed configuration of screw pumps

Figure 5-36: Proposed new gravity main



HATCH

ETHEKWINI MUNICIPALITY – Water & Sanitation: Wastewater Planning Study
Decommissioning of Pump Stations

5.13.5.2 *Cost Calculation – Screw Pumps*

The estimated cost for the screw pump option is shown in Table 26.

- The current MCC building would need to be demolished and moved to the south west corner.
- The current building would need to be modified and refurbished to allow for the screw pump configuration.
- The costs for the new screw pumps' civil works.
- The new gravity line.

Table 26: Estimated costs for Screw Pump Option

Description	Estimated Cost
P&G 15%	R 7,630,000
Civil	R 5,160,000
Mechanical	R 38,700,000
Electrical	R 7,030,000
Sub Total excl. VAT	R 58,520,000
Contingencies 10%	R 5,850,000
Escalation 5%	R 290,000
Sub Total Construction	R 64,660,000
Engineering Fees 12%	R 7,760,000
Construction Monitoring	R 1,800,000
Disbursements	R 2,100,000
Sub Total Engineering & Management	R 11,660,000
TOTAL excl. VAT	R 76,320,000

5.13.5.3 *Option 2- Refurbishment of Current Pump Station*

The current station is in a poor condition and requires major upgrades.

Civil Replacements Required:

- Asbestos roof.

Mechanical Replacements Required:

- Ventilation systems.
- Mechanical rake and lift.
- Pumps and motors (immersible pumps).
- Electrical fittings, wiring and switches.

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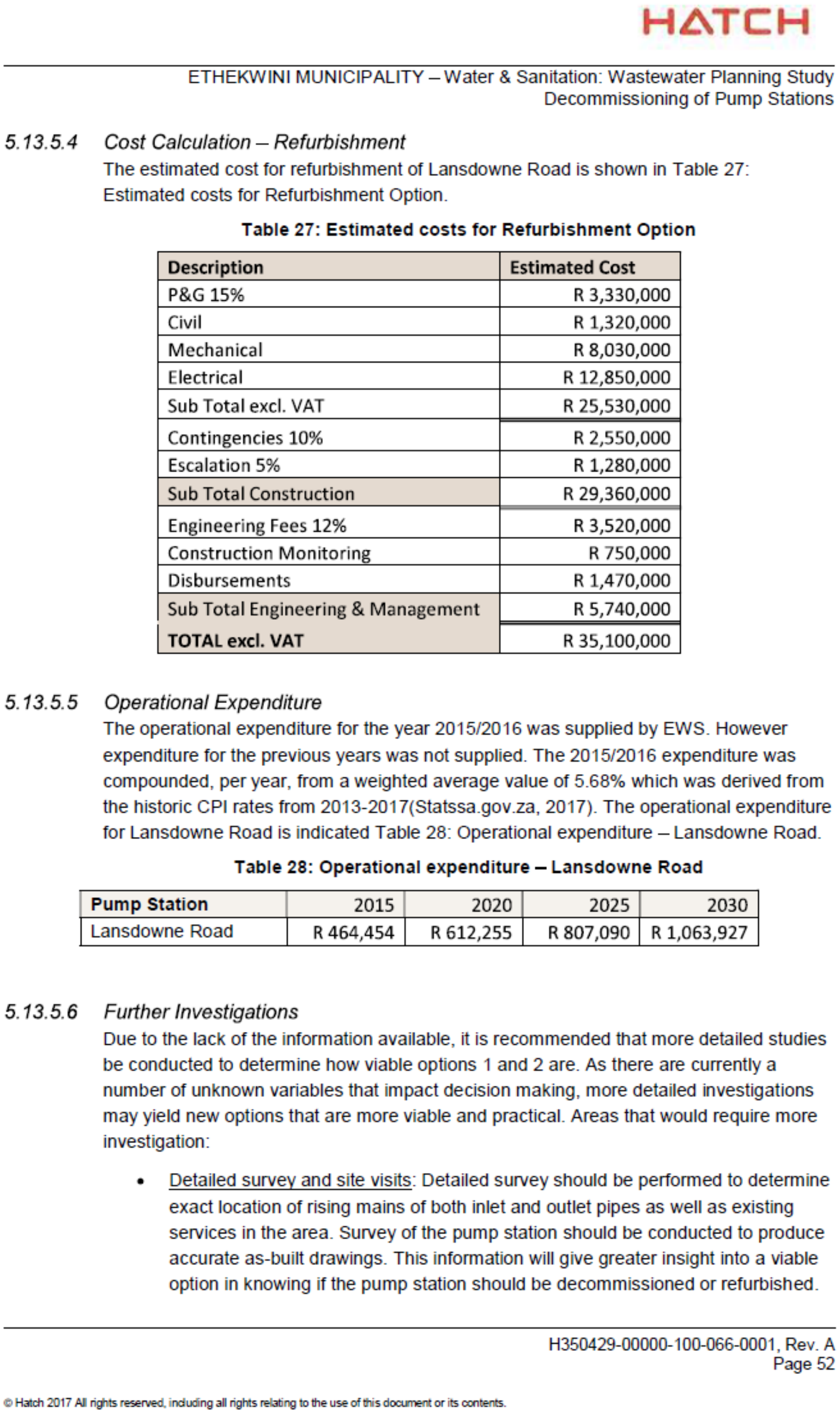

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Decommissioning of Pump Stations

5.13.5.4 *Cost Calculation – Refurbishment*

The estimated cost for refurbishment of Lansdowne Road is shown in Table 27: Estimated costs for Refurbishment Option.

Table 27: Estimated costs for Refurbishment Option

Description	Estimated Cost
P&G 15%	R 3,330,000
Civil	R 1,320,000
Mechanical	R 8,030,000
Electrical	R 12,850,000
Sub Total excl. VAT	R 25,530,000
Contingencies 10%	R 2,550,000
Escalation 5%	R 1,280,000
Sub Total Construction	R 29,360,000
Engineering Fees 12%	R 3,520,000
Construction Monitoring	R 750,000
Disbursements	R 1,470,000
Sub Total Engineering & Management	R 5,740,000
TOTAL excl. VAT	R 35,100,000

5.13.5.5 *Operational Expenditure*

The operational expenditure for the year 2015/2016 was supplied by EWS. However expenditure for the previous years was not supplied. The 2015/2016 expenditure was compounded, per year, from a weighted average value of 5.68% which was derived from the historic CPI rates from 2013-2017(Statssa.gov.za, 2017). The operational expenditure for Lansdowne Road is indicated Table 28: Operational expenditure – Lansdowne Road.

Table 28: Operational expenditure – Lansdowne Road

Pump Station	2015	2020	2025	2030
Lansdowne Road	R 464,454	R 612,255	R 807,090	R 1,063,927

5.13.5.6 *Further Investigations*

Due to the lack of the information available, it is recommended that more detailed studies be conducted to determine how viable options 1 and 2 are. As there are currently a number of unknown variables that impact decision making, more detailed investigations may yield new options that are more viable and practical. Areas that would require more investigation:

- Detailed survey and site visits: Detailed survey should be performed to determine exact location of rising mains of both inlet and outlet pipes as well as existing services in the area. Survey of the pump station should be conducted to produce accurate as-built drawings. This information will give greater insight into a viable option in knowing if the pump station should be decommissioned or refurbished.

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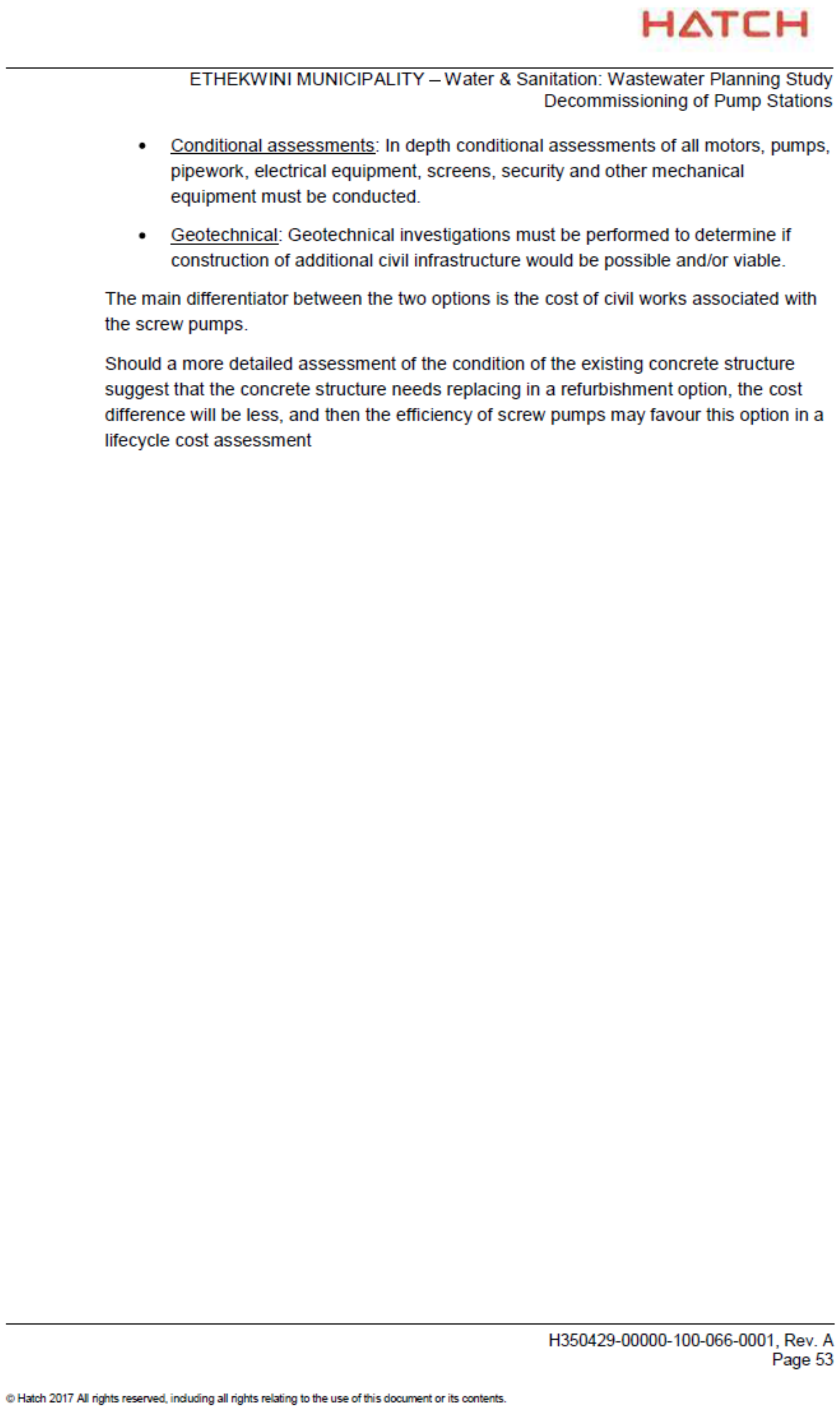

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Decommissioning of Pump Stations

- Conditional assessments: In depth conditional assessments of all motors, pumps, pipework, electrical equipment, screens, security and other mechanical equipment must be conducted.
- Geotechnical: Geotechnical investigations must be performed to determine if construction of additional civil infrastructure would be possible and/or viable.

The main differentiator between the two options is the cost of civil works associated with the screw pumps.

Should a more detailed assessment of the condition of the existing concrete structure suggest that the concrete structure needs replacing in a refurbishment option, the cost difference will be less, and then the efficiency of screw pumps may favour this option in a lifecycle cost assessment

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Appendix B: Prefeasibility

Prefeasibility:

ETHEKWINI WATER AND SANITATION

ENGINEERING

WASTE WATER: DESIGN BRANCH

PROJECT NO. Y6652 LANSDOWNE SEWAGE PUMP STATION REPLACEMENT

PREFEASIBILITY FOR REPLACING EXISTING SYSTEM WITH ARCHIMEDEAN SCREW PUMPS

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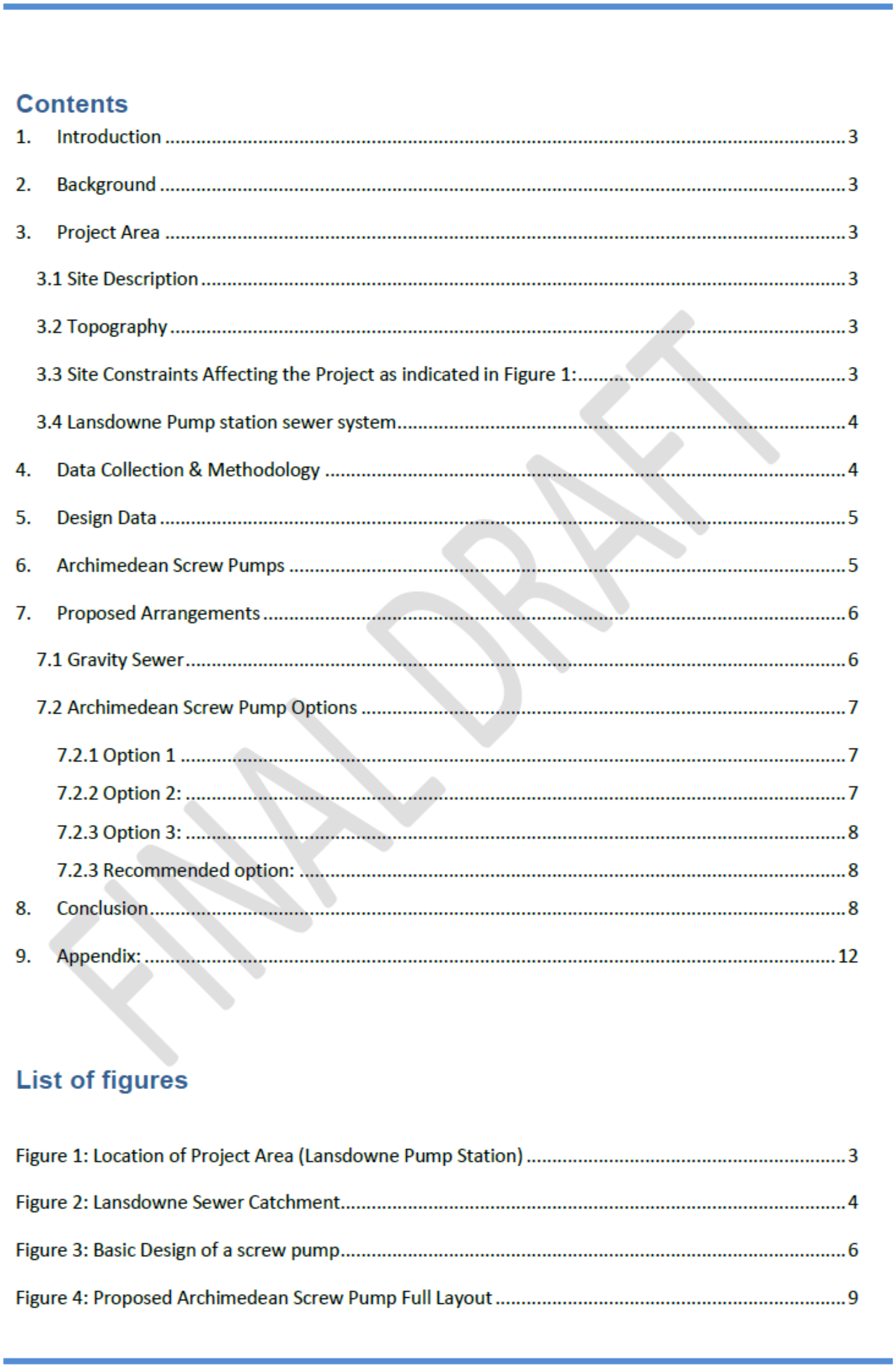
Date: March 2019



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1. Introduction

The report is a feasibility study to determine whether it is financially viable to replace the existing sewage pumping system at Lansdowne Road with Archimedean screw pumps. The report focuses on achieving the required pumping head and the appropriate gradient for the laying of a new gravity sewer from the outlet chamber, to tie into the existing trunk sewer on Lansdowne Road. The report presents a cost analysis between the two systems.

2. Background

Lansdowne Road sewage pump station was constructed in the early 1960's. Since then it is old and has deteriorated substantially. The pump station requires major upgrades to keep it operational. Wastewater design was tasked to assess the two options of refurbishing the existing pump station or replacing the system with Archimedean screw pumps.

This report assesses the financial viability of installing Archimedean screw pumps within the existing pump station site focusing on achieving the required pumping head and appropriate gradient for the new gravity sewer linking the pump station and the existing trunk sewer.

3. Project Area

3.1 Site Description

The pump station site is situated in Ward 75 within the Mobeni east industrial area which is located south of Durban as shown in Figure 1 below. eThekwini Municipality owns the site. The pump station site according to measurements from ArcGIS is approximately 1 800m^2.

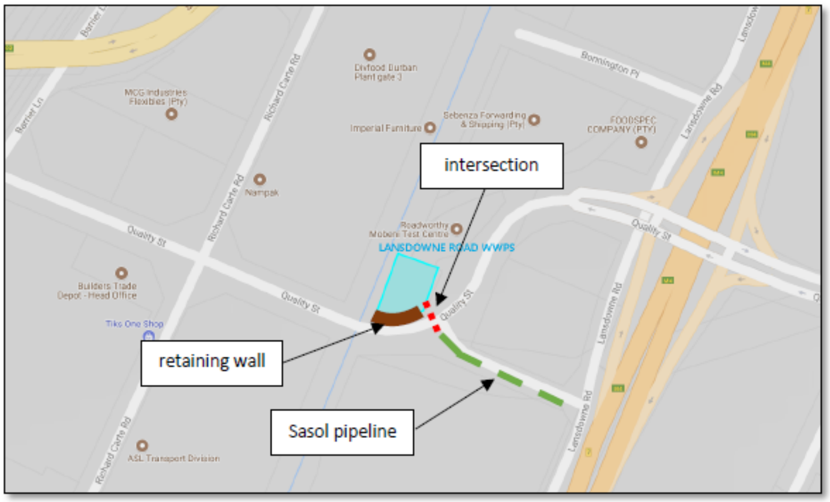


Figure 1: Location of Project Area (Lansdowne Pump Station)

3.2 Topography

The project site is fairly flat at a ground level between 10 -12 mSL with the Amanzimyana canal running adjacent, along the western boundary.

3.3 Site Constraints Affecting the Project as indicated in Figure 1:

- An existing 8m high retaining wall at the southern side of the pump station.

- A wide intersection at the junction of Quality Street.
- A 0.5m servitude containing a Sasol methane gas steel pipeline
- A 300 MPVC water main runs along the route of the Sasol pipeline in Figure 1

3.4 Lansdowne Pump station sewer system

The sewer catchment to Lansdowne pump station is made up of 8 pump stations namely Unit Avenue, Seaward Road, Carolina, Richard Carte, Robert Grove, River Drive, Edwin Swales and Rossburgh Bus Depot. The flow from the aforementioned pump stations are conveyed via the Umhlatuzana and Umbilo trunk sewers to Lansdowne pump station. The raw sewage from Lansdowne then pumps up to a connection manhole and gravitates to the Southern wastewater treatment plant via the Jacobs line.

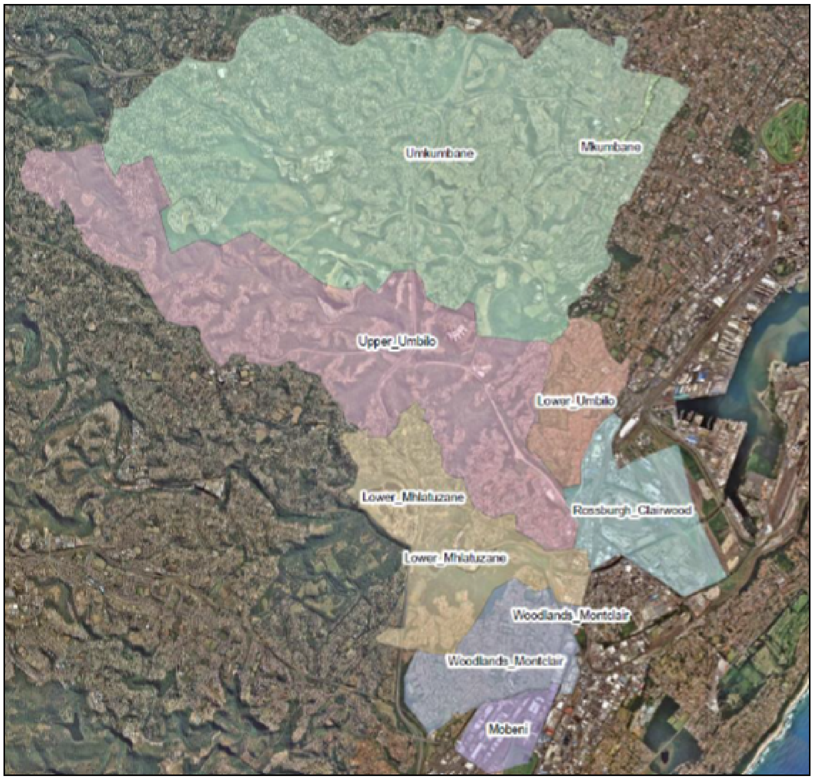


Figure 2: Lansdowne Sewer Catchment

Domestic flow from the Umkumbane Upper Umbilo, Lower Umbilo, Woodlands Montclair, Upper Umbilo, Upper and Lower Umhlatuzane make up a major portion of the total flow into Lansdowne. Industrial areas such as Rossburgh Clairwood and Mobeni contribute to the remainder.

4. Data Collection & Methodology

A clear understanding of the catchment area was ascertained through communication with the experienced engineers. Available existing information was obtained during the process. A draw down test was conducted at the pump station to ascertain the average sewage inflow and capacity of the pumps. The capacity of two pumps was assumed to be 1042l/s.

As-built records indicate that there is an inlet main of 1 500mm Ø to the pump station with two rising mains exiting, 500 mm Ø and 900mm Ø respectively and connecting to a 1 500mm Ø trunk sewer on Lansdowne road. Invert levels of the inlet pipe and terminal manhole was determined via a survey with spot shots conducted in March 2018. Furthermore, as-built drawings of the inlet main were obtained which confirmed the invert levels.

ArcMap GIS and site visits were undertaken to determine available space at the site for the proposed pumping system. Findings from GIS revealed that a 450mm Ø stormwater pipeline cut across the pump station site. Coastal Stormwater & Catchment Management department confirmed that the line does not traverse the pump station site. GIS was the used to develop the proposed pumping arrangement.

5. Design Data

Below are the parameters that guided the design of the new system:

Table 1: Set Parameters for Design

Set Parameters for Design		
Invert level of the inlet	6	mSL
Diameter of proposed gravity sewer	1 500	mm Ø
Total length of pipeline from inlet to existing trunk sewer	189.5	m
Invert level of terminal manhole	11	m
Design flow (Peak plus 10% future flow)	600	l/s
Required pumping head	5	m

As-built drawings were used to ascertain the size of the existing trunk sewer and the proposed gravity sewer to match the size.

6. Archimedean Screw Pumps

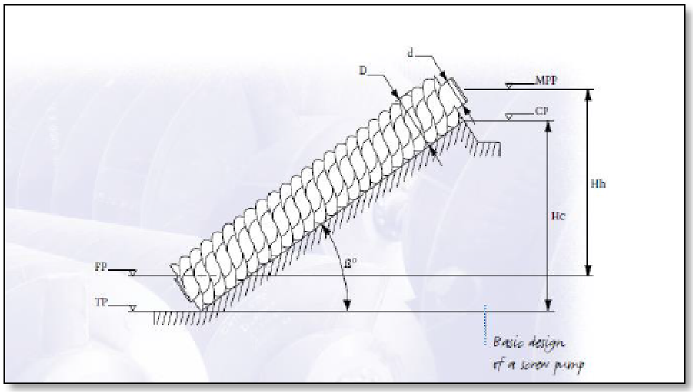


Figure 3: Basic Design of a screw pump

TP (m)	Touch Point (Screw pump capacity 0%)
FP (m)	Filling Point (Screw pump capacity 100%)
MPP(m)	Maximum Pumping Point (Highest pumping level possible and ηmax
CP (m)	Chute point (invert of trough at outlet)
β	Angle of inclination (Variable between 22° and 38°, depends on required capacity and head)
Hc	Building Height (Difference between TP and CP)
Hh	Maximum Head (Difference between FP & MPP)
D	Diameter of the screw
d	Diameter of centre tube

7. Proposed Arrangements

Spaans Babcock tables guided the design for the selection of the Archimedean screw pumps. The invert level of the inlet to the pump station was taken as the Filling Point (FP) and the level of the terminal manhole was used as the Chute point.

7.1 Gravity Sewer

The desired gradient for the 1 500mm Ø gravity sewer was determined using *Pipeline Hydraulics for Partially Filled Pipes* programme which was developed by a professional engineer. This gradient ensured that the flow was within the subcritical zone.

Input parameters into the programme were as follows:

Table 2: Input Parameters for the Pipeline Hydraulics programme

Input parameters into programme		
Internal diameter of sewer (concrete pipe)	1 345	mm
Design flow (Q)	0.60	m^3/s
Manning's roughness coefficient	0.012	
ϕ	203.1	°

And calculated the gradient as 1: 4111 for the pipe to flow at 0.6D. Charts are produced as part of the results which highlighted the subcritical and supercritical zone. It was established that a gradient steeper than 1:343 would lie within the supercritical zone. Therefore 1:350 was used as an acceptable gradient (See Appendix B).

7.2 Archimedean Screw Pump Options

Archimedean screw pump options were determined using the set design parameters in conjunction with the tables from Spaans Babcock. It should be noted that at least one backup screw is required for each alternative. The maximum angle of inclination is 38° due to economic reasons. Furthermore, as the angle of inclination decreases, the capacity that the screw can handle increases. Consideration must be given to ensure the operation of two pumps as per normal during the construction phase.

Peak flow plus 10% future flow **600 l/s** was used as the design flow (**Q**) with required pumping head of **5m** for each option.

7.2.1 Option 1

Using Spaans Babcock tables, the figures as shown in Table 3 can be achieved.

Table 3: Screw Pump Option 1 (Spaans Babcock)

Option 1		
Diameter of screw	1 100	mm
No of screws incl. back-up	3	
Q (1 screw – 310l/s)	620	l/s
FP-CP (max pumping head)	5	m
β	30	°

This option requires a significant amount of space to install 3 Archimedean screws of diameter 1 100mm. In this option,1 screw can handle only 52 % of the flow, which is minimal and not preferred in the case of failure. The maximum head that the screw can pump is on target with the required pumping head and has no safety factor.

7.2.2 Option 2:

Using Spaans Babcock table, the figures as shown in Table 4 can be achieved.

Table 4: Screw Pump Option 2 (Spaans Babcock)

Option 2		
Diameter of screw	1 200	mm
No of screws incl. back-up	3	
Q (1 screw – 300l/s)	600	l/s
FP-CP (max pumping head)	6.5	m
β	35	°

This option requires the installation of 3 screws of diameter 1 200mm including back-up. Alternative needs substantial space for the 3 screws. In this alternative 1 screw can handle 50% of the peak flow. The maximum pumping head is a little higher than the requirement which provides a buffer zone. The depth of the discharge trough or channel can be constructed to drop the head to a required level if necessary to enable the gradient 1:350 for the gravity sewer. The angle of inclination is greater than option 1 at 35° therefore decreasing the horizontal space required.

7.2.3 Option 3:

Using Spaans Babcock table, the figures as shown in Table 5 can be achieved.

Table 5: Screw Pump Option 3 (Spaans Babcock)

Option 3		
Diameter of screw	1 600	mm
No of screws incl. back-up	2	
Q (1 screw)	745	l/s
FP-CP (max pumping head)	6.3	m
β	30	°

The installation of 2 screws with diameter 1 600mm is sufficient for the required capacity including back-up. It should be noted that 1 screw can handle more than 100% of the peak flow at maximum capacity. With an increased pumping head of 6.3m, there is a greater safety factor. However, if the head is too high, it can be dropped by the construction of a discharge channel or trough of depth 0.5m to collect the sewer at the outlet whilst maintaining the gradient 1:350 for the gravity sewer.

7.2.3 Recommended option:

Based on the discussions above, option 3 is preferred i.e. **2 Archimedean screws** of diameter **1 600 mm** of capacity **745l/s** pumping a head of **6.3m.** The use of two Archimedean screws including back-up is the best solution as minimal space is required.

According to the as-built drawings, the existing pump station is a round caisson having a diameter of 18.6m with a 700mm thick wall containing a section for the dry & wet well. There is a dividing wall within the wet well for the 2 sumps. This substructure was constructed in the 1960s. Since the pump station is required to operate as per normal during the construction period, it is advisable not to disturb the current site with a new construction within the substructure. Therefore, the pump arrangement for the proposed system is located on the outside of the existing pump station but within the existing site.

The proposed solution can fit within the available footprint outside the existing pump station as per Appendix C & D.

8. Conclusion

The existing pump station can be replaced by the use of Archimedean screws as there is adequate land available at the site to make this option viable. The cost for the design will be determined at the preliminary design stage.

Considerations for the design should include screening prior to the sewage entering the screw pumps. Redundancy for screenings, pumps, power must be included in the design. Since degritting is not feasible at the site, the ends of the screw flights must be welded on by stainless steel tips. A new access gate may be required to accommodate the proposed Archimedean screw pump setup. A discharge channel of approximately 0.8m deep would allow for the desired gradient of 1:350.

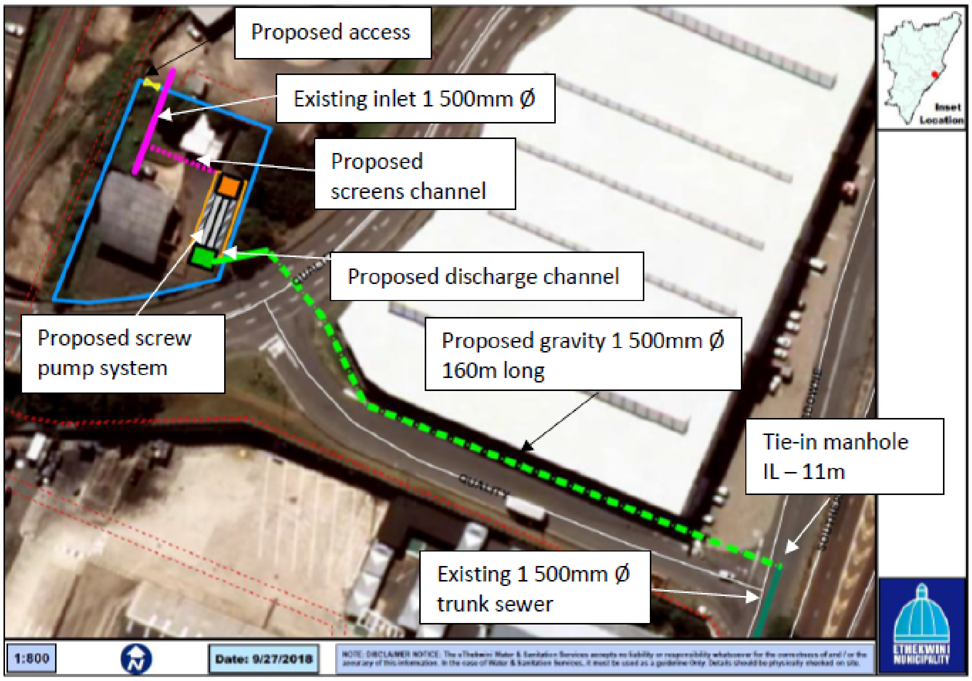


Figure 4: Proposed Archimedean Screw Pump Full Layout

The proposed route of the pipeline avoids the gas pipeline buy running along the opposite side of the road.

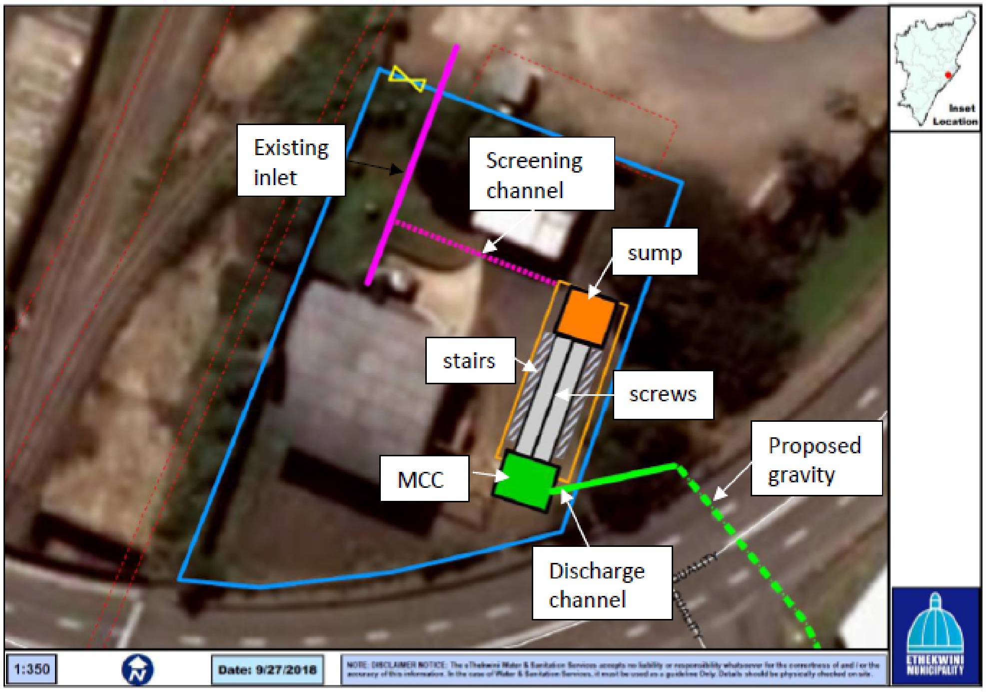


Figure 5: Screw Pump Configuration

A 13m section of the1 500mm Ø concrete pipe will need to be jacked under Quality Street (Refer to Appendix C)
Detailed investigation is required to check the viability of pipe jacking or micro-tunnelling. The gravity main will tie into a terminal manhole which is connected to a 1 500mm Ø trunk sewer on Lansdowne Road. The trunk main terminates at Southern Wastewater Treatment Works. The existing 500mm Ø and 900mm Ø rising mains will be decommissioned.

8.2 Considerations for Design

- Environmental Considerations
  Overflow at the pump station currently enters the Amanzimyama canal. This is a health hazard and should be avoided, therefore allowance of a generator must be made to minimise overflow due to power outages. However, the existing overflow channel will be maintained in case of emergencies.
- Health & Safety
  The current pump station does not have adequate ventilation; therefore, this must be considered when constructing the new system. Odour control is required as this is a raw sewage pumping station. Enclosing the system is necessary such as covers for the channels and screws.
- Accessibility
  Adequate access should be provided for operating and maintaining equipment. A new access gate may be required. Work space must be taken into account during the preliminary design.
- Protection of the Archimedean screws from heat
  There are two options to consider i.e. enclosing the screws completely or spraying with water to keep the temperature down. Enclosure of the screws is preferred as it would also maintain odour control.

- Tie-in to the existing system
  Diversion of most of the flow coming through the Umhlatuzana and Umbilo trunk to the Island view pump station is a possibility. The capacity of the line and the Island view pump station has to be assessed. It should be noted that only the flow upstream of the channel can be diverted.
- Existing services
  All existing services must be verified on the ground prior to construction
- Back-up power supply
  Allowance for a generator and fuel storage area must be made considered.
- Automation
  Installation of flow meters at the inlet and outlet connected up to telemetry for monitoring online
- Security
  Consideration must be given to materials used and security of equipment on site to minimise theft and vandalism.

8.3 Further investigations required:

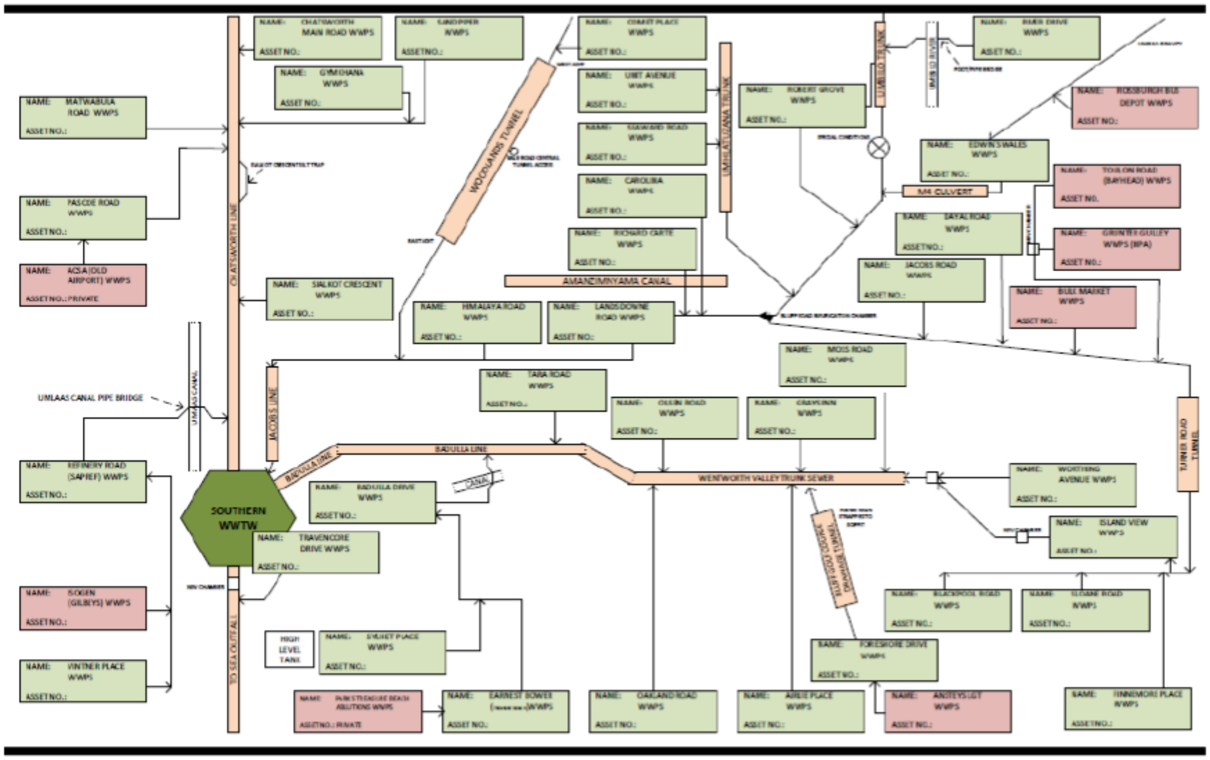
- Extension to survey and site visits:
  Extended survey should be performed to determine exact location of rising mains of both inlet and outlet pipes as well as existing services in the area. Ground levels for the proposed pipeline route is required.
- Conditional assessments: In depth conditional assessments of all motors, pumps, pipework, electrical equipment, screens, and other mechanical equipment must be conducted.
- Preliminary design which will include cost estimate for the proposed pumping system must be done
- Geotechnical: Geotechnical investigations must be performed to determine if construction of additional civil infrastructure would be possible and/or viable including pipe jacking for the new gravity sewer
- Study to determine the method of installation for the proposed 1 500mm Ø gravity sewer under Quality Street.
- Actual measurement of the incoming flow to the pump station must be determined.
- Study on dealing with existing flow during replacement of centrifugal axial pumps.
- A Hazop study as requested by M&E

In conclusion there is space to replace the existing pumping system with Archimedean screw pumps however, a preliminary design will be required to ascertain the cost.

9. Appendix:

APPENDIX A: PUMP STATION SCHEMATIC

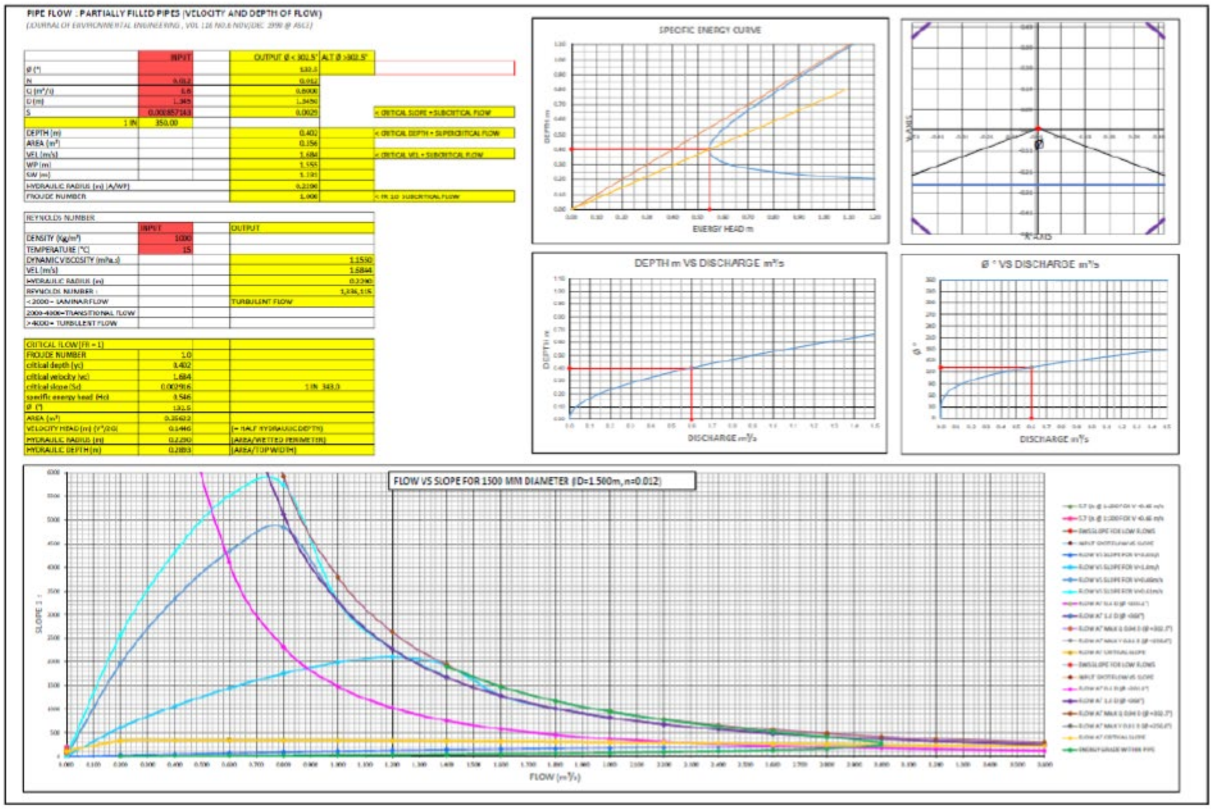
SOUTHERN COASTAL AREA: SOUTHERN WASTEWATER NETWORK – PUMP STATION SCHEMATIC



08 August 2018

Revised by:
Roy Maharaj

APPENDIX B: PIPELINE HYDRAULICS FOR 1:350



PIPE FLOW : PARTIALLY FILLED PIPES (VELOCITY AND DEPTH OF FLOW)

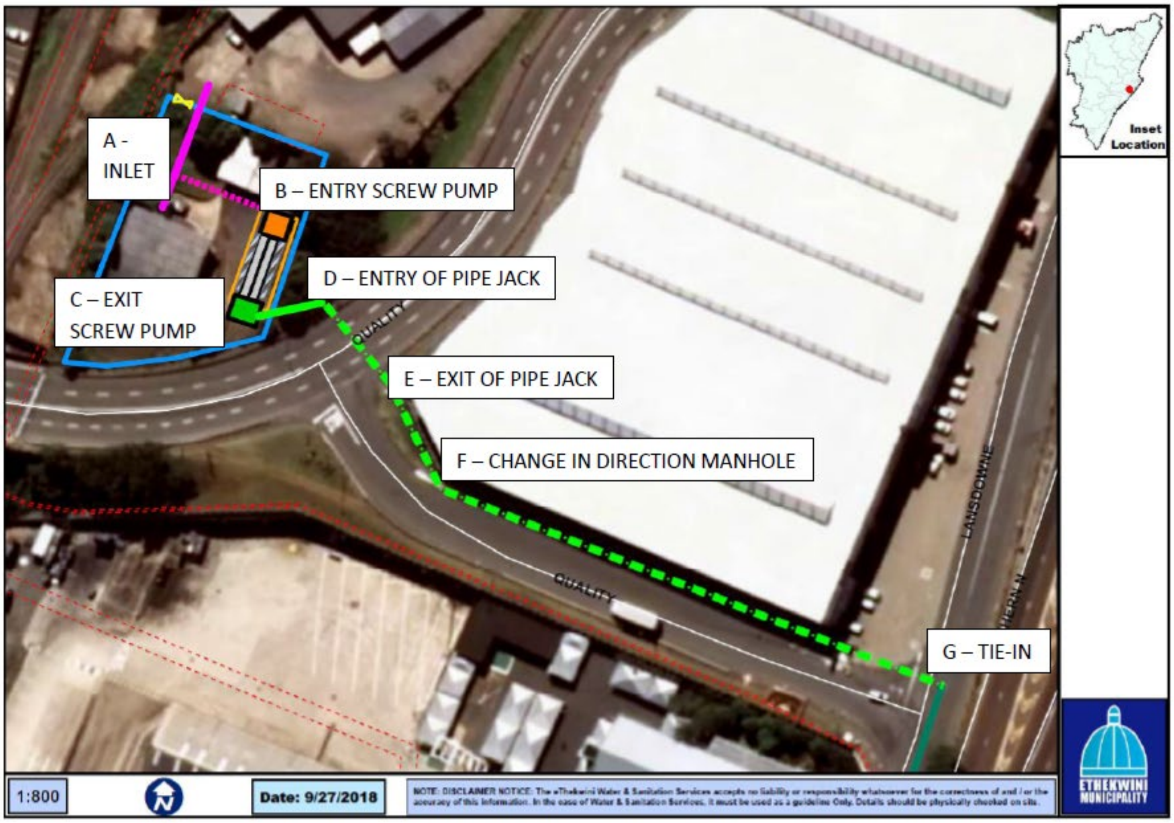
(JOURNAL OF ENVIRONMENTAL ENGINEERING , VOL 116 NO.6 NOV/DEC 1990 @ ASCE)

	INPUT		OUTPUT Ø < 302.5°	ALT Ø >302.5°	
Ø (°)			132.5		
N	0.012		0.012		
Q (m³/s)	0.6		0.6000		
D (m)	1.345		1.3450		
S	0.002857143		0.0029		< CRITICAL SLOPE = SUBCRITICAL FLOW
1 IN	350.00				
DEPTH (m)			0.402		< CRITICAL DEPTH = SUPERCRITICAL FLOW
AREA (m²)			0.356		
VEL (m/s)			1.684		< CRITICAL VEL = SUBCRITICAL FLOW
WP (m)			1.555		
SW (m)			1.191		
HYDRAULIC RADIUS (m) (A/WP)			0.2290		
FROUDE NUMBER			1.000		< FR 1.0 SUBCRITICAL FLOW

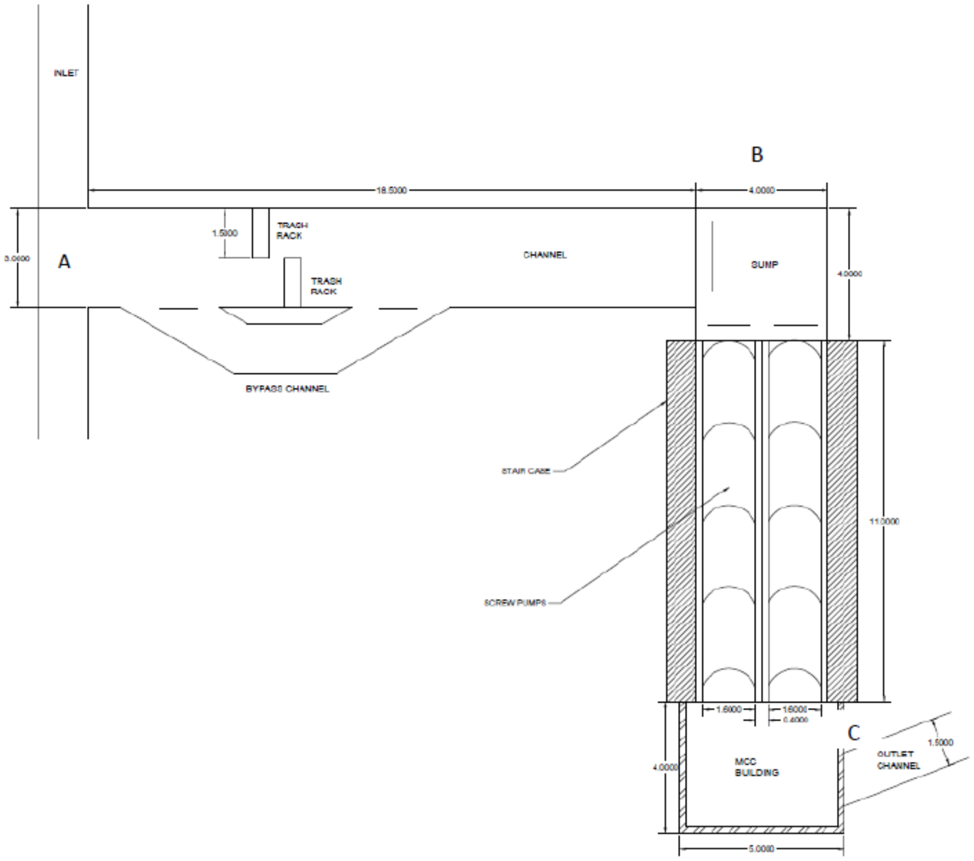
REYNOLDS NUMBER			
	INPUT		OUTPUT
DENSITY (Kg/m³)	1000		
TEMPERATURE (°C)	15		
DYNAMIC VISCOSITY (mPa.s)			1.1550
VEL (m/s)			1.6844
HYDRAULIC RADIUS (m)			0.2290
REYNOLDS NUMBER :			1,336,115
<2000 = LAMINAR FLOW			TURBULENT FLOW
2000-4000=TRANSITIONAL FLOW			
>4000 = TURBULENT FLOW			

CRITICAL FLOW (FR = 1)			
FROUDE NUMBER	1.0		
critical depth (yc)	0.402		
critical velocity (vc)	1.684		
critical slope (Sc)	0.002916		1 IN 343.0
specific energy head (Hc)	0.546		
Ø (°)	132.5		
AREA (m²)	0.35622		
VELOCITY HEAD (m) (V²/2G)	0.1446		(= HALF HYDRAULIC DEPTH)
HYDRAULIC RADIUS (m)	0.2290		(AREA/WETTED PERIMETER)
HYDRAULIC DEPTH (m)	0.2893		(AREA/TOP WIDTH)

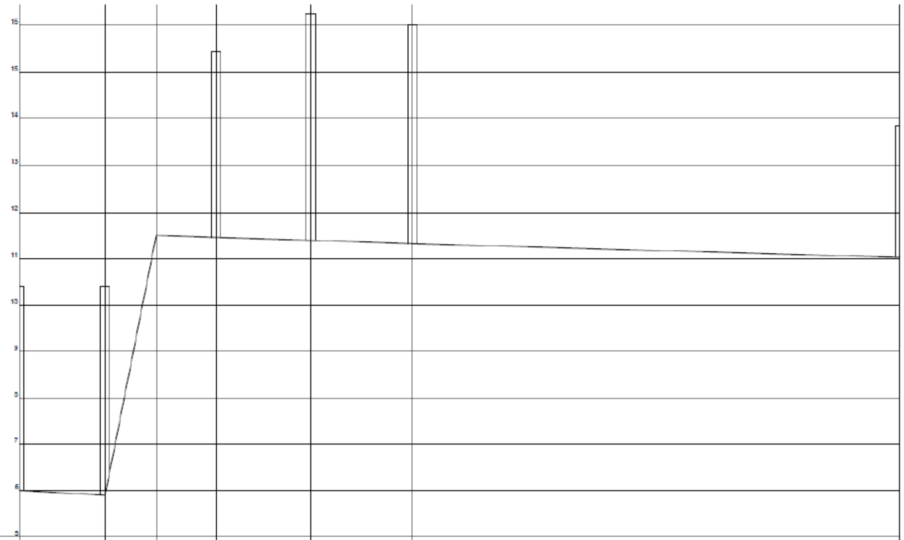
APPENDIX C: PROPOSED ARCHIMEDEAN SCREW PUMP ARRANGEMENT



APPENDIX D: DRAWING OF PROPOSED ARCHIMEDEAN SCREW PUMP ARRANGEMENT

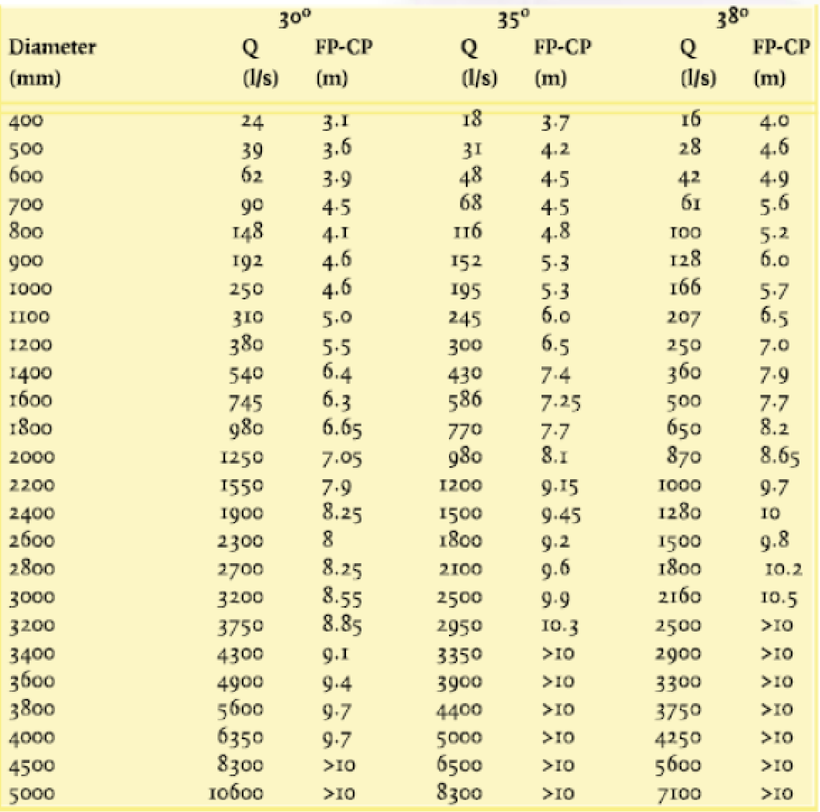


APPENDIX E: LONG SECTION FOR PROPOSED ARCHIMEDEAN SCREW PUMP ARRANGEMENT



MANHOLE NAME	A	B	C	D	E	F	G
CHAINAGE (m)	0.00	18.5	29.5	42.5	62.5	84.5	189.5
GROUND LEVEL (m)	10.4	10.4		15.44	16.24	16.03	13.85
INVERT LEVEL (m)	6.0	5.9	11.5	11.46	11.4	11.33	11.03
DEPTH (m)	4.4	4.5		3.98	4.84	4.7	2.82

APPENDIX F: TABLE CONTAINING CAPACITY & MAXIMUM (FP-CP) VALUES AT DIFFERENT HEADS (SPAANS BABCOCK)



	30°		35°		38°	
Diameter (mm)	Q (l/s)	FP-CP (m)	Q (l/s)	FP-CP (m)	Q (l/s)	FP-CP (m)
400	24	3.1	18	3.7	16	4.0
500	39	3.6	31	4.2	28	4.6
600	62	3.9	48	4.5	42	4.9
700	90	4.5	68	4.5	61	5.6
800	148	4.1	116	4.8	100	5.2
900	192	4.6	152	5.3	128	6.0
1000	250	4.6	195	5.3	166	5.7
1100	310	5.0	245	6.0	207	6.5
1200	380	5.5	300	6.5	250	7.0
1400	540	6.4	430	7.4	360	7.9
1600	745	6.3	586	7.25	500	7.7
1800	980	6.65	770	7.7	650	8.2
2000	1250	7.05	980	8.1	870	8.65
2200	1550	7.9	1200	9.15	1000	9.7
2400	1900	8.25	1500	9.45	1280	10
2600	2300	8	1800	9.2	1500	9.8
2800	2700	8.25	2100	9.6	1800	10.2
3000	3200	8.55	2500	9.9	2160	10.5
3200	3750	8.85	2950	10.3	2500	>10
3400	4300	9.1	3350	>10	2900	>10
3600	4900	9.4	3900	>10	3300	>10
3800	5600	9.7	4400	>10	3750	>10
4000	6350	9.7	5000	>10	4250	>10
4500	8300	>10	6500	>10	5600	>10
5000	10600	>10	8300	>10	7100	>10

Appendix C: Basic Life Cycle Cost Analysis (LCCA)

Basic Life Cycle Cost Analysis (LCCA):

ETHEKWINI WATER AND SANITATION

ENGINEERING

WASTEWATER: DESIGN BRANCH

PROJECT NO. Y6652: LANSDOWNE ROAD SEWAGE PUMP STATION

BASIC LIFE CYCLE COST ANALYSIS REPORT

Prepared by: Hope Joseph
BSc Civil Eng.
Tel: 031 3118577

Date: 24 April 2019

Checked by: Ian Duncum
PR Eng.

Project No. Y6652 Lansdowne Road Sewage Pump Station: Basic Life Cycle Cost Analysis

Contents

1. Introduction and Background

Lansdowne sewage pump station was constructed in the early 1960s. It is nearing it's expected lifespan of 50 years. Operational and maintenance tasks have become challenging to keep the pump station functioning as the pumps, motors and other components have deteriorated. The 4 Allen Gwynne pumps cannot be maintained properly as spares are no longer available due to the pump manufacturer winding up their operation in South Africa. Thus eThekwini Water and Sanitation (EWS) is seeking to replace the current pumps and upgrade the pump station to the specified EWS standard with a suitable system that would be effective and efficient in the long-term.

This report is to assess two options by doing a life cycle cost analysis to determine the preference. The two options that were investigated viz. (a) Replacing existing pumps with centrifugal (submersible) pumps (b) Replacing existing pumps with Archimedean screw pumps.

The pump station site is located in the southern part of Durban, within the Mobeni east industrial area in Ward 75 as shown in Figure 1.

Figure 1: Locality of the Lansdowne Road Sewage Pump Station

2. Data Collected

Data was obtained from various sources.

2.1 Sewer Flow

ArcGIS was used to identify the spatial catchment of the pump station. A geodatabase that merged ArcGIS and COINS to produce a spatial file with actual meter readings of water consumption was used to calculate the average sewer flow generated from the catchment. Since the file only had historic data for 2015/2016, the values were raised by a factor of 1.5 (as per EWS, Planning) to calculate the current average sewer flow of 45 Ml/day which equates to 521l/s. The design of a pump station is based on the worst case scenario which includes the peak factor, infiltration and future development. Therefore, using a peak factor of 1.8, 15% infiltration and 25% for future flow, the calculated design flow yielded 1 346l/s. From the sump draw-down test performed by EWS, Mechanical & Electrical (M&E) branch, the pumps (two pumps in parallel) maximum handling capacity was measured to be 1 164l/s, an addition of 25% for future development yielded 1 455l/s. Therefore, the design flow of 1 460l/s was used for the purposes of this study as indicated in

Table 1.

Table 1: Set Parameters for Options Investigated

Set Parameters Used		
Design Flow	1460	l/s
Required pumping head for Archimedean screw pumps	6	m
Required pumping head for Centrifugal submersible pumps	8	m

The required pumping head for centrifugal pumps was ascertained from as-built drawings and identifying the current pumping base level and invert level of the connection manhole. Pumping head for Archimedean screw pumps were taken as the difference between the invert level of the inlet and connection manhole since Archimedean screw pumps don't require a sewage sump to store flow as dry running is acceptable.

2.2 Life Cycle Costs

The life cycle cost of each option was determined over a period of 60 years for comparative purposes. Capital, operational and maintenance costs were used for the analysis. Costs used were obtained from EWS M&E branch, suppliers and a report titled *EThekwini Municipality – Water & Sanitation: Wastewater Planning Study: Decommissioning of Pump Stations* (2017) done by Hatch Ltd.

The following costs were considered for both options:

- Capital
    - Civil – cost adopted with a 30% inflation from Hatch's report as an immediate once-off cost.
    - Mechanical – pricing obtained from suppliers, replaced as per lifespan of the pumps. Centrifugal pumps replaced at 20 year intervals and Archimedean screw pumps replaced every 30 years.
    - Electrical – cost adopted with a 30% inflation from Hatch's report, replacements done in conjunction with mechanical replacements.
    - Ventilation and odour control – Replacement of fans and motors at 10 year intervals.

  - Pipe jacking and pipeline – pricing from service providers, once-off cost. Construct immediately for option 1 and construct at year 10 for option 2.
  - P&G – 15% of the total capital life cycle cost
  - Engineering fees – 10% of the total capital life cycle cost

- Operational
  - Electricity – pricing per kwh and percentage escalation for the 60-year period were obtained through consultation with EThekwini Electricity department. Electricity consumption for option 2: centrifugal pumps was determined from a January 2019 utility bill with meter readings between June 2018 to September 2018 of Lansdowne Road Pump station, that was available at the time of the investigation. The figure of 800kwh per day was then verified by EWS, M&E branch to be within range of a previous study that they had conducted. It should be noted that the current pump station is old and therefore has a low efficiency. Electricity requirement for the Archimedean screw pumps were taken as 5/7th (as per Wastewater Design management) of that of Option 2 with an efficiency of 80% as per a recognised Archimedean screw pump supplier.

- Maintenance

  All maintenance requirements for both options were obtained from EWS, M&E branch.

  Option 1: Archimedean screw pumps
  - Gearbox oil
  - Grease
  - V belts
  - Back stops (one direction bearings)

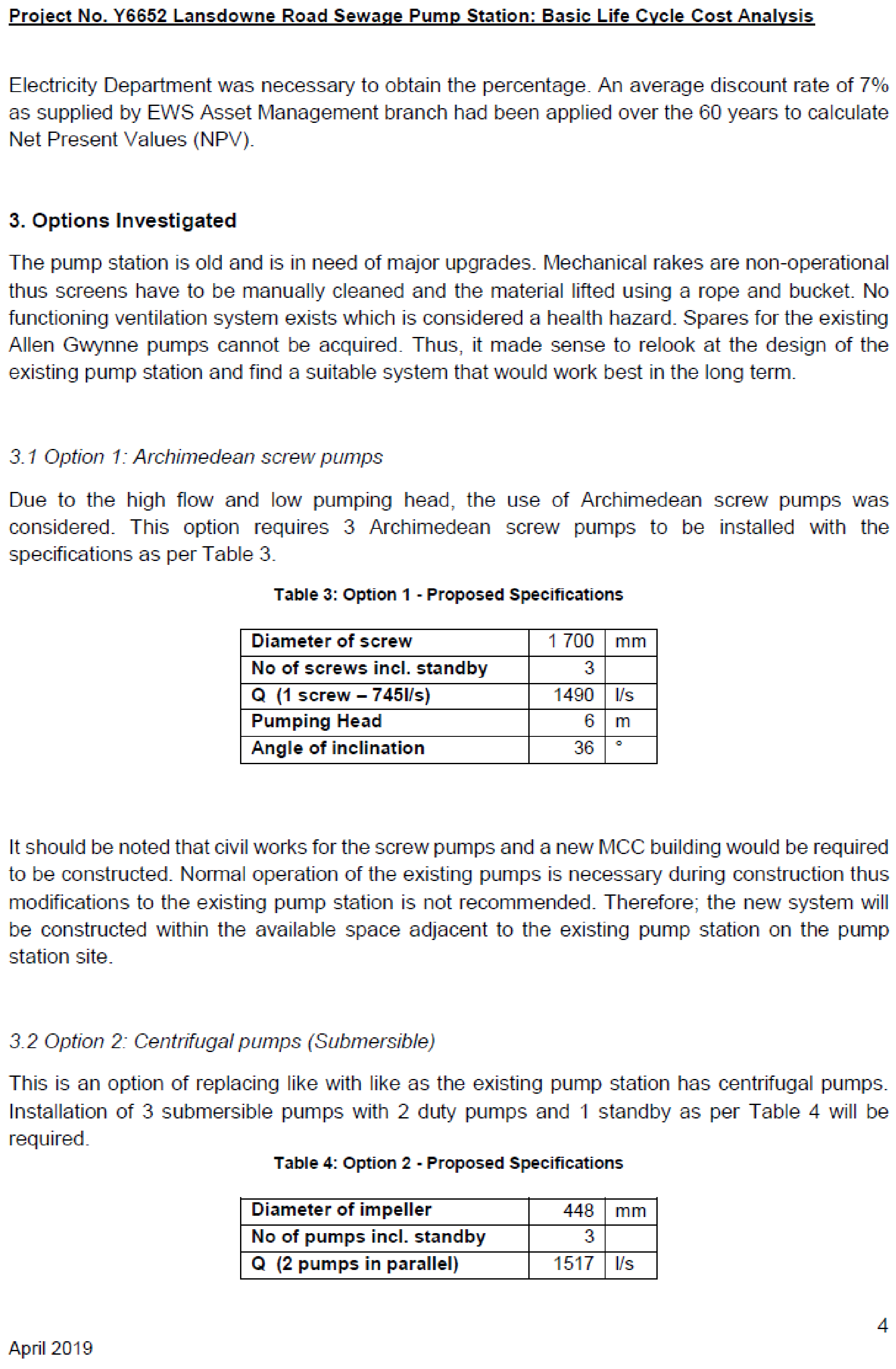
  Option 2: Centrifugal pumps
  - Glycol (Pump cooling jacket)
  - Oil
  - Grease (valve spindles/screen gearboxes/compactor gearboxes)

The rates that had been used for the life cycle cost analysis is depicted in Table 2.

Table 2: Rates used for the life cycle cost analysis

Rates Used		
CPI	5	%
Escalation for electricity	8	%
Discount rate	7	%

CPI was calculated from a 10-year historic record for South Africa (Accessed on the 4 April 2019, https://www.inflation.eu/inflation-rates/south-africa/historic-inflation/cpi-inflation-south-africa.aspx). Escalation for electricity varies erratically over the years, therefore consultation with



Electricity Department was necessary to obtain the percentage. An average discount rate of 7% as supplied by EWS Asset Management branch had been applied over the 60 years to calculate Net Present Values (NPV).

3. Options Investigated

The pump station is old and is in need of major upgrades. Mechanical rakes are non-operational thus screens have to be manually cleaned and the material lifted using a rope and bucket. No functioning ventilation system exists which is considered a health hazard. Spares for the existing Allen Gwynne pumps cannot be acquired. Thus, it made sense to relook at the design of the existing pump station and find a suitable system that would work best in the long term.

3.1 Option 1: Archimedean screw pumps

Due to the high flow and low pumping head, the use of Archimedean screw pumps was considered. This option requires 3 Archimedean screw pumps to be installed with the specifications as per Table 3.

Table 3: Option 1 - Proposed Specifications

Diameter of screw	1 700	mm
No of screws incl. standby	3	
Q (1 screw – 745l/s)	1490	l/s
Pumping Head	6	m
Angle of inclination	36	°

It should be noted that civil works for the screw pumps and a new MCC building would be required to be constructed. Normal operation of the existing pumps is necessary during construction thus modifications to the existing pump station is not recommended. Therefore; the new system will be constructed within the available space adjacent to the existing pump station on the pump station site.

3.2 Option 2: Centrifugal pumps (Submersible)

This is an option of replacing like with like as the existing pump station has centrifugal pumps. Installation of 3 submersible pumps with 2 duty pumps and 1 standby as per Table 4 will be required.

Table 4: Option 2 - Proposed Specifications

Diameter of impeller	448	mm
No of pumps incl. standby	3	
Q (2 pumps in parallel)	1517	l/s

Pumping Head	8	m

However, the current pump station is in a poor condition and will require major upgrades to meet the acceptable EWS standard. Civil works such as the replacement of the pump station's asbestos roof will be required. Other replacements include mechanical and electrical such as ventilation system, mechanical rake and lift, electrical fittings, wirings and switches.

4. Basic Life Cycle Cost Analysis

The analysis was conducted based on a period of 60 years in order to enable reasonable comparisons (Refer to Appendix). Archimedean screw pumps have a lifespan of 30 years and centrifugal pumps, a lifespan of 20 years respectively.

4.1 Option 1: Archimedean screw pumps

Table 5 shows the full life cycle cost over a 60-year period including capital, operational and maintenance costs for Option 1.

Table 5: Life cycle cost for option 1: Archimedean Screw Pumps

Item	Cost	NPV
Total Capital Life Cycle Cost	R132,488,347.96	R40,121,491.18
Total Operational Life Cycle Cost	R413,528,533.65	R26,080,397.73
Total Maintenance Life Cycle Cost	R5,659,079.80	R567,964.36
Total Life Cycle Cost	**R551,675,961.41**	**R66,769,853.27**

4.2 Option 2: Centrifugal pumps

Table 6 shows the full life cycle cost over a 60-year period including capital, operational and maintenance costs for Option 2.

Table 6 Life cycle cost for option 2: Centrifugal Submersible Pumps

Item	Cost	NPV
Total Capital Life Cycle Cost	R288,296,342.54	R57,936,899.07
Total Operational Life Cycle Cost	R578,939,947.11	R36,512,556.82

Total Maintenance Life Cycle Cost	R7,989,289.13	R801,832.04
Total Life Cycle Cost	**R875,225,578.78**	**R95,251,287.93**

Figure 2 shows the comparison between the two options in terms on NPVs for capital, operational, maintenance and total life cycle costs.

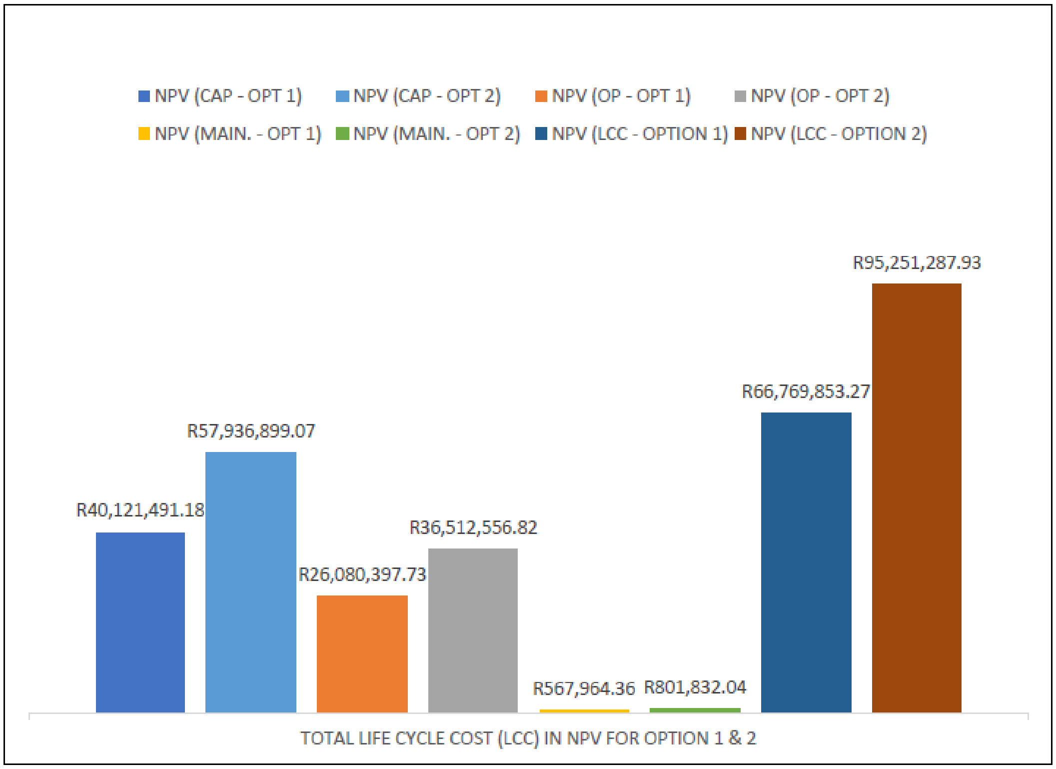


Figure 2: Graph showing comparative NPV costs for Option 1 & 2

It can be seen from *Figure 2* that Option 1 has a lower NPV for capital, operational and maintenance cost. The difference between the Total Life Cycle Cost is R28,481,434.66. Therefore, from a Basic Life Cycle Cost Analysis (LCCA), Option 1: Archimedean screw pumps is preferred.

5. Other Considerations

Other factors that need to be considered in the decision making process is as follows:

- Available space on site
  The available space on site is inadequate to accommodate the screw pump configuration of 3 screws. Boundary wall will need to be removed and encroachment into the adjacent property is inevitable. This is based on the dimensions (6.6m x 9m) of the sump that is required to accommodate the 3 screws comfortably with sufficient space for penstocks.
  It should be noted that the property adjacent to the pump station that will be encroached on, belongs to council however a hand plan will be required.

  Figure 3 shows a typical layout of Archimedean screw pump system as supplied by a recognised Archimedean screw supplier.

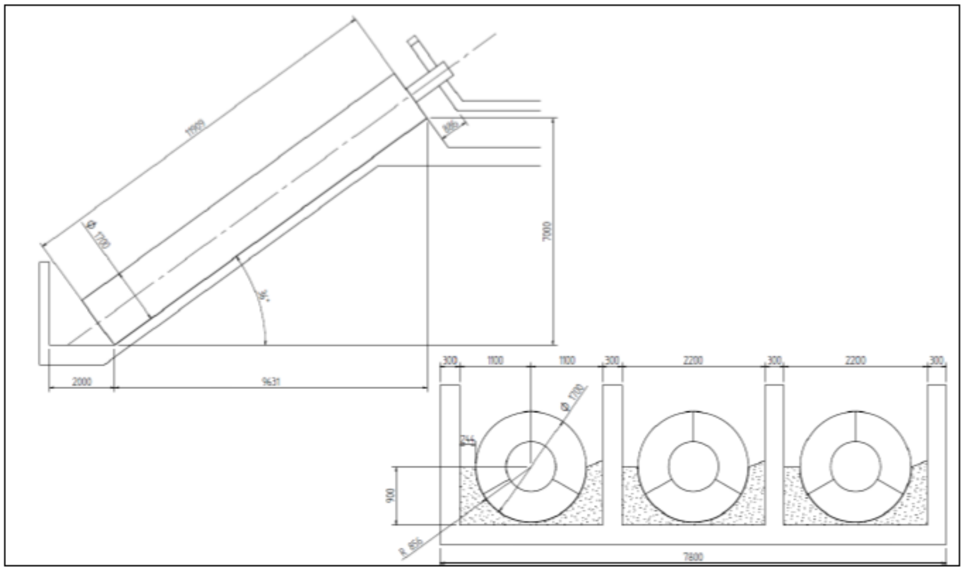
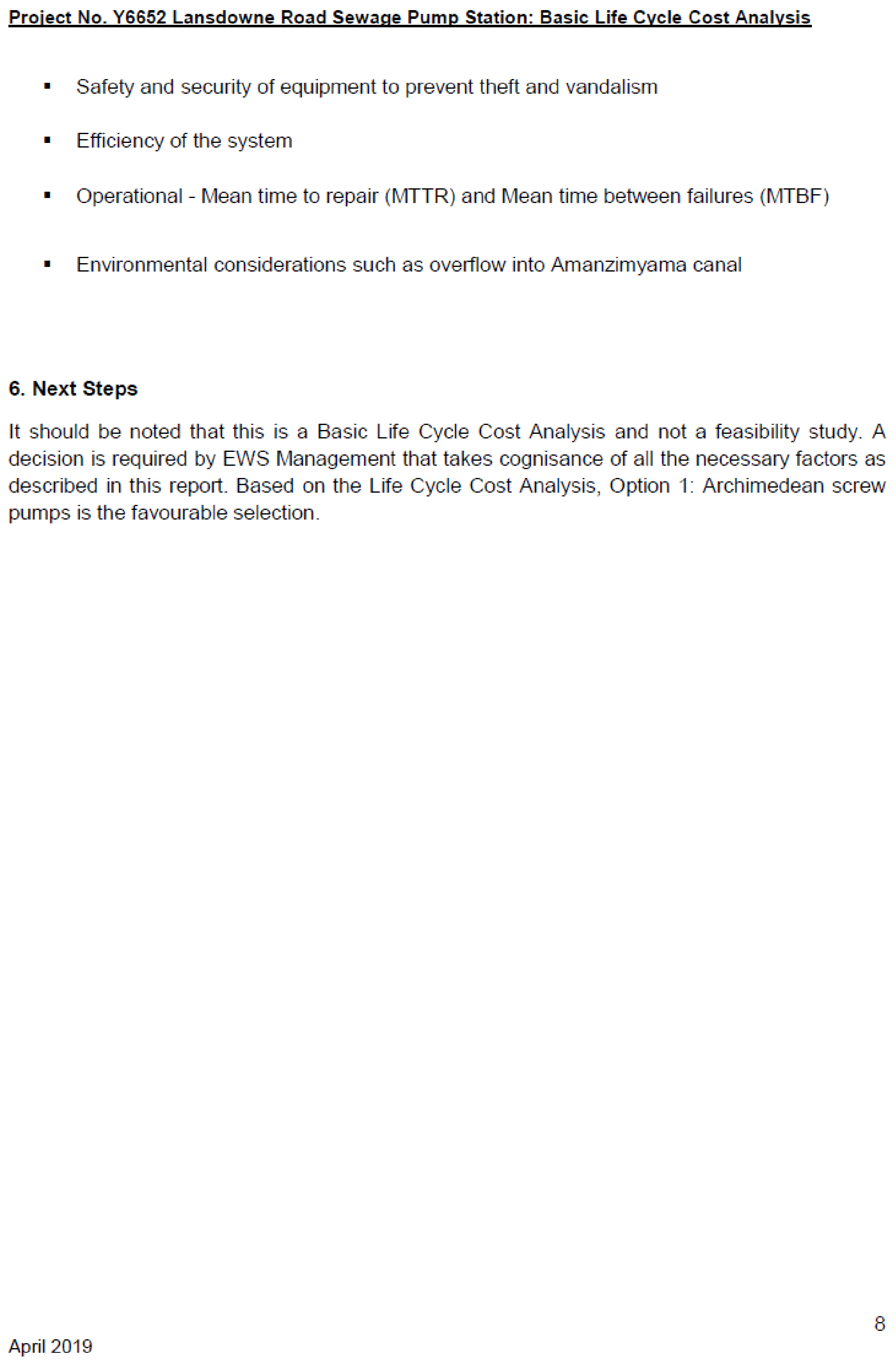


Figure 3: Showing space requirements for the Archimedean screws

- Advantages of Archimedean screw pumps
  - Longer lifespan
  - Lower maintenance
  - Dry running is acceptable

- Automation
  The system selected for the pump station should be one that can be fully automated.

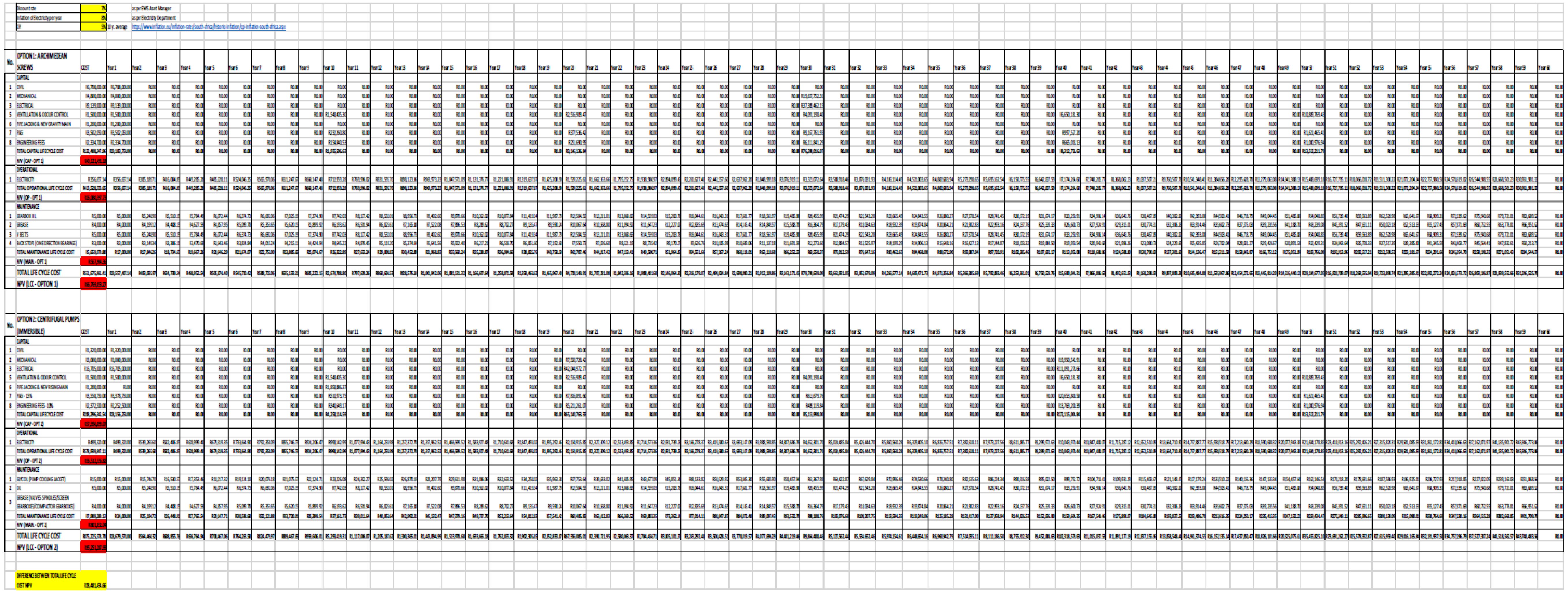


- Safety and security of equipment to prevent theft and vandalism
- Efficiency of the system
- Operational - Mean time to repair (MTTR) and Mean time between failures (MTBF)
- Environmental considerations such as overflow into Amanzimyama canal

6. Next Steps

It should be noted that this is a Basic Life Cycle Cost Analysis and not a feasibility study. A decision is required by EWS Management that takes cognisance of all the necessary factors as described in this report. Based on the Life Cycle Cost Analysis, Option 1: Archimedean screw pumps is the favourable selection.

7. Appendix: Basic Life Cycle Cost Analysis of the Options



Appendix D: Geotechnical Investigation

Geotechnical Investigation:

DRENNAN MAUD (PTY) LTD
GEOTECHNICAL ENGINEERS AND ENGINEERING GEOLOGISTS
Incorporating Drennan Maud & Partners (Est.1975)
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Our Ref : 32742
Your Ref : WQ 64/10188

9th September 2019

Ethekwini Municipality Water & Sanitation
Waste Water Design Department
3 Prior Road
DURBAN
4001

Attention: Ms. H. Joseph Email: Hope.joseph@durban.gov.za

Dear Madam,

GEOTECHNICAL INVESTIGATION FOR THE CONSTRUCTION OF A NEW SEWAGE PUMP STATION AND GRAVITY TRUNK SEWER AT LANSDOWNE ROAD, MOBENI (WQ 64/10188)

1. INTRODUCTION AND TERMS OF REFERENCE

Following the submission of tender documentation for the above mentioned project, Drennan Maud (Pty) Ltd was named the successful bider and was awarded the contract to carry out the required geotechnical investigation.

Drennan Maud (Pty) Ltd subsequently carried out the field testing component of the assessment with out findings and geotechnical assessment set down in this report along with our recommendations for the proposed development.

Directors: **M.J.F BENET** [Pr.Sci.Nat. B.Sc. (Hons) M.Sc. FSAIEG], **M.J.HADLOW** [Pr.Sci.Nat. B.Sc.(Hons.) MSAIEG]
Managers: **M.J.F. BENET** (Durban), **G. NTAKA** (Margate)
BEE RATED
B-BBEE LEVEL 4 CONTRIBUTOR

2. SUPPLIED AND AVAILABLE INFORMATION

Ethekwini Municipality supplied Drennan Maud (Pty) Ltd with a 1:500 scale contoured orthophotograph of the site area titled “Services Layout Drawing” which included, to the best available knowledge, the positions of services in the area, which include gas, water and sewerage pipelines as well as electrical and fibre optic cables.

The supplied drawing is included as the site plan of this report (Drawing 32742-01).

The locations of the test positions were predetermined by Ethekwini Municipality Engineers and indicated on a supplied hard copy of the drawing. The approximate co-ordinates of the test positions were determined from the co-ordinated drawing and are appended the site plan of this report.

No detailed drawings of the existing pump-station structure, existing sewer line as well as proposed new pump-station structures and gravity main pipeline were available for the geotechnical investigation at this stage of the development.

Notwithstanding the above, from the tender document, it is understood that the proposed development will comprise the following;

- Installation of three number Archimedean screw pumps at the existing pump station site.
- Construction of a sump and trash rack - structures may be as deep as 6m below existing ground level
- Construction of a gravity sewer (1500mm diameter) from the outlet of the pump-station to the connection manhole at the intersection of Lansdowne Road and Quality Street - a distance of some 160m. Pipeline invert level to be in the order of 4m below ground level and will be installed via pipe-jacking along a section of Quality street and via open trench excavation along the remainder of the pipeline alignment.

Drennan Maud (Pty) Ltd, formerly Drennan Maud and Partners has carried out several investigations in the Quality Street / Clairwood racecourse area and thus have information available to us regarding the prevailing geological and geotechnical conditions of the general area.

Most applicable of the information available is work carried out for the installation of the NMPP pipeline along a section of the Amanzimyama canal running along the western boundary of the site. The relevant field work comprises borehole drilling and hand auger excavations some 200 - 300m north east and south west of the pump-station site.

Furthermore, Drennan Maud (Pty) Ltd were involved in the installation of lateral support and an access ramp to the canal at a location to the immediate east of the pump-station site.

3. SITE DESCRIPTION

The layout of the site area can be appreciated from the site plan image accompanying this report as Drawing 32742-01.

The site is located in the area of Mobeni / Jacobs approximately 9km south west of the Durban CBD as the crow flies.

The investigated site comprises two areas, namely the existing pump-station and a section of Quality Street.

The pump-station site, currently accommodates a pump station and guard hut/ablution building on the central to south western and north eastern portions of the site respectively. The fenced site is entirely paved except for the grassed north western corner of the facility.

The pump-station site is bound by a road embankment which supports Quality street to the east and south, a factory/warehouse to the north and the Amanzimyama canal to the west. The earth embankment along the eastern boundary of the site pinches out towards a concrete retaining wall positioned along the southern boundary of the site - the retaining wall supporting the section of the Quality street spanning the Amanzimyama canal.

The pump-station site is relatively level across its entire area and is elevated at approximately 10 to 11 meters above mean sea level (amsl).

The Quality Street road section of the site comprises a portion of the main road (a double carriage-way road in both directions) as well as the north west - south east orientated limb of Quality street, this limb comprising a single lane road in both directions that connects the main road with Lansdowne Road to the east. Lansdowne Road runs parallel and directly adjacent the M4 highway.

The road embankment in the vicinity of the proposed pipeline route is elevated at approximately 16m amsl, decreasing to approximately 14m amsl at the junction with Lansdowne Road.

As alluded to in Section 2 above, several services exist below the investigated portion of Quality Street. Furthermore, the road is heavily trafficked by vehicles ranging from light motor cars, mid-sized vans and very large trailer carrying trucks.

4. METHODOLOGY

4.1 Information Review

Information obtained from the relevant nearby previous investigations mentioned in Section 2 above was reviewed as part of the current assessment.

4.2 Field Assessment

In compliance with the scope of works and Bill of Quanitites set out in the project tender document, the field work comprised the excavation of several hand augers as well as Dynamic Cone Penetrometer (DCP) probing as close as possible (within 1 - 3m depending on GPS accuracy at the time) of the predetermined positions identified across the pump-station site and pipeline alignment.

However, despite providing traffic accommodation, deemed suitable for the size and nature of the road, due to the large size and frequency of trucks utilising the section of Quality Street and the space they required to adequately turn/manouevre along the road, certain test positions could not be safely achieved and thus were relocated to the nearest and safest possible location along the road side, taking into consideration the position of existing services.

The original test positions as well as alternate positions are indicated on the site plan (Drawing 32742-01) of this report.

Hand Augers: A total of three hand augers (AH1 - 3) were manually excavated to maximum depths of 3.3m prior to being halted in saturated alluvial material due to poor recovery and continual sidewall collapse.

In addition, AH4 and AH5 were conducted along the grassed verge of Quality street and attained maximum depths ranging between 0.6 - 1.1m below existing ground level prior to refusal being met in dense gravelly fill material.

Dynamic Cone Penetrometer (DCP) Testing: A total of nine DCP probes, designated DCP 1 - 9, were carried out to determine the consistency of the subsoils underlying the various portions of the site.

DCP's 1 - 3 and DCP's 7 and 8, were conducted successfully at or within 1-3m of their predetermined positions, however, as alluded to above DCP's 4 - 6 were repositioned accordingly.

DCP's 1 - 3 and 9, located within/adjacent to the pump station facility were progressed to a maximum depth of 7.8m below existing ground level (EGL) without meeting refusal within the underlying in-situ material.

DCP's 4 - 8 located along the Quality Street portion of the site met with refusal in inferred dense fill material and/or on possible obstruction/horizons therein at depths ranging between 0.5 - 4.5m below existing ground level.

Material Sampling: Representative disturbed samples of the prevailing subsoil materials encountered on site were retrieved from the hand auger excavations and submitted for laboratory testing at Thekwini Soils Laboratory in Durban.

The material removed from the hand augers holes was examined and logged by a SACNASP registered professional in accordance with the Guidelines for Soil and Rock Logging in South Africa, 2nd Impression 2002, edited by RMH Bruin and ABA Brink with the resultant soil profile logs included in Appendix A of this report.

The DCP probe results are recorded in Appendix B of this report with Table 1 below included as a guide for the interpretation thereof with regards to number of blows required to progress the probe and the consistency of the cohesive and non-cohesive material.

The results of the laboratory testing conducted are recorded in Appendix C of this report and further summarised and discussed in Section 6 of this report.

Table 1: Guideline to Interpreting Drennan Maud's DCP Test Results

Non Cohesive Soils		Cohesive Soils	
№ Of Blows/300mm Penetration	**Subsoil Consistency**	№ 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

5. GEOLOGY, SOILS AND GROUNDWATER

5.1 Regional Geology

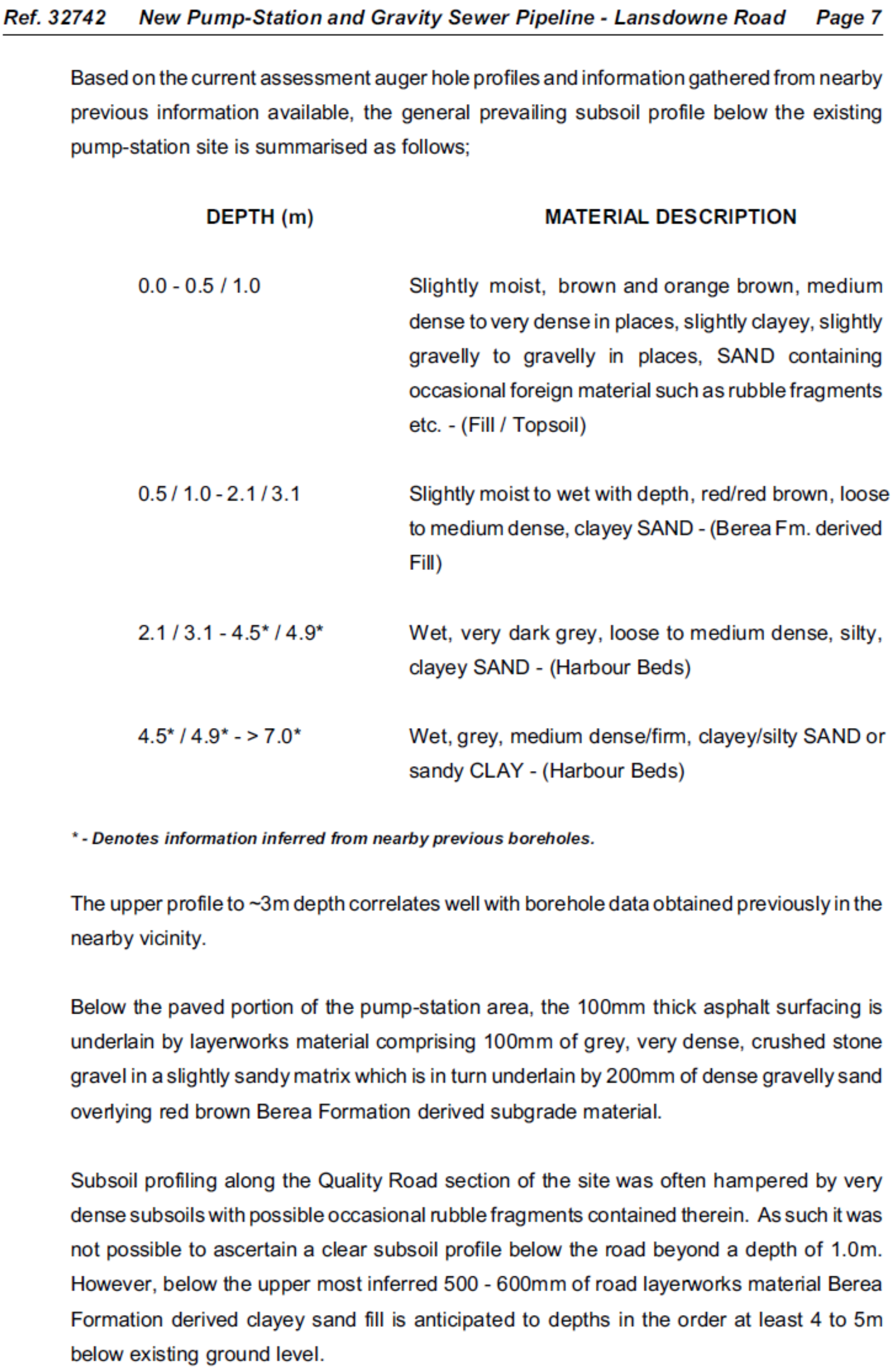
The site and immediate surrounding area is underlain by late Quaternary alluvial and estuarine sediments which form part of the commonly known "Harbour beds" in the Durban area.

The sand, sandy clay to clayey/silty sand as well as highly compressible clays and silts which comprise the Harbour Beds have been deposited in the environs of meandering river deltas and estuaries, thus the occurrence of subsoil materials may vary both laterally and vertically due to the variations in the depositional mechanisms.

Given the developed nature of the site, Berea Formation derived clayey sand fill material, likely sourced from nearby elevated areas to the east has been placed on top of the in-situ Harbour beds as a reclamation fill for the creation of building platforms and road embankments.

4.1 Prevailing Subsoils

The inferred geology underlying the entire investigated area is depicted in the geological section A-A included as Drawing 32742-02 of this report.



Based on the current assessment auger hole profiles and information gathered from nearby previous information available, the general prevailing subsoil profile below the existing pump-station site is summarised as follows;

DEPTH (m)	MATERIAL DESCRIPTION
0.0 - 0.5 / 1.0	Slightly moist, brown and orange brown, medium dense to very dense in places, slightly clayey, slightly gravelly to gravelly in places, SAND containing occasional foreign material such as rubble fragments etc. - (Fill / Topsoil)
0.5 / 1.0 - 2.1 / 3.1	Slightly moist to wet with depth, red/red brown, loose to medium dense, clayey SAND - (Berea Fm. derived Fill)
2.1 / 3.1 - 4.5* / 4.9*	Wet, very dark grey, loose to medium dense, silty, clayey SAND - (Harbour Beds)
4.5* / 4.9* - > 7.0*	Wet, grey, medium dense/firm, clayey/silty SAND or sandy CLAY - (Harbour Beds)

- Denotes information inferred from nearby previous boreholes.

The upper profile to ~3m depth correlates well with borehole data obtained previously in the nearby vicinity.

Below the paved portion of the pump-station area, the 100mm thick asphalt surfacing is underlain by layerworks material comprising 100mm of grey, very dense, crushed stone gravel in a slightly sandy matrix which is in turn underlain by 200mm of dense gravelly sand overlying red brown Berea Formation derived subgrade material.

Subsoil profiling along the Quality Road section of the site was often hampered by very dense subsoils with possible occasional rubble fragments contained therein. As such it was not possible to ascertain a clear subsoil profile below the road beyond a depth of 1.0m. However, below the upper most inferred 500 - 600mm of road layerworks material Berea Formation derived clayey sand fill is anticipated to depths in the order at least 4 to 5m below existing ground level.

It is considered possible that the Berea Formation fill material and underlaying in-situ Harbour Beds sediment may be separated by a pioneer layer potentially comprising course gravel/rubble/brick fragments. Additional excavations on site would be required to confirm the presence of such a potential pioneer layer if present.

4.2 Subsoil Consistency

DCP's 1 - 3 and 9 conducted within the Pump house facility reveal medium dense to occasionally dense conditions to depths ranging between 1.5 - 3.0m below existing ground level, thereunder becoming loose to depths ranging between 4.2 - 4.5m, with the exception of DCP1 in which loose conditions occur in a thin band between 1.5 - 2.1m depth.

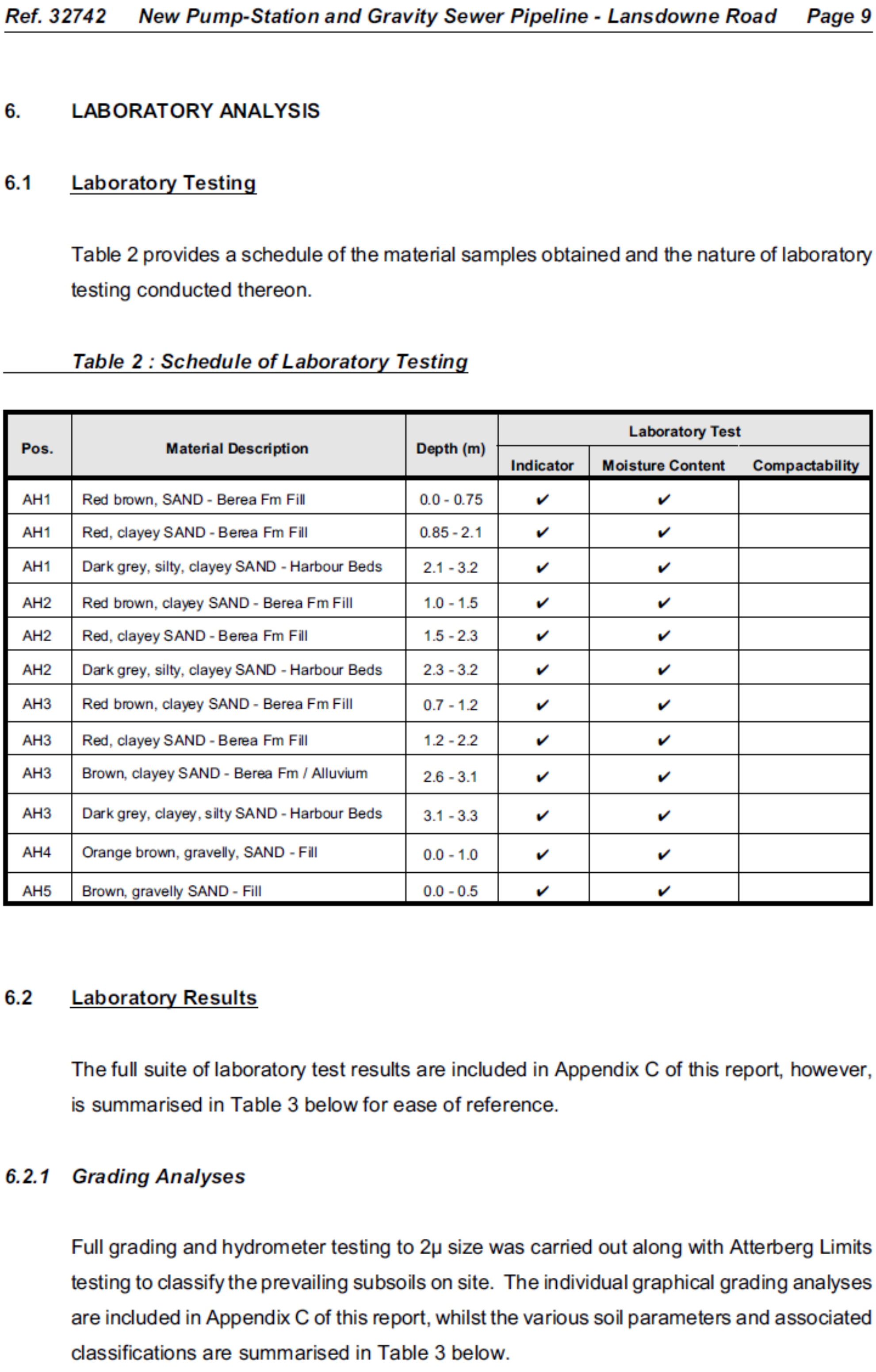
Below the above mentioned depths medium dense conditions prevail in the Harbour beds sediment to maximum probed depths of 7.8m below existing ground level.

Contrasting with the above, DCP's 4 - 8 carried out along the road section of the site generally encountered dense to very dense conditions in the upper layerworks materials, thereunder the inferred Berea Formation derived fill ranging between medium dense to very dense prior to DCP refusal being met at depths ranging between 0.5 - 4.5m depth.

As alluded to above, the refusal of DCP 7 at a depth of 4.5m depth is inferred on a possible pioneer layer horizon at the base of the Berea Formation derived fill material.

4.3 Groundwater Seepage

Ground water seepage occurs at depths ranging between 2.4 - 2.7m below existing ground level across the existing pump station facility, and generally coincides with the contact of the Berea Formation fill and underlying saturated Harbour Beds sediment. No ground water seepage was encountered below the relatively well elevated road section of the investigated area.



6. LABORATORY ANALYSIS

6.1 Laboratory Testing

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

Table 2 : Schedule of Laboratory Testing

Pos.	Material Description	Depth (m)	Laboratory Test		
			Indicator	Moisture Content	Compactability
AH1	Red brown, SAND - Berea Fm Fill	0.0 - 0.75	✔	✔	
AH1	Red, clayey SAND - Berea Fm Fill	0.85 - 2.1	✔	✔	
AH1	Dark grey, silty, clayey SAND - Harbour Beds	2.1 - 3.2	✔	✔	
AH2	Red brown, clayey SAND - Berea Fm Fill	1.0 - 1.5	✔	✔	
AH2	Red, clayey SAND - Berea Fm Fill	1.5 - 2.3	✔	✔	
AH2	Dark grey, silty, clayey SAND - Harbour Beds	2.3 - 3.2	✔	✔	
AH3	Red brown, clayey SAND - Berea Fm Fill	0.7 - 1.2	✔	✔	
AH3	Red, clayey SAND - Berea Fm Fill	1.2 - 2.2	✔	✔	
AH3	Brown, clayey SAND - Berea Fm / Alluvium	2.6 - 3.1	✔	✔	
AH3	Dark grey, clayey, silty SAND - Harbour Beds	3.1 - 3.3	✔	✔	
AH4	Orange brown, gravelly, SAND - Fill	0.0 - 1.0	✔	✔	
AH5	Brown, gravelly SAND - Fill	0.0 - 0.5	✔	✔	

6.2 Laboratory Results

The full suite of laboratory test results are included in Appendix C of this report, however, is summarised in Table 3 below for ease of reference.

6.2.1 Grading Analyses

Full grading and hydrometer testing to 2µ size was carried out along with Atterberg Limits testing to classify the prevailing subsoils on site. The individual graphical grading analyses are included in Appendix C of this report, whilst the various soil parameters and associated classifications are summarised in Table 3 below.

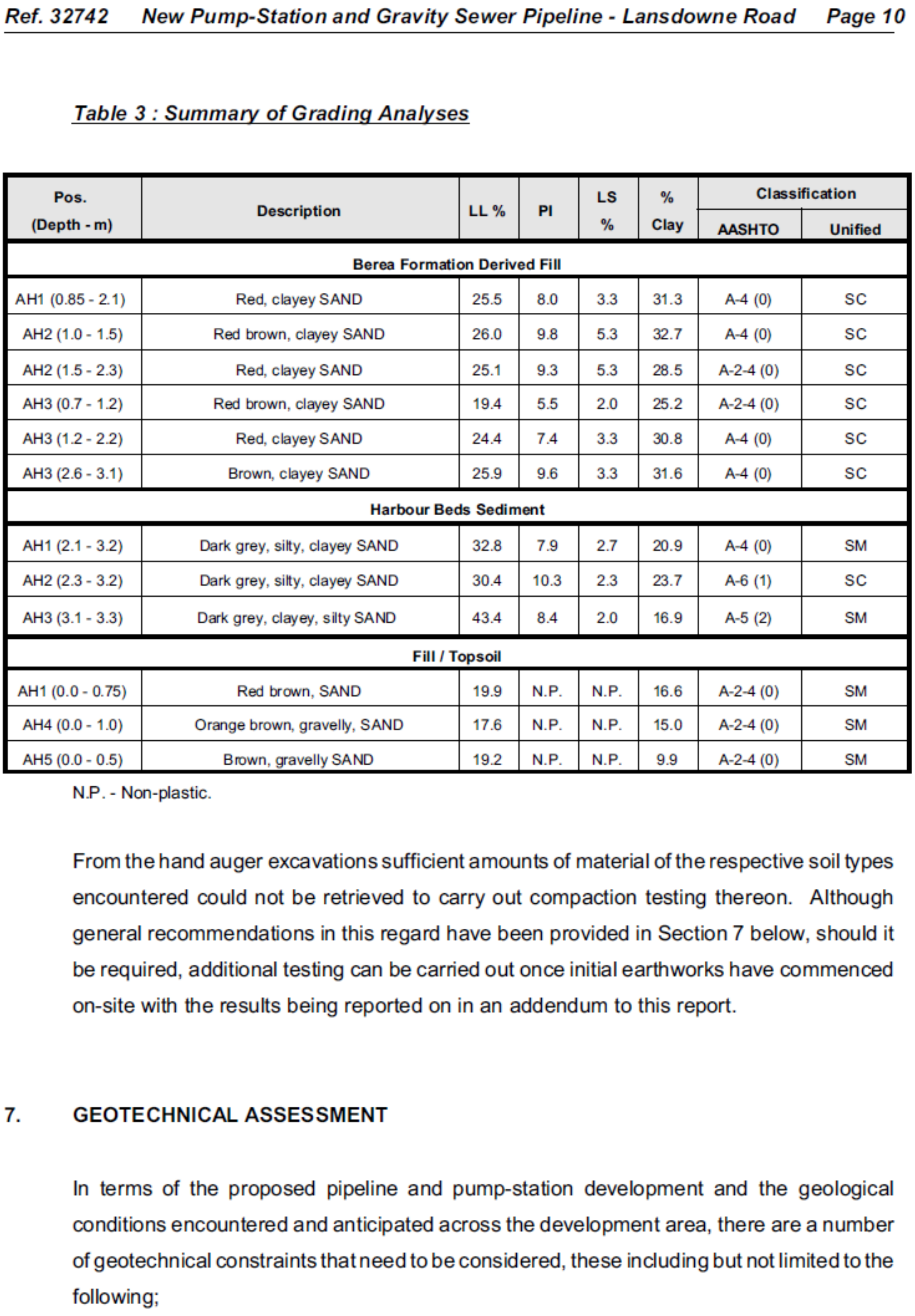


Table 3 : Summary of Grading Analyses

Pos. (Depth - m)	Description	LL %	PI	LS %	% Clay	Classification	
						AASHTO	Unified
Berea Formation Derived Fill							
AH1 (0.85 - 2.1)	Red, clayey SAND	25.5	8.0	3.3	31.3	A-4 (0)	SC
AH2 (1.0 - 1.5)	Red brown, clayey SAND	26.0	9.8	5.3	32.7	A-4 (0)	SC
AH2 (1.5 - 2.3)	Red, clayey SAND	25.1	9.3	5.3	28.5	A-2-4 (0)	SC
AH3 (0.7 - 1.2)	Red brown, clayey SAND	19.4	5.5	2.0	25.2	A-2-4 (0)	SC
AH3 (1.2 - 2.2)	Red, clayey SAND	24.4	7.4	3.3	30.8	A-4 (0)	SC
AH3 (2.6 - 3.1)	Brown, clayey SAND	25.9	9.6	3.3	31.6	A-4 (0)	SC
Harbour Beds Sediment							
AH1 (2.1 - 3.2)	Dark grey, silty, clayey SAND	32.8	7.9	2.7	20.9	A-4 (0)	SM
AH2 (2.3 - 3.2)	Dark grey, silty, clayey SAND	30.4	10.3	2.3	23.7	A-6 (1)	SC
AH3 (3.1 - 3.3)	Dark grey, clayey, silty SAND	43.4	8.4	2.0	16.9	A-5 (2)	SM
Fill / Topsoil							
AH1 (0.0 - 0.75)	Red brown, SAND	19.9	N.P.	N.P.	16.6	A-2-4 (0)	SM
AH4 (0.0 - 1.0)	Orange brown, gravelly, SAND	17.6	N.P.	N.P.	15.0	A-2-4 (0)	SM
AH5 (0.0 - 0.5)	Brown, gravelly SAND	19.2	N.P.	N.P.	9.9	A-2-4 (0)	SM

N.P. - Non-plastic.

From the hand auger excavations sufficient amounts of material of the respective soil types encountered could not be retrieved to carry out compaction testing thereon. Although general recommendations in this regard have been provided in Section 7 below, should it be required, additional testing can be carried out once initial earthworks have commenced on-site with the results being reported on in an addendum to this report.

7. GEOTECHNICAL ASSESSMENT

In terms of the proposed pipeline and pump-station development and the geological conditions encountered and anticipated across the development area, there are a number of geotechnical constraints that need to be considered, these including but not limited to the following;

7.1 Problem Soils

The area in general is underlain by variably thick silty clay and sensitive sands that are highly susceptible to consolidation and settlement when surcharged. Unlike sand material which undergoes immediate settlement upon loading, these compressible clays/silts inferred at depths in excess of 4.5m below existing ground level take considerable time to dissipate excess pore water pressure when consolidating under load.

The amount of settlement is largely dependant on the magnitude and distribution of the surcharge load with the settlement rate being strongly time-dependant.

Furthermore the saturated alluvial material overlying the potentially compressible clay at depth is generally loose and thus may be prone to collapse settlement under an applied load with a critical fluctuation in the materials in-situ moisture content.

The generally upper sandy fill material will likely be highly susceptible to erosion by both flowing water and/or wind forces and thus will need to be carefully managed during construction.

7.2 Excavatability/Trenchability

Open trench excavation within the upper Berea Formation derived fill and underlying in-situ Harbour Beds clayey sandy sediment along the proposed pipeline route as well as pump station facility will classify as 'soft' excavation through out as defined by SANS 1200D standards.

Furthermore, pipe jacking below a portion of the Quality Street road embankment within the Berea Formation derived fill is considered to classify similarly however, occasional rubble and foreign materials considered potentially contained therein, if encountered, may hinder progress somewhat and thus should be accounted for.

7.3 Groundwater Seepage

Ground water seepage is likely to be encountered below the lower lying pump station facility at depths ranging between 2 - 3m below existing ground level at or just above the contact of the upper Berea Formation fill material and underlying Harbour Beds sediment.

Permanent ground water seepage is not anticipated within the scope of the pipe-jacking operation given the relatively well elevated nature of the road embankment however, due to the clayey sandy nature of the material of which it is likely largely comprised, a temporary perched ground water seepage may be encountered, especially after rainfall events or in the wetter summer months.

Towards the junction of Quality Street and Lansdowne Road the depth to the perched ground water table is unknown, but is likely to coincide with the contact of the upper fill and in-situ material, inferred at a depth in the order of 3 - 4m below existing ground water. This should be potentially confirmed with additional localised testing.

Should the proposed new pipeline be located below the depth of the perched ground water table it will be necessary to incorporate certain buoyancy control measures during the installation of the pipeline.

7.4 Trench/Excavation Sidewall Stability

Subsoil excavations within the existing pump station property, in particular within the low cohesion silty sandy Harbour Beds sediment are considered unstable and prone to collapse, especially where saturated below the water table at depths typically greater than 2m below existing ground level. Hence, temporary lateral support will be required to prevent sidewall collapse of excavations on site, in particular deep excavations for the installation of the proposed sump/s.

Although the typically clayey sand Berea Formation fill material is likely to stand, near vertical, upon initial excavation, the material is prone to potential collapse upon either drying out or becoming saturated. As such excavations therein and the overlying sandier variant of the fill should be offered lateral support (i.e. shoring) in open excavations and pipe trenches.

7.5 Material Suitability

7.5.1 General Earthworks and Subgrade Material

Based on the laboratory test results the clayey silty sand Harbour Beds material encountered below the pump station site classifies generally as A-4 to A-6 (SM - SC) material after the AASHTO and Unified classification systems respectively.

If/where removed/encountered it is unlikely the saturated material will classify as suitable for use as bulk fill or subgrade material.

The clayey sand Berea Formation derived fill material classifying as A-2-4 to A-4 (SC) type material is considered suitable for reuse as bulk fill or as subgrade material, likely generally attaining a G10 classification after TRH 14 - 1985, but may potentially classify as G10+ if more clayey variants are locally encountered.

The gravel material underlying paved/asphalted areas is likely to classify as A-1-b, G5/G6 material and thus where encountered, should be removed and stockpiled for later reinstating of the road, provided it is removed without being contaminated with underlying lower quality material.

7.5.2 Pipe Bedding Material

According to SABS 1200 LB standards pipe bedding material requirements should conform to the following;

a) Selected Granular Material: Non-cohesive, singularity graded between 0.6mm - 19mm, having a compactability factor not exceeding 0.4.

b) Selected fill material: Material with a plasticity index not exceeding 6 and free of vegetation and lumps or stones exceeding 30mm.

In terms of the above the suitability of the typical materials encountered on site is as follows;

The clayey sandy Berea Formation material as well as in-situ Harbour Beds sediment, in addition to being cohesive, will not meet the minimum grading requirements for selected granular material and thus is precluded from being used as such despite no compactability testing being conducted thereon.

The upper sandy fill material, although non-plastic, will not fall within the singularly graded envelope required for consideration as selected granular material.

In terms of use as selected fill material the Berea Formation fill material generally exceeds the lower plasticity index limit of 6 and thus is considered unsuitable, whilst the upper non-plastic general fill material may be considered acceptable should all vegetation and oversized particles be screened out.

7.6 Settlement Analyses

Due to the generally granular nature of the upper fill and Harbour Beds material underlying the pump station site foundation design for any new structures should be based on tolerable settlement and not ultimate bearing capacity failure. As no indication of potential structural loading was available at this preliminary stage, indicative settlement analyses were carried out based on the lower bound (worst case) DCP information obtained during the field investigation for a range of column base foundation dimensions and strip footing widths at a foundation depth of 0.9m and at allowable bearing pressures of 100 kPa and 150 kPa. The results of the analyses are summarised in Table 4 below;

Table 4: Summary of indicative settlement amount

Foundation type	Dimensions (m)	Indicative Load (kN)	Allowable Bearing Pressure (kPa)	Anticipated Settlement (mm)
Column Bases	1.73 x 1.73	300	100	10
	1.41 x 1.41		150	13
	2.45 x 2.45	600	100	14
	2.0 x 2.0		150	18
	3.0 x 3.0	900	100	17
	2.45 x 2.45		150	21
Strip Footing	0.7 wide	70	100	10
		105	150	15
	0.9 wide	90	100	12
		135	150	18

The indicative settlement analyses indicate that total settlement amounts in the order of 10 - 17mm and 13 - 21mm can be expected for allowable bearing pressures of 100 and 150kPa respectively. Considering differential settlement as 75% of total settlement for sandy material differential settlement amounts are likely to range between 8 - 16mm.

In mitigation of the above, due to the sandy nature of the upper fill material it is likely that a component of the above mentioned anticipated settlement will occur during initial construction.

8. DEVELOPMENT RECOMMENDATIONS

Notwithstanding the above geotechnical constraints associated with the prevailing subsoils, the intended development is considered geotechnically feasible, provided the development is carried out in terms of the recommendations provided below.

These amount to no more than sound building practices appropriate for the subsoil conditions occurring on site and the envisaged scope of the development.

8.1 Earthworks

The existing pump station site is relatively level and as such it is envisaged that only minor earthworks will be required for the development, with the exception of the sump pit construction to depths in the order of 6m.

Foundation or services trenches in the upper sandy and clayey sand fill greater than 1.0m depth should be battered back to a maximum temporary batter of 1:1,5 (33°) or alternatively shored, especially where in close proximity to existing structures or services that may be potentially undermined. The design and installation supervision of such shoring should be carried out by an experienced Structural/Geotechnical Engineer.

With regards to trench earthworks along Quality Street cognisance must be paid to suitably accommodate the existing services, in particular, the gas pipeline along the southern curb of the road.

The prescribed maximum batter of 1:2 (26°) in the in-situ Harbour Beds sediment is likely to be impractical and thus where intersected, especially below the water table, sidewalls exposing this material will need to be suitable shored/retained to prevent sidewall collapse therein.

Where filling/backfilling is required, suitable granular fill material (G10 or better) should be placed in layers of 300mm loose thickness and compacted to 95% Mod AASHTO density prior to the placement of the next layer.

Prior to the placement of any fill the in-situ fill/Harbour Beds sediment should be scarified and compacted to 93% Mod AASHTO density.

Vibratory compaction of backfill material at the base of excavations at or near the perched water table (~2m) may lead to temporary conditions of heaving (pumping) due to a build up of excess pore water pressure. If encountered, in the event that a short rest period does not alleviate the problem, the following two options are recommended;

- Build up the lowest portion of the fill with lighter compaction equipment in thin successive layers or,
- Construct a gravel/sand pioneer layer at the base of the excavation to allow for dissipation of pore water pressure during compaction.

8.2 Site Drainage

Control of stormwater drainage is essential, and stormwater from all new and existing roof and paved areas within the pump house facility should be piped or carried in surface drains to discharge into the stormwater system provided for in the area.

Post construction the site should be graded such that stormwater is carried effectively into drains and prevented from ponding at the surface adjacent to structures.

During construction stormwater run-off should be prevented from topping the cuff of any open trench excavation.

Where seepage is encountered in shallow excavations (service/foundation trenches), this should be dealt with symptomatically as and when it occurs. Should seepage persist a sump and pump means of ground water seepage management is considered the most feasible option.

8.3 Dewatering

However, with regards to deep excavations on site for the sump structure, temporary dewatering during construction will be required. As such provision for well point installation is recommended. Depending on the depth of the perched ground water table and depth of the new sewer pipe, dewatering may also be required along the lower lying, eastern portion of Quality street to maintain dry conditions within the open pipe trench.

8.4 Lateral Support

With regard to temporary / permanent lateral support during the construction of the sump structure, consideration could be lent to the installation of deep sheet piles, Soldier ‘H’ piles with horizontal lagging or alternatively the construction of a secant piled wall.

The advantage of the piled wall is that the temporary lateral support could be incorporated into the permanent lateral support of the pit structure.

The lateral support system/s adopted on site should be designed according to the following typical strength characteristics inferred for the Harbour Beds clayey, silty sand and clayey sand fill material;

Table 5 : Typical Subsoil Shear Strength Parameters

Material	**Cohesion ‘C’ (kPa)**	**Angle of Internal Friction ‘Phi’ (Deg)**	**Unit Weight (kN/m^3)**
Berea Fm. Clayey Sand	0	31	1900
Silty Sand / Clayey Sand - Harbour Beds	0	28	1850
Silty Clay	3	23	1750

The lateral support system should be design by an experienced Structural/Geotechnical Engineer familiar with such materials and structures and similarly, constructed by a contractor suitably experienced in this regard.

The base of the sump pit should be designed accordingly to account for uplift buoyancy forces.

8.5 Founding

No details of the proposed structures on site have been provided at this preliminary stage.

Nonetheless, based on the theoretical settlement analyses discussed in Section 7.6 above, if the total, and more importantly differential settlement amounts, are deemed by the project Structural Engineer to fall within tolerable limits, it is considered feasible that the proposed structures associated with the new pump station development are founded at shallow depths within the upper fill on either conventional strip footings or ground beams spanning column bases.

In this regard the base of the foundation excavations should be wetted and tamped via pneumatic whacker to create a uniformly dense founding medium and induce any potential collapse potential in the upper clayey sandy fill.

It is recommended that the allowable bearing pressure for foundation design is restricted to between 100 - 150 kPa in the upper Berea Formation derived fill material.

Alternatively, in the event relatively compact structures are proposed on site, these may be feasibly founded on a suitably designed and stiffened raft foundation if preferred.

In the event anticipated structural loading is much higher and thus will likely result in excessive settlement amounts, it is recommended that the structure/s are founded on reinforced ground beams spanning pile foundations

Due to the shallow water table occurring in the area, we consider that pressure grout injected CFA piles are the most suitable. The design of the piles should be carried out by a Geotechnical / Structural Engineer familiar with the conditions occurring on site. Given the anticipated depth to bedrock below the site piles would need to be designed to act in skin friction and not end-bearing.

With the exception of any structures founded on a raft, the floor slab of new structures should be isolated from all walls, columns and foundations to allow for settlement with time. Similarly, new and existing structures should be isolated to ensure independent settlement of one another.

The sump pit should be fully tanked or incorporate water-bars to avoid ingress of ground water into the sump or egress of effluent into the ground water table.

8.6 Pipe-Jacking

With regards to pip-jacking below Quality Street it is recommended the following considerations are taken into account.

At the pipe-jack location sufficient space will likely be available to the north west of the road fill embankment however, limited space will be available along the Quality street intersection and thus it is recommended that the north west - south east trending limb of Quality street be completely closed and traffic accordingly diverted during the course of the sewer pipe installation.

In this regard we would recommend that the launching pit is placed on the lower lying north western portion of the road embankment with the relatively smaller receiving pit on the south eastern side to maximise the available space.

In the event a thrust wall is required from which to launch the pipe-jack this should be designed by an experienced Structural Engineer familiar with such structures and the prevailing subsoils. The founding of such a thrust wall should be based on the recommendations provided in Section 8.5 above.

Suitable shoring of receiving/jacking pits will be necessary especially within the fill and unconsolidated sandy sediment to prevent the collapse of excavation sidewalls. In this regard measures suggested in Section 8.4 should be considered.

9. CONCLUSION

The site is underlain at depths in excess of 15m below existing ground level by weathered tillite or sandstone bedrock, which is directly overlain by thick accumulations of sandy, clayey and silty sediment known as the Harbour Beds. The Harbour bed sediment is in turn overlain by a 2.0 - 3.0m thick mantle of clayey sand Berea Formation derived fill material present to existing ground level across the existing pump station site with the adjacent Quality street road embankment inferred to largely comprise of similar material. A shallow ground water table occurs at depths ranging between approximately 2.4 - 2.7m below current ground level within the pump station facility. The depth to the water table below the Quality street road embankment is inferred to be in the order of 6 - 7m depth, decreasing in a easterly direction along the eastern most portion of Quality street to an inferred depth of 3 - 4m at the intersection of Lansdowne Road. It should be noted that these depths are inferred and have not been confirmed visually or via field testing.

Notwithstanding the findings of the geotechnical investigation the proposed development of the new pump station and installation of gravity sewer line is considered feasible provided the geotechnical considerations and recommendations provided in this report are consulted and adhered to during the design and construction of the proposed development.

Taking into account the nature of the sump structure, required earthworks/trenching and nature of the prevailing subsoils/groundwater table, lateral support of excavations will be required.

In this regard it is recommended deep excavations be laterally supported by a suitably designed secant piled wall or by driven sheet/soldier piles, whilst open trenches should be suitably shored and/or braced. Dewatering measures will be required during the construction process, especially across lower lying portions of the development area.

It is considered feasible to found proposed new pump station structures on shallow strip footings or ground beams spanning column bases provided the theoretical settlement amounts provided in this report are deemed tolerable by the Structural Engineer.

10. REPORT LIMITATIONS

The ground conditions described in this report refer specifically to those encountered in the subsoil excavations and DCP probes. Given the nature of the geological environment in which the subsoils were deposited it is therefore quite possible that conditions at variance with those in the excavations/DCP probes could be encountered elsewhere on site during pipeline and pump station construction. The information in this report is given in good faith, as an indication of materials and conditions likely to be encountered during construction along the pipeline route. There is no warranty that the information is totally representative of the whole route and no responsibility will be accepted for any consequences arising from actual conditions being different from those indicated in this document.

We trust that this meets with your immediate requirements and will be pleased to furnish you with any additional information you may require.

Yours faithfully,
DRENNAN MAUD (PTY) LTD

AJoubert.

A. JOUBERT Pr.Sci.Nat.
/aj

Enclosures:
Appendix A - Subsoil Profiles
Appendix B - DCP Results
Appendix C - Laboratory Test Results
Drawing 1 - Site Plan
Drawing 2 - Inferred Geological Section A-A

APPENDIX A

Soil Profiles

(AH1 - 5)

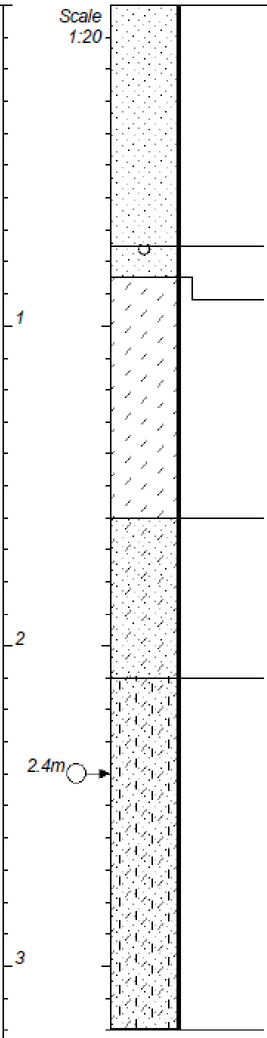
DRENNAN MAUD (PTY) LTD

Geotechnical Engineers & Engineering Geologists

ETHEKWINI WATER AND SANITATION
LANSDOWNE ROAD – NEW SEWAGE PUMPSTATION

***HOLE No:* AH1**
Sheet 1 of 1

***JOB NUMBER:* 32742**



0.00	Slightly moist, brown and reddish brown, dense to medium dense, slightly clayey, slightly gravelly in places, SAND – (Fill)
0.75	Slightly moist, dark brown, medium dense, gravelly SAND – (Fill)
0.85	Slightly moist, red brown, medium dense, clayey SAND containing lenses of fine to medium sand – (Berea Fm derived fill)
1.60	Slightly moist to moist, red brown, loose to medium dense, clayey SAND becoming less clayey with depth – (Berea Fm derived Fill)
2.10	Moist to wet, very dark grey, soft to firm, silty, clayey, SAND – (Estuarine/Alluvium)
3.20	

I,
MC

I,
MC

I,
MC

NOTES

1) No hand auger refusal encountered – hole stopped due to sidewall collapse and poor recovery in very wet alluvial material at 3.2m

2) Samples taken for laboratory testing

3) Slight ground water seepage encountered from approximately 2.4m depth

SAMPLE *DEPTH (m)*

CONTRACTOR :
MACHINE : HAND AUGER
DRILLED BY : NA
PROFILED BY : A.J
TYPE SET BY : AJ
SETUP FILE : DMPSP.SET

INCLINATION : NA
DIAM : NA
DATE : NA
DATE : 14-15/08/2019
DATE : 02/09/19 15:32
TEXT : ..C:\DOTIN\SPMASTER.DOC

ELEVATION :
X-COORD :
Y-COORD :

***HOLE No:* AH1**

D06B DRENNAN MAUD & PARTNERS

dot.PLOT 5008 J&W

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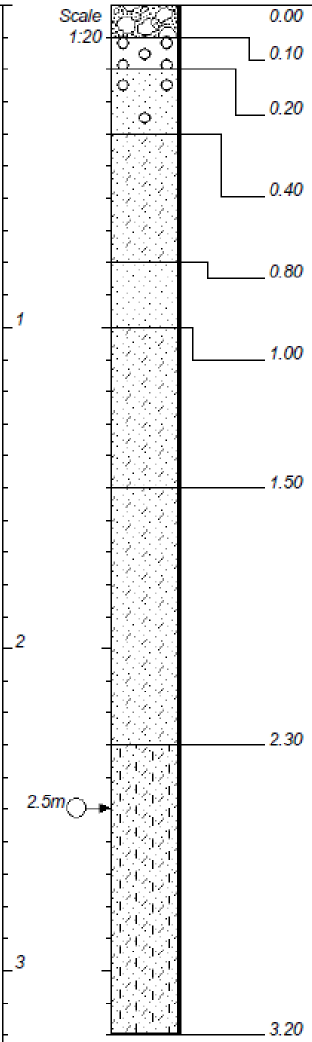
ETHEKWINI WATER AND SANITATION
LANSDOWNE ROAD – NEW SEWAGE PUMPSTATION

***HOLE No*: AH2**
Sheet 1 of 1

***JOB NUMBER*: 32742**



0.00 – 0.10	<u>ASPHALT</u> – (Paving)
0.10 – 0.20	Slightly moist, grey, very dense, slightly sandy <u>GRAVEL</u> – (Paving Layerworks)
0.20 – 0.40	Slightly moist, brown to reddish brown and grey, very dense, <u>gravelly SAND</u> – (Paving Layerworks)
0.40 – 0.80	Slightly moist, red brown, medium dense to loose, <u>clayey SAND</u> containing occasional lenses of fine to medium sand – (Berea Fm derived Fill)
0.80 – 1.00	Slightly moist, orange brown and red brown, loose, slightly clayey, fine to medium <u>SAND</u> – (Fill)
1.00 – 1.50	Slightly moist to moist, red brown, medium dense, <u>clayey SAND</u> – (Berea Fm derived Fill)
1.50 – 2.30	Moist to slightly wet, brown to reddish brown, medium dense, <u>clayey SAND</u> – (Berea Fm derived fill)
2.30 – 3.20	Wet, very dark grey, medium dense to loose with depth, <u>silty, clayey SAND</u> – (Alluvium/Estuarine)

SAMPLE	DEPTH (m)
I, MC	
I, MC	
I, MC	

NOTES

1) No hand auger refusal encountered – hole stopped due to sidewall collapse and poor recovery in very wet alluvial material at 3.2m

2) Samples taken for laboratory testing

3) Slight ground water seepage encountered from approximately 2.5m depth

CONTRACTOR :
MACHINE : HAND AUGER
DRILLED BY : NA
PROFILED BY : A.J

TYPE SET BY : AJ
SETUP FILE : DMPSP.SET

INCLINATION :
DIAM : NA
DATE : NA
DATE : 14-15/08/2019

DATE : 02/09/19 15:32
TEXT : ..C:\DOTIN\SPMASTER.DOC

ELEVATION :
X-COORD :
Y-COORD :

***HOLE No*: AH2**

D06B DRENNAN MAUD & PARTNERS

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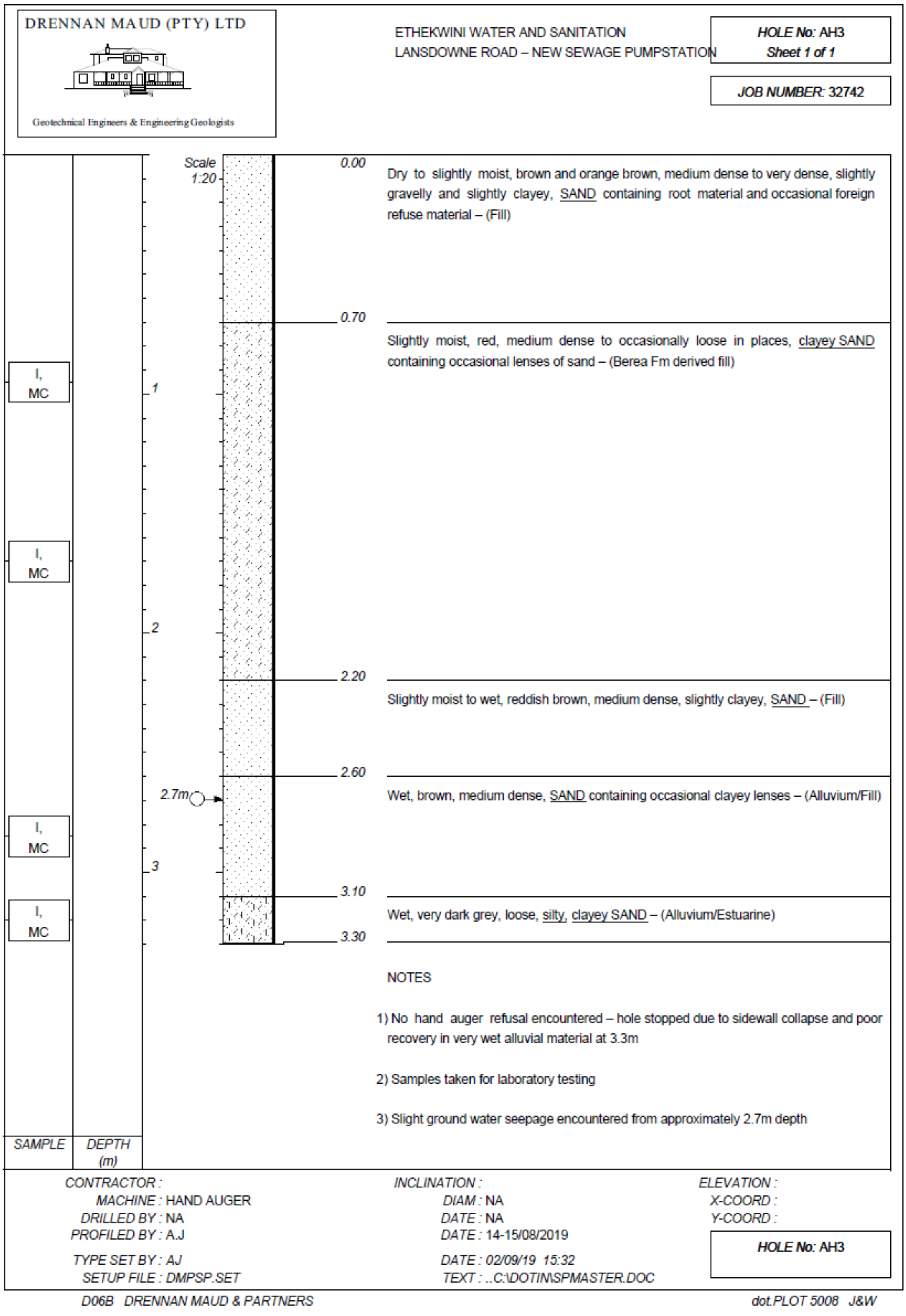

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ETHEKWINI WATER AND SANITATION
LANSDOWNE ROAD – NEW SEWAGE PUMPSTATION

HOLE No: AH3
Sheet 1 of 1

JOB NUMBER: 32742

Scale 1:20

Depth (m)	Description
0.00	Dry to slightly moist, brown and orange brown, medium dense to very dense, slightly gravelly and slightly clayey, <u>SAND</u> containing root material and occasional foreign refuse material – (Fill)
0.70	Slightly moist, red, medium dense to occasionally loose in places, <u>clayey SAND</u> containing occasional lenses of sand – (Berea Fm derived fill)
2.20	Slightly moist to wet, reddish brown, medium dense, slightly clayey, <u>SAND</u> – (Fill)
2.60	Wet, brown, medium dense, <u>SAND</u> containing occasional clayey lenses – (Alluvium/Fill)
3.10	Wet, very dark grey, loose, <u>silty</u>, <u>clayey SAND</u> – (Alluvium/Estuarine)
3.30	

NOTES

1) No hand auger refusal encountered – hole stopped due to sidewall collapse and poor recovery in very wet alluvial material at 3.3m

2) Samples taken for laboratory testing

3) Slight ground water seepage encountered from approximately 2.7m depth

CONTRACTOR :
MACHINE : HAND AUGER
DRILLED BY : NA
PROFILED BY : A.J
TYPE SET BY : AJ
SETUP FILE : DMPSP.SET

INCLINATION :
DIAM : NA
DATE : NA
DATE : 14-15/08/2019
DATE : 02/09/19 15:32
TEXT : ..C:\DOTIN\SPMASTER.DOC

ELEVATION :
X-COORD :
Y-COORD :

HOLE No: AH3

D06B DRENNAN MAUD & PARTNERS

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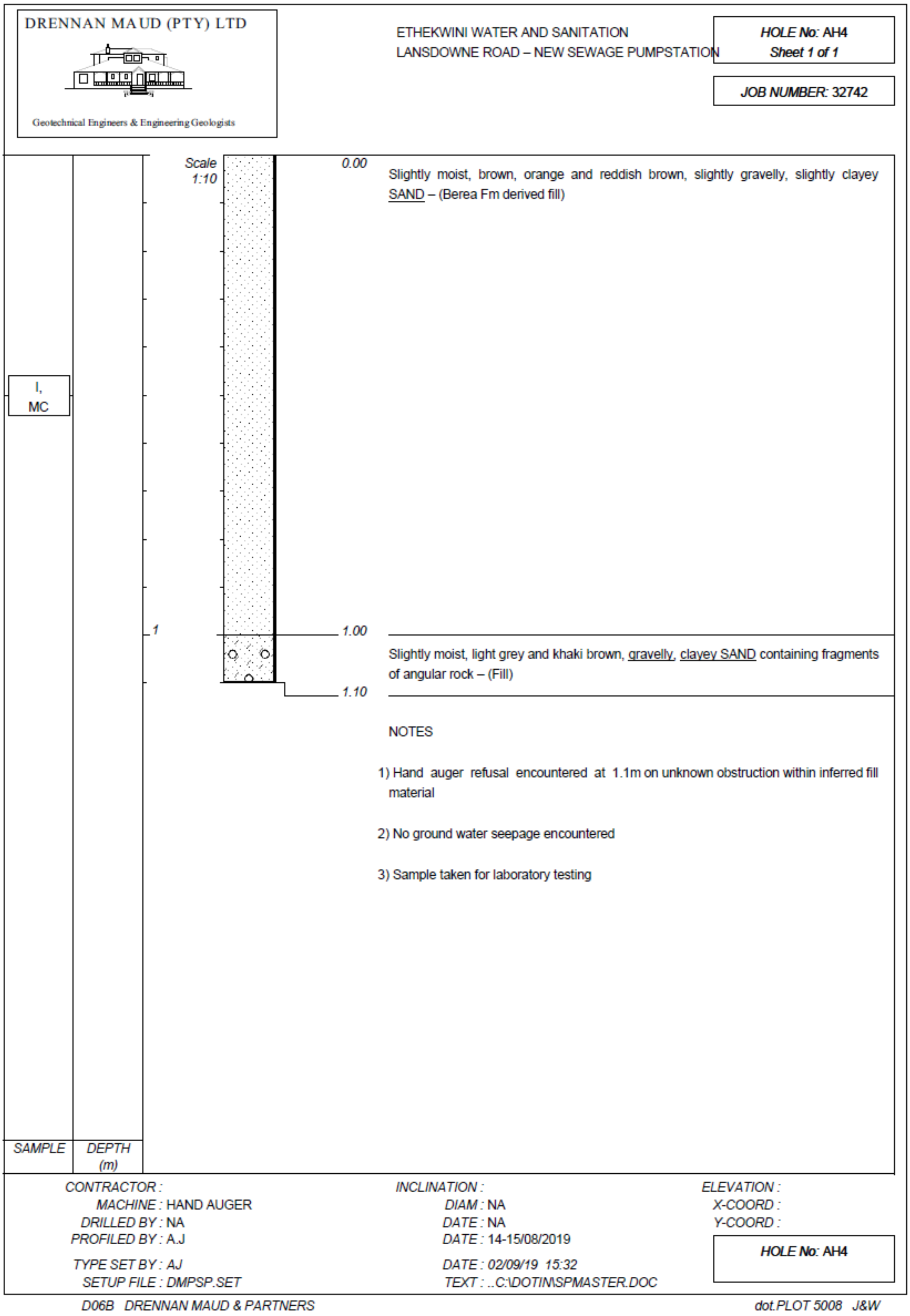

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ETHEKWINI WATER AND SANITATION
LANSDOWNE ROAD – NEW SEWAGE PUMPSTATION

HOLE No: **AH4**
Sheet 1 of 1

JOB NUMBER: **32742**

Scale 1:10

SAMPLE	DEPTH (m)		
		0.00	Slightly moist, brown, orange and reddish brown, slightly gravelly, slightly clayey SAND – (Berea Fm derived fill)
I, MC	1	1.00	Slightly moist, light grey and khaki brown, gravelly, clayey SAND containing fragments of angular rock – (Fill)
		1.10	

NOTES

1) Hand auger refusal encountered at 1.1m on unknown obstruction within inferred fill material

2) No ground water seepage encountered

3) Sample taken for laboratory testing

CONTRACTOR :
MACHINE : HAND AUGER
DRILLED BY : NA
PROFILED BY : A.J

TYPE SET BY : AJ
SETUP FILE : DMPSP.SET

INCLINATION :
DIAM : NA
DATE : NA
DATE : 14-15/08/2019

DATE : 02/09/19 15:32
TEXT : ..C:\DOTIN\SPMASTER.DOC

ELEVATION :
X-COORD :
Y-COORD :

HOLE No: **AH4**

D06B DRENNAN MAUD & PARTNERS

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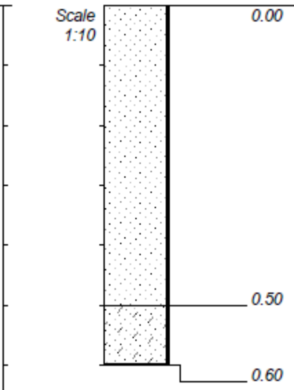
ETHEKWINI WATER AND SANITATION
LANSDOWNE ROAD – NEW SEWAGE PUMPSTATION

HOLE No: AH5
Sheet 1 of 1

JOB NUMBER: 32742

SAMPLE	DEPTH (m)
I, MC	



0.00	Slightly moist, brown, medium dense, slightly gravelly SAND containing occasional fragments of brick and rubble – (Fill)
0.50	Slightly moist, red brown, medium dense, slightly gravelly, clayey SAND containing occasional fragments of rubble – (Berea Fm derived fill)
0.60	

NOTES

1) Hand auger refusal encountered at 0.6m on unknown obstruction within inferred fill material

2) No ground water seepage encountered

3) Sample taken for laboratory testing

4) Nearby hand auger carried out for proving of services to 1.5m depth encountered yellow brown sand from approximately 0.5 – 1.5m depth.

CONTRACTOR :
MACHINE : HAND AUGER
DRILLED BY : NA
PROFILED BY : A.J
TYPE SET BY : AJ
SETUP FILE : DMPSP.SET

INCLINATION :
DIAM : NA
DATE : NA
DATE : 14-15/08/2019
DATE : 02/09/19 15:32
TEXT : ..C:\DOTIN\SPMASTER.DOC

ELEVATION :
X-COORD :
Y-COORD :

HOLE No: AH5

D06B DRENNAN MAUD & PARTNERS

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APPENDIX B
DCP Test Results
(DCP 1 - 9)

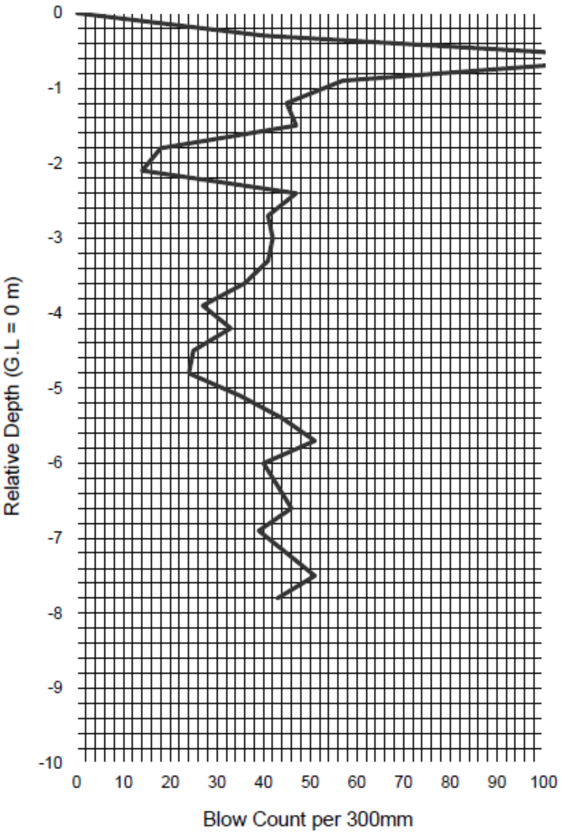
Dynamic Cone Penetrometer

Test No. : 1

Project :	**Lansdowne Road Sewer Pump Station Development**			
Client:	**Ethekwini Water and Sanitation**			
Date:	14-08-2019	Remarks:	-	
Test Location:	Quality Street - Jacobs		-	
Date of Test:	14-08-2019	Depth Interval (m) :		0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	40
-0.6	122
-0.9	57
-1.2	45
-1.5	47
-1.8	18
-2.1	14
-2.4	47
-2.7	41
-3.0	42
-3.3	41
-3.6	36
-3.9	27
-4.2	33
-4.5	25
-4.8	24
-5.1	35
-5.4	44
-5.7	51
-6.0	40
-6.3	43
-6.6	46
-6.9	39
-7.2	45
-7.5	51
-7.8	43
-	
-	
-	
-	
-	
-	
-	

Blow Count vs Depth



Reference No. : 32748

Drennan Maud (Pty) Ltd.

Fig. No. -

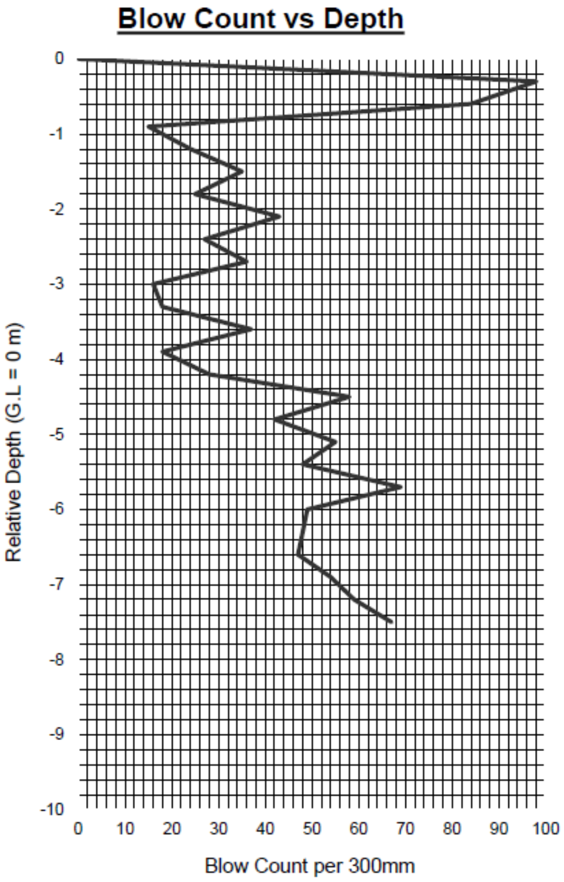
Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 2

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	98
-0.6	84
-0.9	15
-1.2	24
-1.5	35
-1.8	25
-2.1	43
-2.4	27
-2.7	36
-3.0	16
-3.3	18
-3.6	37
-3.9	18
-4.2	28
-4.5	58
-4.8	42
-5.1	55
-5.4	48
-5.7	69
-6.0	49
-6.3	48
-6.6	47
-6.9	54
-7.2	59
-7.5	67
-	
-	
-	
-	
-	
-	
-	
-	



Reference No. : **32748**

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 3

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	80
-0.6	122
-0.9	46
-1.2	37
-1.5	26
-1.8	10
-2.1	16
-2.4	10
-2.7	11
-3.0	8
-3.3	13
-3.6	12
-3.9	10
-4.2	16
-4.5	20
-4.8	32
-5.1	50
-5.4	34
-5.7	57
-6.0	35
-6.3	33
-6.6	43
-6.9	45
-7.2	54
-7.5	105
-	
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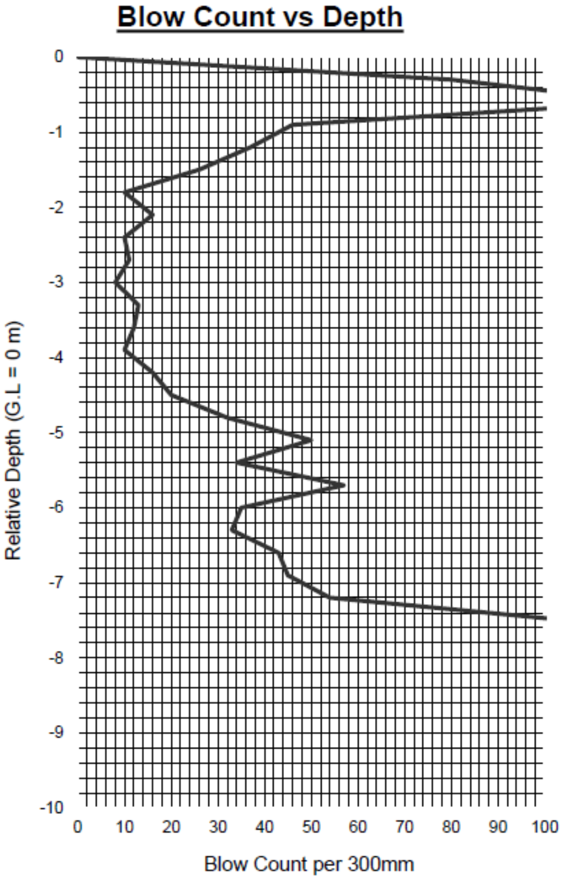
-	
-	
-	
-	
-	


Blow Count vs Depth

Reference No. : **32748**

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 4A

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	Count Blows/0.3m
0	0
-0.3	17
-0.6	42
-0.9	35
-1.2	38
-1.5	90
-1.8	100
-	
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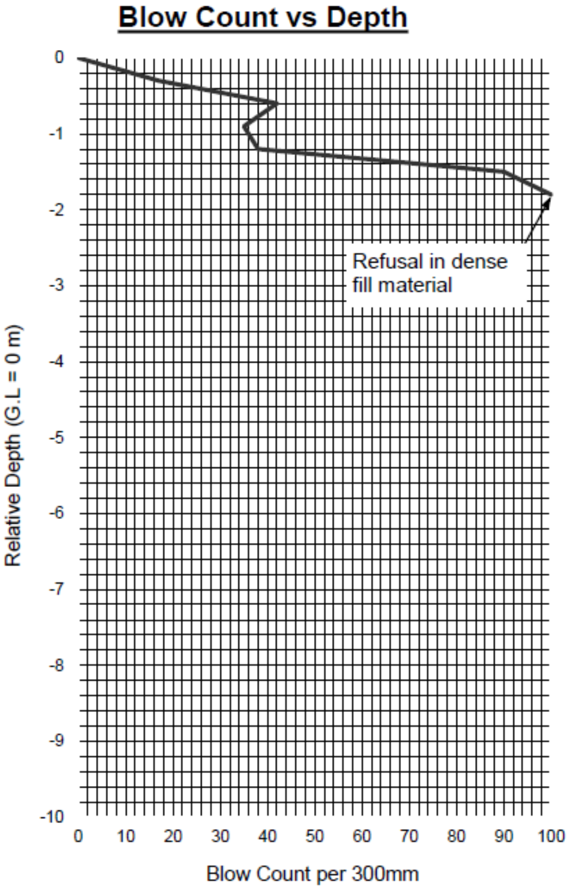

Reference No. : **32748** **Drennan Maud (Pty) Ltd.**

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 4B

Project :	**Lansdowne Road Sewer Pump Station Development**		
Client:	**Ethekwini Water and Sanitation**		
Date:	14-08-2019	Remarks:	-
Test Location:	Quality Street - Jacobs		-
Date of Test:	14-08-2019	Depth Interval (m) :	0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	23
-0.6	109
-0.9	60
-	
-	
-	
-	
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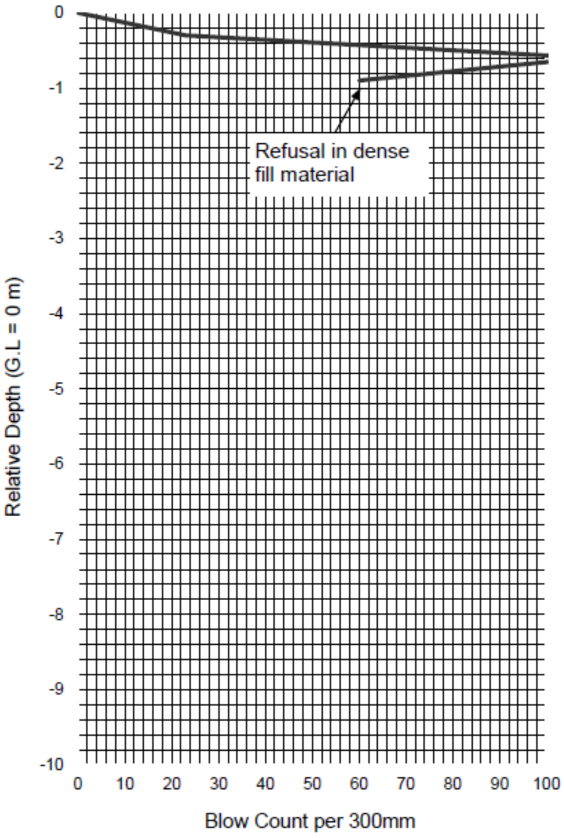
Blow Count vs Depth



Reference No. : 32748

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 5

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	Count Blows/0.3m
0	0
-0.3	30
-0.6	159
-0.9	67
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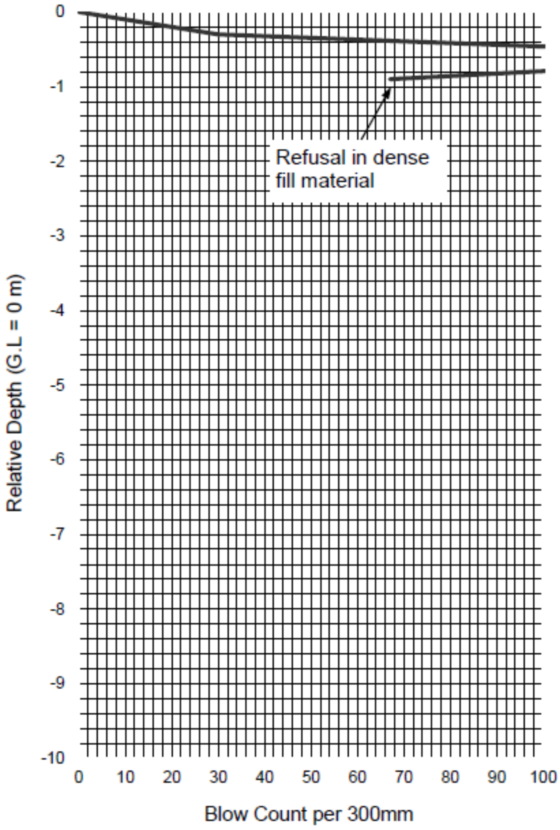
Blow Count vs Depth



Reference No. : **32748**

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 6A

Project :	**Lansdowne Road Sewer Pump Station Development**		
Client:	**Ethekwini Water and Sanitation**		
Date:	14-08-2019	Remarks:	-
Test Location:	Quality Street - Jacobs		-
Date of Test:	14-08-2019	Depth Interval (m) :	0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	27
-0.6	40
-0.9	15
-1.2	5
-1.5	4
-1.8	5
-2.1	34
-2.4	35
-	
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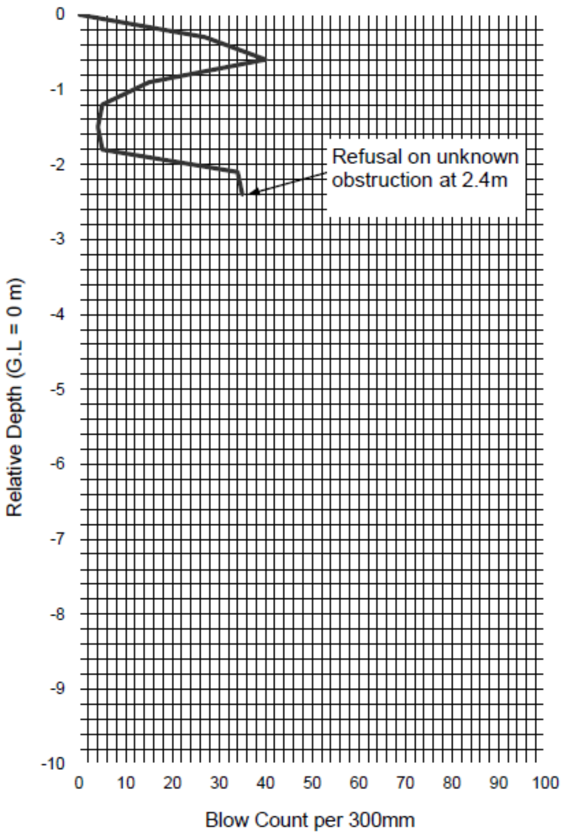
Blow Count vs Depth



Refusal on unknown obstruction at 2.4m

Reference No. : 32748

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 6B

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	Count Blows/0.3m
0	0
-0.3	30
-0.6	63
-0.9	37
-1.2	20
-1.5	14
-1.8	12
-2.1	36
-2.4	26
-2.7	33
-3.0	11
-3.3	20
-3.6	13
-3.9	4
-4.2	5
-	
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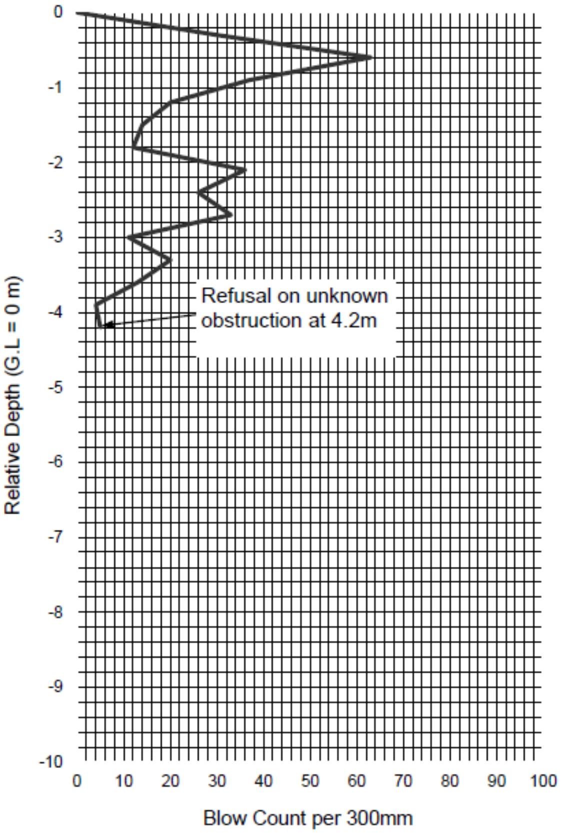
Blow Count vs Depth



Refusal on unknown obstruction at 4.2m

Reference No. : **32748**

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 7

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	
-0.6	105
-0.9	55
-1.2	56
-1.5	65
-1.8	38
-2.1	62
-2.4	115
-2.7	81
-3.0	130
-3.3	110
-3.6	50
-3.9	24
-4.2	72
-4.5	117
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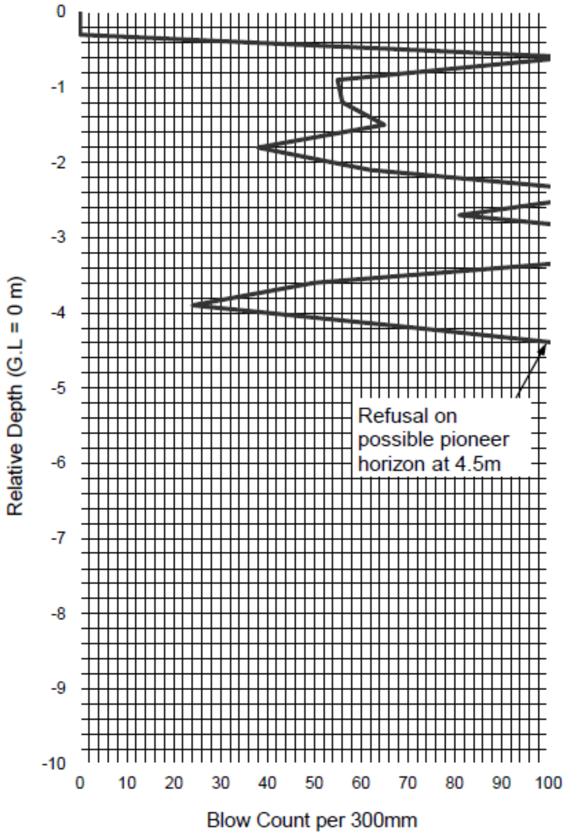
Blow Count vs Depth



Reference No. : **32748**

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 8

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	
-0.6	72
-	
-	
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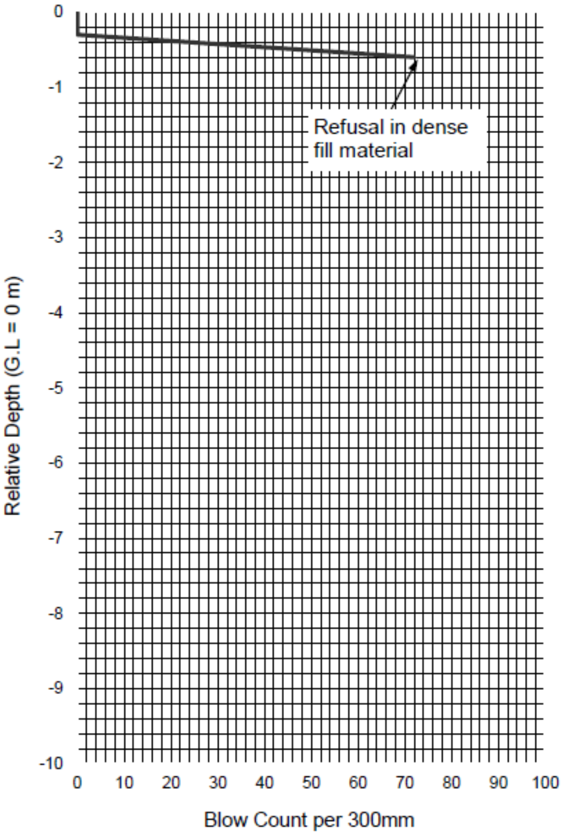
Blow Count vs Depth



Refusal in dense fill material

Reference No. : **32748**

Drennan Maud (Pty) Ltd.

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

Dynamic Cone Penetrometer

Test No. : 9

Project : **Lansdowne Road Sewer Pump Station Development**
Client: **Ethekwini Water and Sanitation**
Date: 14-08-2019 Remarks: -
Test Location: Quality Street - Jacobs -
Date of Test: 14-08-2019 Depth Interval (m) : 0.3

Depth (m)	**Count** Blows/0.3m
0	0
-0.3	73
-0.6	37
-0.9	40
-1.2	54
-1.5	24
-1.8	12
-2.1	18
-2.4	18
-2.7	19
-3.0	11
-3.3	12
-3.6	11
-3.9	8
-4.2	14
-4.5	20
-4.8	32
-5.1	35
-5.4	26
-5.7	33
-6.0	32
-6.3	34
-6.6	31
-6.9	33
-7.2	39
-7.5	60
-7.8	32
-	
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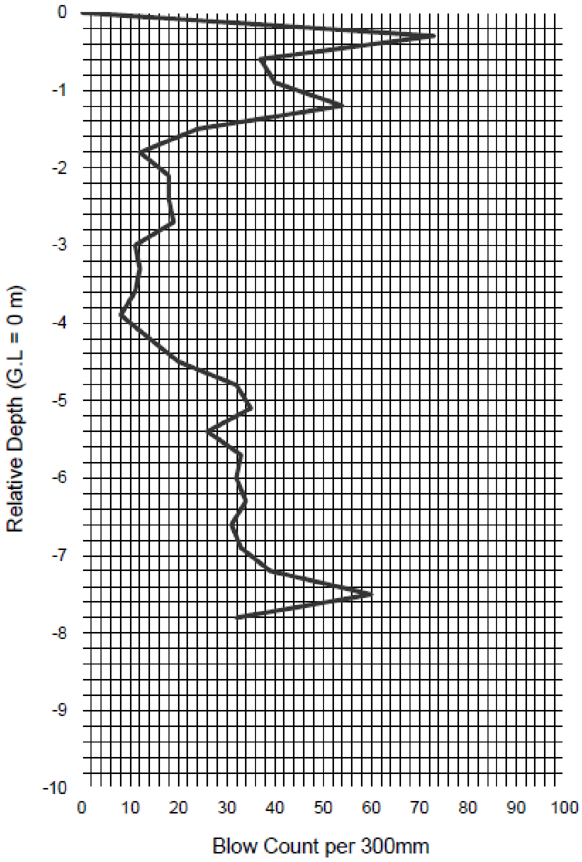
-	

Blow Count vs Depth



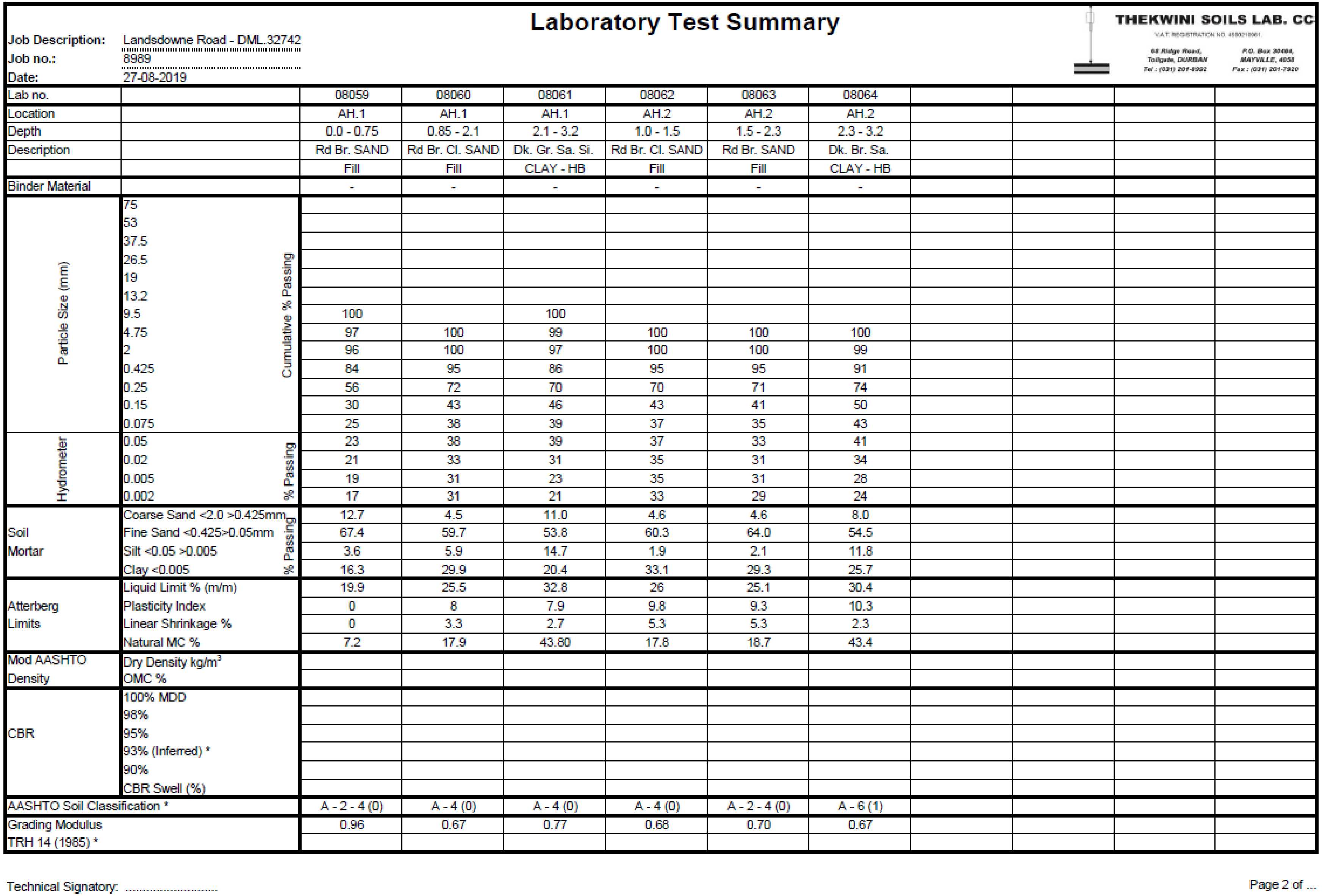
Reference No. : 32748 **Drennan Maud & Partners.**

Fig. No. -

Note: DCP Blow Count equals the number of blows of a 10kg hammer dropping 450mm required to drive a 25mm diameter 60° cone a distance of 300mm.

APPENDIX C

Laboratory Test Results Summary



Laboratory Test Summary

THEKWINI SOILS LAB. CC
V.A.T. REGISTRATION NO. 4590218061
68 Ridge Road, Tollgate, DURBAN Tel : (031) 201-8992
P.O. Box 30484, MAYVILLE, 4058 Fax : (031) 201-7920

Job Description: Landsdowne Road - DML.32742
Job no.: 8989
Date: 27-08-2019

Lab no.			08059	08060	08061	08062	08063	08064				
Location			AH.1	AH.1	AH.1	AH.2	AH.2	AH.2				
Depth			0.0 - 0.75	0.85 - 2.1	2.1 - 3.2	1.0 - 1.5	1.5 - 2.3	2.3 - 3.2				
Description			Rd Br. SAND	Rd Br. Cl. SAND	Dk. Gr. Sa. Si.	Rd Br. Cl. SAND	Rd Br. SAND	Dk. Br. Sa.				
			Fill	Fill	CLAY - HB	Fill	Fill	CLAY - HB				
Binder Material			-	-	-	-	-	-				
Particle Size (mm)	75	Cumulative % Passing										
	53											
	37.5											
	26.5											
	19											
	13.2											
	9.5		100		100							
	4.75		97	100	99	100	100	100				
	2		96	100	97	100	100	99				
	0.425		84	95	86	95	95	91				
	0.25		56	72	70	70	71	74				
	0.15		30	43	46	43	41	50				
	0.075		25	38	39	37	35	43				
Hydrometer	0.05	% Passing	23	38	39	37	33	41				
	0.02		21	33	31	35	31	34				
	0.005		19	31	23	35	31	28				
	0.002		17	31	21	33	29	24				
Soil Mortar	Coarse Sand <2.0 >0.425mm	% Passing	12.7	4.5	11.0	4.6	4.6	8.0				
	Fine Sand <0.425>0.05mm		67.4	59.7	53.8	60.3	64.0	54.5				
	Silt <0.05 >0.005		3.6	5.9	14.7	1.9	2.1	11.8				
	Clay <0.005		16.3	29.9	20.4	33.1	29.3	25.7				
Atterberg Limits	Liquid Limit % (m/m)		19.9	25.5	32.8	26	25.1	30.4				
	Plasticity Index		0	8	7.9	9.8	9.3	10.3				
	Linear Shrinkage %		0	3.3	2.7	5.3	5.3	2.3				
	Natural MC %		7.2	17.9	43.80	17.8	18.7	43.4				
Mod AASHTO Density	Dry Density kg/m^3											
	OMC %											
CBR	100% MDD											
	98%											
	95%											
	93% (Inferred) *											
	90%											
	CBR Swell (%)											
AASHTO Soil Classification *			A - 2 - 4 (0)	A - 4 (0)	A - 4 (0)	A - 4 (0)	A - 2 - 4 (0)	A - 6 (1)				
Grading Modulus TRH 14 (1985) *			0.96	0.67	0.77	0.68	0.70	0.67				

Technical Signatory: ....

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TEST REPORT

MATERIALS ANALYSIS

THEKWINI SOILS LAB. CC
V.A.T. REGISTRATION NO. 4590218061.
68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992
P.O. Box 30464,
MAYVILLE, 4058
Fax : (031) 201-7920

Project: Landsdowne Road - DML.32742

Ref no.: 8989 **Lab no.:** 08059 **Borehole/Pit no.:** AH.1
Description: Rd Br. SAND
Fill
Depth: 0.0 - 0.75

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	100.0
4.75	97.4
2	95.7
0.425	83.6
0.25	56.1
0.15	30.3
0.075	24.8
0.05	22.7
0.02	20.7
0.005	18.6
0.002	16.6

M.I.T SIZE CLASSIFICATION *

Cobble%		0.0
Gravel%	4.3	
Coarse		0.0
Medium		1.9
Fine		2.3
Sand%	72.1	
Coarse		10.8
Medium		41.7
Fine		19.7
Silt%	7.0	
Coarse		2.9
Medium		1.9
Fine		2.2
Clay%	16.6	

PLASTICITY

Liquid Limit, %	19.9
Plasticity Index	0
Linear Shrinkage, % (L/L)	0

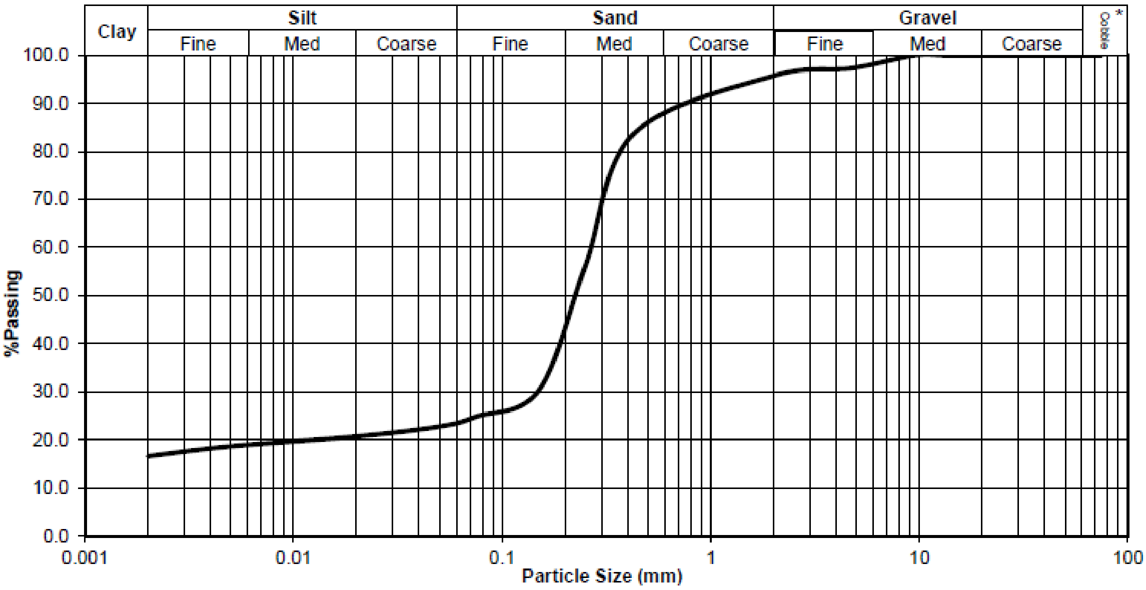
GRADING

D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.96

CLASSIFICATION *

Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 2 - 4
Unified Classification	SM

Grading Curve



Ref no.: 8989 Fig no.: -

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THEKWINI SOILS LAB. CC
V.A.T. REGISTRATION NO. [illegible]
68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992
P.O. Box 30464,
MAYVILLE, 4058
Fax : (031) 201-7920

Project: Landsdowne Road - DML.32742

Ref no.: 8989 **Lab no.:** 08060 **Borehole/Pit no.:** AH.1
Description: Rd Br. Cl. SAND
Fill
Depth: 0.85 - 2.1

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

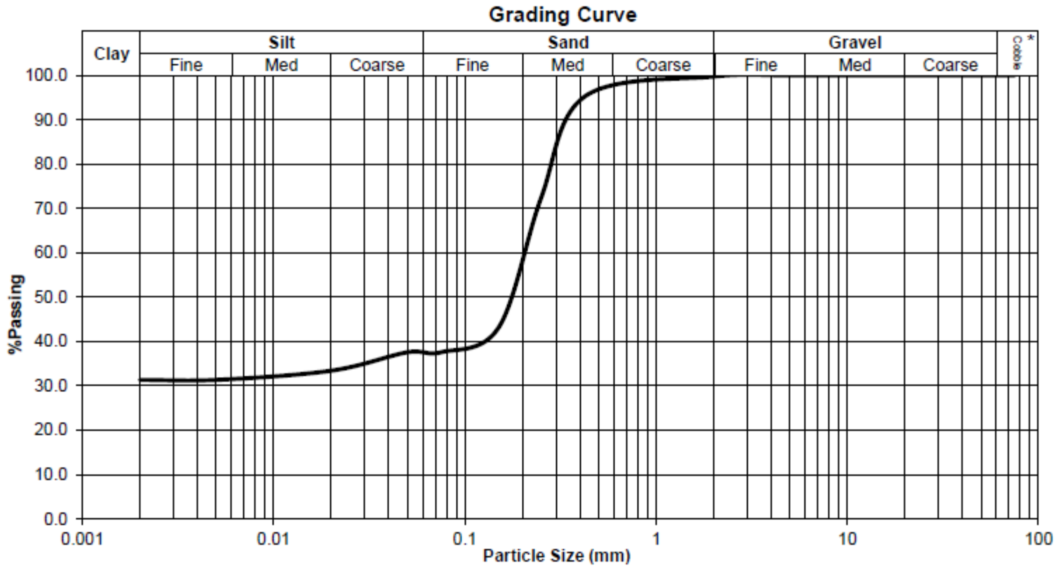
Grading Analysis	
Grain Size (mm)	%Passing
75	100.0
53	100.0
37.5	100.0
26.5	100.0
19	100.0
13.2	100.0
9.5	100.0
4.75	100.0
2	99.7
0.425	95.3
0.25	72.3
0.15	43.1
0.075	37.6
0.05	37.6
0.02	33.4
0.005	31.3
0.002	31.3

M.I.T SIZE CLASSIFICATION *		
Cobble%		0.0
Gravel%	0.3	
Coarse		0.0
Medium		0.0
Fine		0.3
Sand%	62.2	
Coarse		4.0
Medium		38.1
Fine		20.2
Silt%	6.2	
Coarse		4.2
Medium		1.9
Fine		0.1
Clay%	31.3	

PLASTICITY	
Liquid Limit	25.5
Plasticity Index	8
Linear Shrinkage	3.3

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.67

CLASSIFICATION *	
Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 4
Unified Classification	SC



Grading Curve

Ref no.: 8989 Fig no.: -

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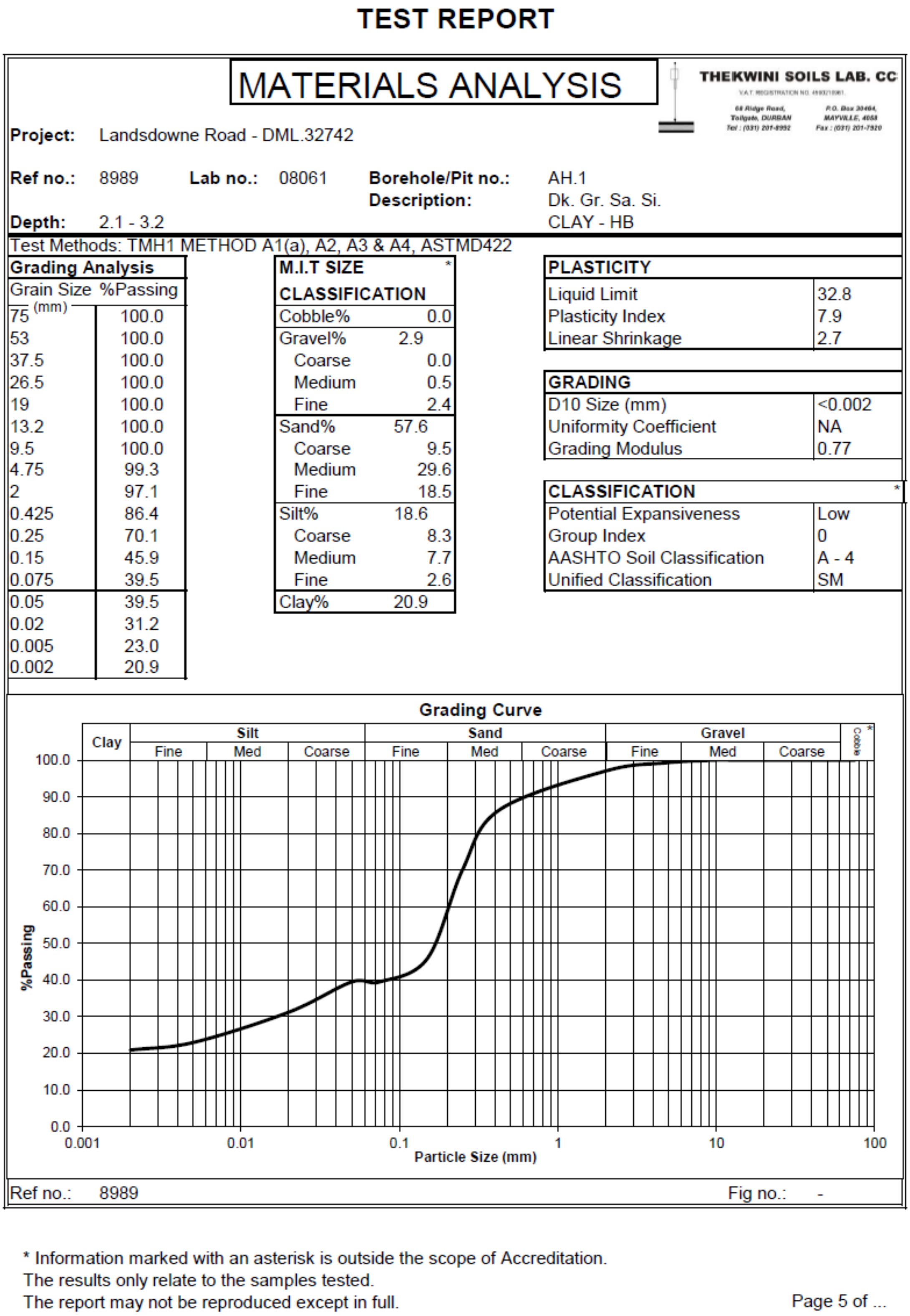

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THEKWINI SOILS LAB. CC

68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992

P.O. Box 30464,
MAYVILLE, 4058
Fax : (031) 201-7920

Project: Landsdowne Road - DML.32742

Ref no.: 8989 **Lab no.:** 08061 **Borehole/Pit no.:** AH.1

Description: Dk. Gr. Sa. Si. CLAY - HB

Depth: 2.1 - 3.2

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	100.0
4.75	99.3
2	97.1
0.425	86.4
0.25	70.1
0.15	45.9
0.075	39.5
0.05	39.5
0.02	31.2
0.005	23.0
0.002	20.9

M.I.T SIZE CLASSIFICATION *

Cobble%		0.0
Gravel%	2.9	
Coarse		0.0
Medium		0.5
Fine		2.4
Sand%	57.6	
Coarse		9.5
Medium		29.6
Fine		18.5
Silt%	18.6	
Coarse		8.3
Medium		7.7
Fine		2.6
Clay%	20.9	

PLASTICITY

Liquid Limit	32.8
Plasticity Index	7.9
Linear Shrinkage	2.7

GRADING

D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.77

CLASSIFICATION *

Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 4
Unified Classification	SM

Grading Curve

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68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992
P.O. Box 30464,
MAYVILLE, 4058
Fax : (031) 201-7920

Project: Landsdowne Road - DML.32742

Ref no.: 8989 **Lab no.:** 08062 **Borehole/Pit no.:** AH.2
Description: Rd Br. Cl. SAND
Fill
Depth: 1.0 - 1.5

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

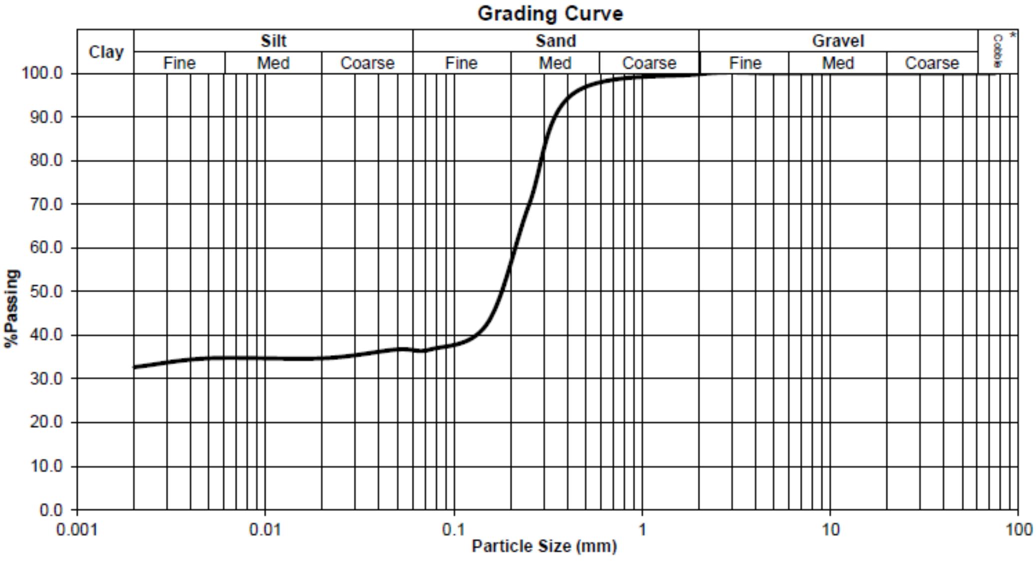
Grading Analysis	
Grain Size (mm)	%Passing
75	100.0
53	100.0
37.5	100.0
26.5	100.0
19	100.0
13.2	100.0
9.5	100.0
4.75	100.0
2	99.8
0.425	95.2
0.25	69.8
0.15	42.7
0.075	36.7
0.05	36.7
0.02	34.7
0.005	34.7
0.002	32.7

M.I.T SIZE CLASSIFICATION *		
Cobble%		0.0
Gravel%	0.2	
Coarse		0.0
Medium		0.0
Fine		0.2
Sand%	63.0	
Coarse		4.1
Medium		39.5
Fine		19.5
Silt%	4.1	
Coarse		2.0
Medium		0.0
Fine		2.0
Clay%	32.7	

PLASTICITY	
Liquid Limit	26
Plasticity Index	9.8
Linear Shrinkage	5.3

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.68

CLASSIFICATION *	
Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 4
Unified Classification	SC



Ref no.: 8989 Fig no.: -

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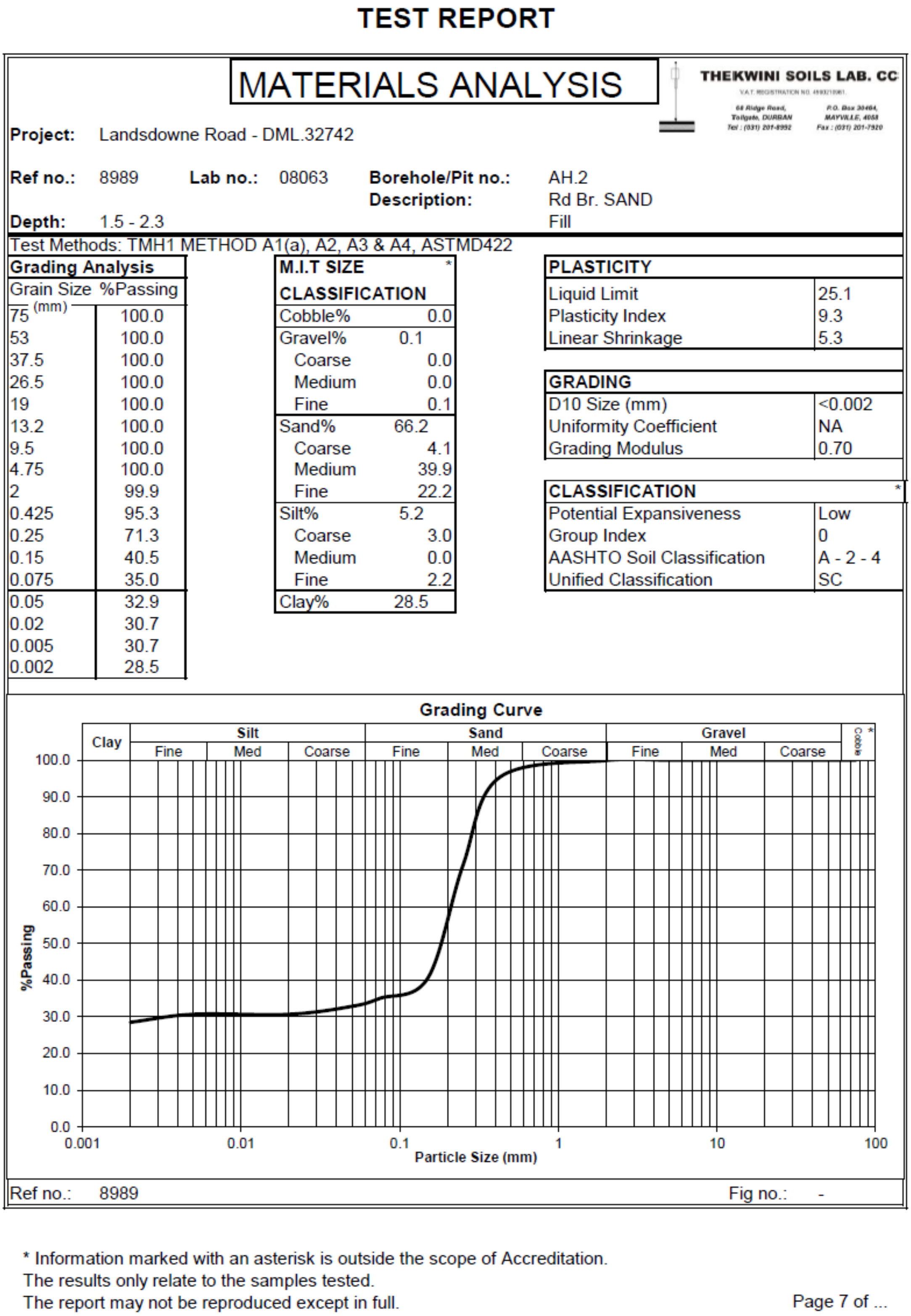

TEST REPORT

MATERIALS ANALYSIS

THEKWINI SOILS LAB. CC
V.A.T. REGISTRATION NO. 4890218961.
68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992
P.O. Box 30464,
MAYVILLE, 4058
Fax : (031) 201-7920

Project: Landsdowne Road - DML.32742

Ref no.: 8989 **Lab no.:** 08063 **Borehole/Pit no.:** AH.2
Description: Rd Br. SAND
Fill
Depth: 1.5 - 2.3

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	100.0
4.75	100.0
2	99.9
0.425	95.3
0.25	71.3
0.15	40.5
0.075	35.0
0.05	32.9
0.02	30.7
0.005	30.7
0.002	28.5

M.I.T SIZE CLASSIFICATION *		
Cobble%		0.0
Gravel%	0.1	
Coarse		0.0
Medium		0.0
Fine		0.1
Sand%	66.2	
Coarse		4.1
Medium		39.9
Fine		22.2
Silt%	5.2	
Coarse		3.0
Medium		0.0
Fine		2.2
Clay%	28.5	

PLASTICITY	
Liquid Limit	25.1
Plasticity Index	9.3
Linear Shrinkage	5.3

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.70

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

Grading Curve

Ref no.: 8989 Fig no.: -

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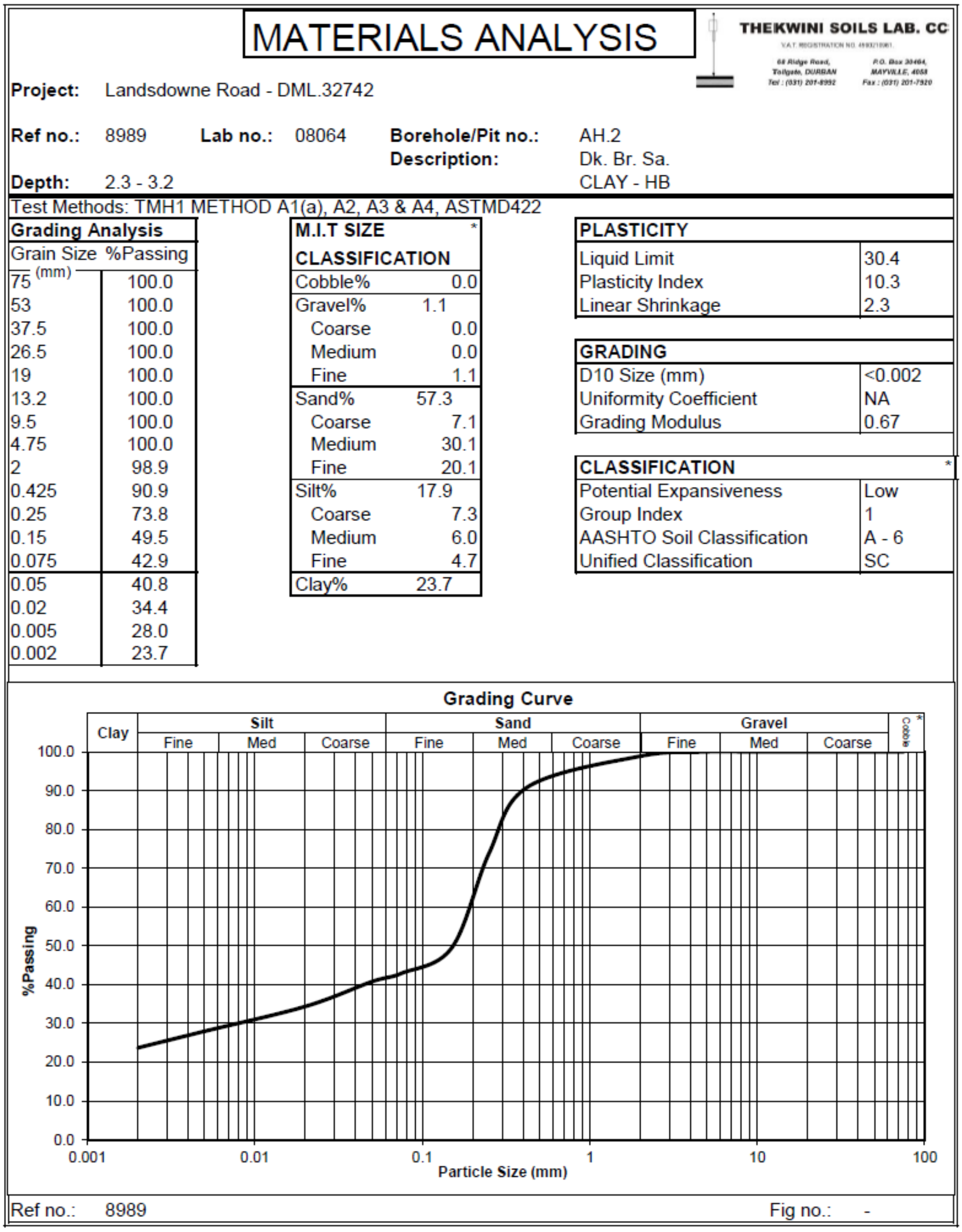
TEST REPORT



MATERIALS ANALYSIS

THEKWINI SOILS LAB. CC
V.A.T. REGISTRATION NO. [illegible]
68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992
P.O. Box 30464,
MAYVILLE, 4058
Fax : (031) 201-7920

Project: Landsdowne Road - DML.32742

Ref no.: 8989 **Lab no.:** 08064 **Borehole/Pit no.:** AH.2
Description: Dk. Br. Sa. CLAY - HB
Depth: 2.3 - 3.2

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	100.0
4.75	100.0
2	98.9
0.425	90.9
0.25	73.8
0.15	49.5
0.075	42.9
0.05	40.8
0.02	34.4
0.005	28.0
0.002	23.7

M.I.T SIZE *
CLASSIFICATION

Cobble%		0.0
Gravel%	1.1	
Coarse		0.0
Medium		0.0
Fine		1.1
Sand%	57.3	
Coarse		7.1
Medium		30.1
Fine		20.1
Silt%	17.9	
Coarse		7.3
Medium		6.0
Fine		4.7
Clay%	23.7	

PLASTICITY	
Liquid Limit	30.4
Plasticity Index	10.3
Linear Shrinkage	2.3

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.67

CLASSIFICATION *	
Potential Expansiveness	Low
Group Index	1
AASHTO Soil Classification	A - 6
Unified Classification	SC

Grading Curve

Ref no.: 8989 Fig no.: -

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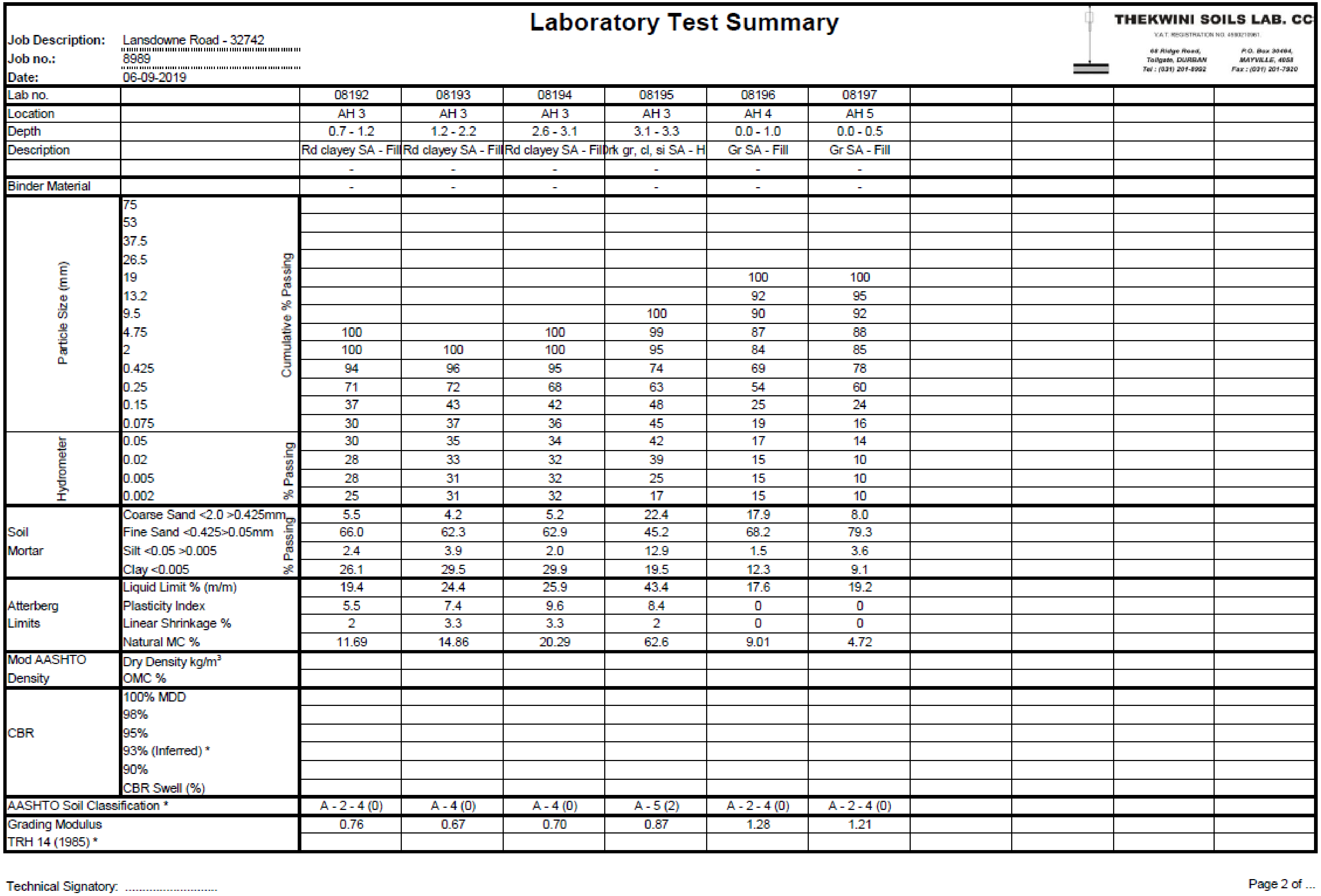


Laboratory Test Summary

THEKWINI SOILS LAB. CC

68 Ridge Road,
Tollgate, DURBAN
Tel : (031) 201-8992

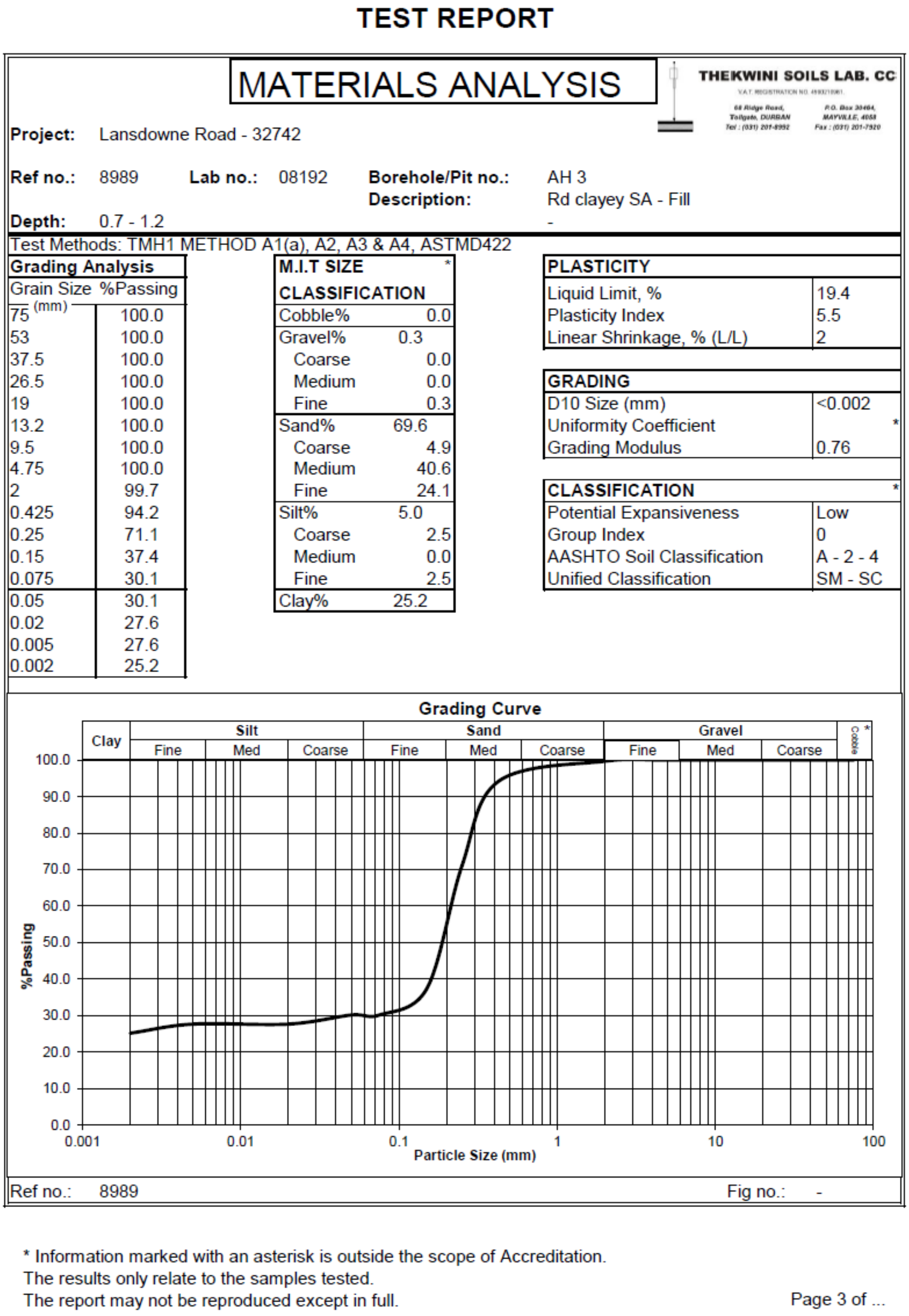
P.O. Box 30494,
MAYVILLE, 4058
Fax : (031) 201-7920

Job Description: Lansdowne Road - 32742
Job no.: 8989
Date: 06-09-2019

Lab no.			08192	08193	08194	08195	08196	08197				
Location			AH 3	AH 3	AH 3	AH 3	AH 4	AH 5				
Depth			0.7 - 1.2	1.2 - 2.2	2.6 - 3.1	3.1 - 3.3	0.0 - 1.0	0.0 - 0.5				
Description			Rd clayey SA - Fill	Rd clayey SA - Fill	Rd clayey SA - Fill	rk gr, cl, si SA - H	Gr SA - Fill	Gr SA - Fill				
			-	-	-	-	-	-				
Binder Material			-	-	-	-	-	-				
Particle Size (mm)	75	Cumulative % Passing										
	53											
	37.5											
	26.5											
	19						100	100				
	13.2						92	95				
	9.5					100	90	92				
	4.75		100		100	99	87	88				
	2		100	100	100	95	84	85				
	0.425		94	96	95	74	69	78				
	0.25		71	72	68	63	54	60				
	0.15		37	43	42	48	25	24				
	0.075		30	37	36	45	19	16				
Hydrometer	0.05	% Passing	30	35	34	42	17	14				
	0.02		28	33	32	39	15	10				
	0.005		28	31	32	25	15	10				
	0.002		25	31	32	17	15	10				
Soil Mortar	Coarse Sand <2.0 >0.425mm	% Passing	5.5	4.2	5.2	22.4	17.9	8.0				
	Fine Sand <0.425>0.05mm		66.0	62.3	62.9	45.2	68.2	79.3				
	Silt <0.05 >0.005		2.4	3.9	2.0	12.9	1.5	3.6				
	Clay <0.005		26.1	29.5	29.9	19.5	12.3	9.1				
Atterberg Limits	Liquid Limit % (m/m)		19.4	24.4	25.9	43.4	17.6	19.2				
	Plasticity Index		5.5	7.4	9.6	8.4	0	0				
	Linear Shrinkage %		2	3.3	3.3	2	0	0				
	Natural MC %		11.69	14.86	20.29	62.6	9.01	4.72				
Mod AASHTO Density	Dry Density kg/m^3											
	OMC %											
CBR	100% MDD											
	98%											
	95%											
	93% (Inferred) *											
	90%											
	CBR Swell (%)											
AASHTO Soil Classification *			A - 2 - 4 (0)	A - 4 (0)	A - 4 (0)	A - 5 (2)	A - 2 - 4 (0)	A - 2 - 4 (0)				
Grading Modulus TRH 14 (1985) *			0.76	0.67	0.70	0.87	1.28	1.21				

Technical Signatory: ....

Page 2 of ...



TEST REPORT

MATERIALS ANALYSIS

THEKWINI SOILS LAB. CC

Project: Lansdowne Road - 32742

Ref no.: 8989 **Lab no.:** 08192 **Borehole/Pit no.:** AH 3

Description: Rd clayey SA - Fill

Depth: 0.7 - 1.2 -

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	100.0
4.75	100.0
2	99.7
0.425	94.2
0.25	71.1
0.15	37.4
0.075	30.1
0.05	30.1
0.02	27.6
0.005	27.6
0.002	25.2

M.I.T SIZE CLASSIFICATION *		
Cobble%		0.0
Gravel%	0.3	
Coarse		0.0
Medium		0.0
Fine		0.3
Sand%	69.6	
Coarse		4.9
Medium		40.6
Fine		24.1
Silt%	5.0	
Coarse		2.5
Medium		0.0
Fine		2.5
Clay%	25.2	

PLASTICITY	
Liquid Limit, %	19.4
Plasticity Index	5.5
Linear Shrinkage, % (L/L)	2

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	*
Grading Modulus	0.76

CLASSIFICATION *	
Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 2 - 4
Unified Classification	SM - SC

Grading Curve

Ref no.: 8989 Fig no.: -

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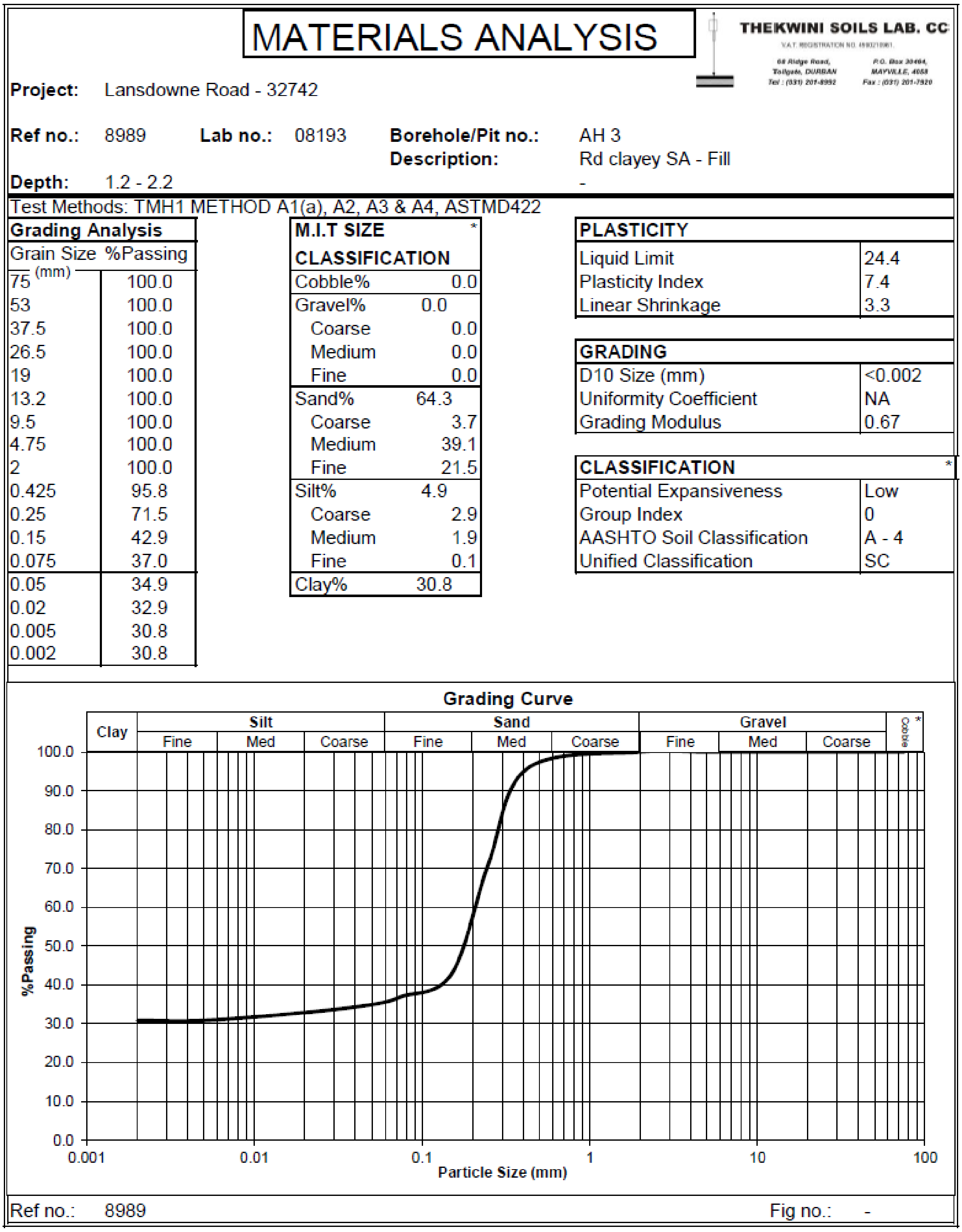
TEST REPORT



MATERIALS ANALYSIS

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Project: Lansdowne Road - 32742

Ref no.: 8989 **Lab no.:** 08193 **Borehole/Pit no.:** AH 3
Description: Rd clayey SA - Fill
-
Depth: 1.2 - 2.2

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	100.0
4.75	100.0
2	100.0
0.425	95.8
0.25	71.5
0.15	42.9
0.075	37.0
0.05	34.9
0.02	32.9
0.005	30.8
0.002	30.8

M.I.T SIZE CLASSIFICATION *

Cobble%		0.0
Gravel%	0.0	
Coarse		0.0
Medium		0.0
Fine		0.0
Sand%	64.3	
Coarse		3.7
Medium		39.1
Fine		21.5
Silt%	4.9	
Coarse		2.9
Medium		1.9
Fine		0.1
Clay%	30.8	

PLASTICITY	
Liquid Limit	24.4
Plasticity Index	7.4
Linear Shrinkage	3.3

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.67

CLASSIFICATION *	
Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 4
Unified Classification	SC

Grading Curve

Ref no.: 8989 Fig no.: -

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TEST REPORT

MATERIALS ANALYSIS

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Fax : (031) 201-7920

Project: Lansdowne Road - 32742

Ref no.: 8989 **Lab no.:** 08194 **Borehole/Pit no.:** AH 3

Description: Rd clayey SA - Fill

Depth: 2.6 - 3.1 -

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	100.0
4.75	100.0
2	99.9
0.425	94.7
0.25	68.5
0.15	41.5
0.075	35.8
0.05	33.7
0.02	31.6
0.005	31.6
0.002	31.6

M.I.T SIZE CLASSIFICATION *

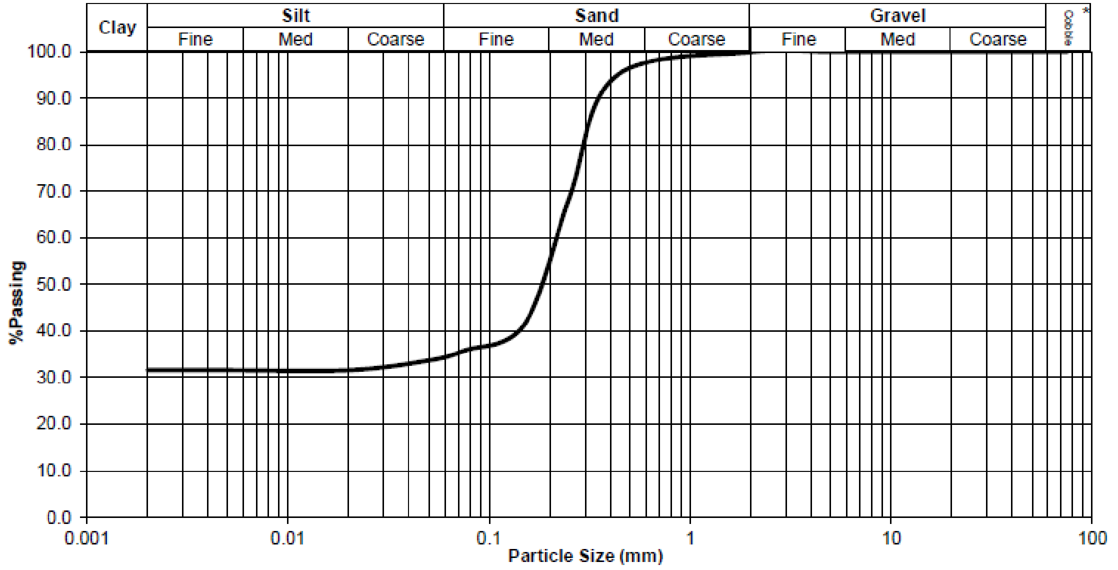
Cobble%		0.0
Gravel%	0.1	
Coarse		0.0
Medium		0.0
Fine		0.1
Sand%	65.4	
Coarse		4.7
Medium		40.3
Fine		20.5
Silt%	2.9	
Coarse		2.9
Medium		0.0
Fine		0.0
Clay%	31.6	

PLASTICITY	
Liquid Limit	25.9
Plasticity Index	9.6
Linear Shrinkage	3.3

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.70

CLASSIFICATION *	
Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 4
Unified Classification	SC

Grading Curve



Ref no.: 8989 Fig no.: -

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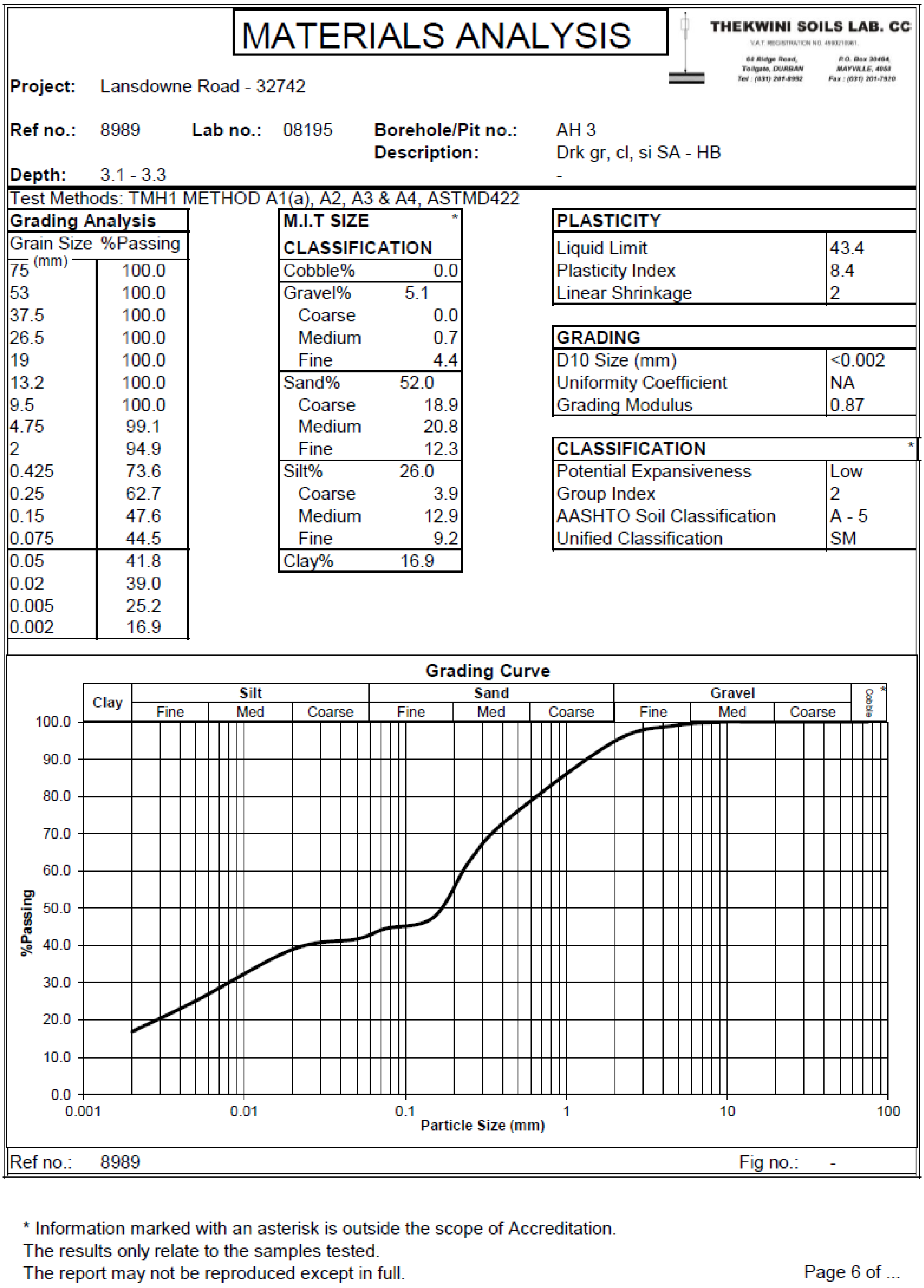
TEST REPORT



MATERIALS ANALYSIS

THEKWINI SOILS LAB. CC
V.A.T. REGISTRATION NO. [illegible]
64 Ridge Road, Tollgate, DURBAN Tel : (031) 201-8992
P.O. Box 30464, MAYVILLE, 4058 Fax : (031) 201-7920

Project: Lansdowne Road - 32742

Ref no.: 8989 **Lab no.:** 08195 **Borehole/Pit no.:** AH 3
Description: Drk gr, cl, si SA - HB
Depth: 3.1 - 3.3 -

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	100.0
4.75	99.1
2	94.9
0.425	73.6
0.25	62.7
0.15	47.6
0.075	44.5
0.05	41.8
0.02	39.0
0.005	25.2
0.002	16.9

M.I.T SIZE CLASSIFICATION *

Cobble%		0.0
Gravel%	5.1	
Coarse		0.0
Medium		0.7
Fine		4.4
Sand%	52.0	
Coarse		18.9
Medium		20.8
Fine		12.3
Silt%	26.0	
Coarse		3.9
Medium		12.9
Fine		9.2
Clay%	16.9	

PLASTICITY

Liquid Limit	43.4
Plasticity Index	8.4
Linear Shrinkage	2

GRADING

D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	0.87

CLASSIFICATION *

Potential Expansiveness	Low
Group Index	2
AASHTO Soil Classification	A - 5
Unified Classification	SM

Grading Curve

Ref no.: 8989 Fig no.: -

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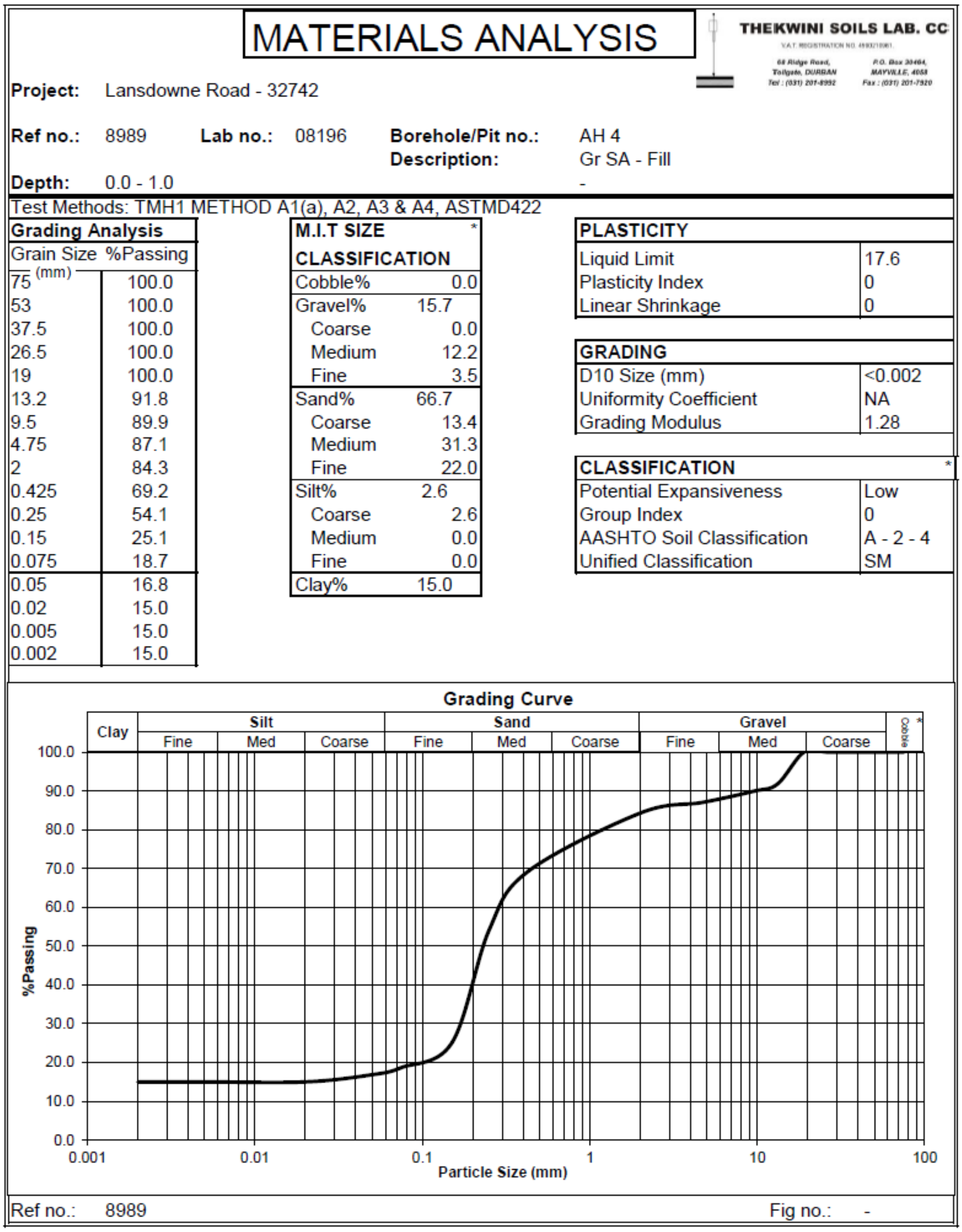
TEST REPORT



MATERIALS ANALYSIS

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Tel : (031) 201-8992
P.O. Box 30464, MAYVILLE, 4058
Fax : (031) 201-7920

Project: Lansdowne Road - 32742

Ref no.: 8989 **Lab no.:** 08196 **Borehole/Pit no.:** AH 4
Description: Gr SA - Fill
Depth: 0.0 - 1.0 -

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	91.8
9.5	89.9
4.75	87.1
2	84.3
0.425	69.2
0.25	54.1
0.15	25.1
0.075	18.7
0.05	16.8
0.02	15.0
0.005	15.0
0.002	15.0

M.I.T SIZE CLASSIFICATION *		
Cobble%		0.0
Gravel%	15.7	
Coarse		0.0
Medium		12.2
Fine		3.5
Sand%	66.7	
Coarse		13.4
Medium		31.3
Fine		22.0
Silt%	2.6	
Coarse		2.6
Medium		0.0
Fine		0.0
Clay%	15.0	

PLASTICITY	
Liquid Limit	17.6
Plasticity Index	0
Linear Shrinkage	0

GRADING	
D10 Size (mm)	<0.002
Uniformity Coefficient	NA
Grading Modulus	1.28

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

Grading Curve

Ref no.: 8989 Fig no.: -

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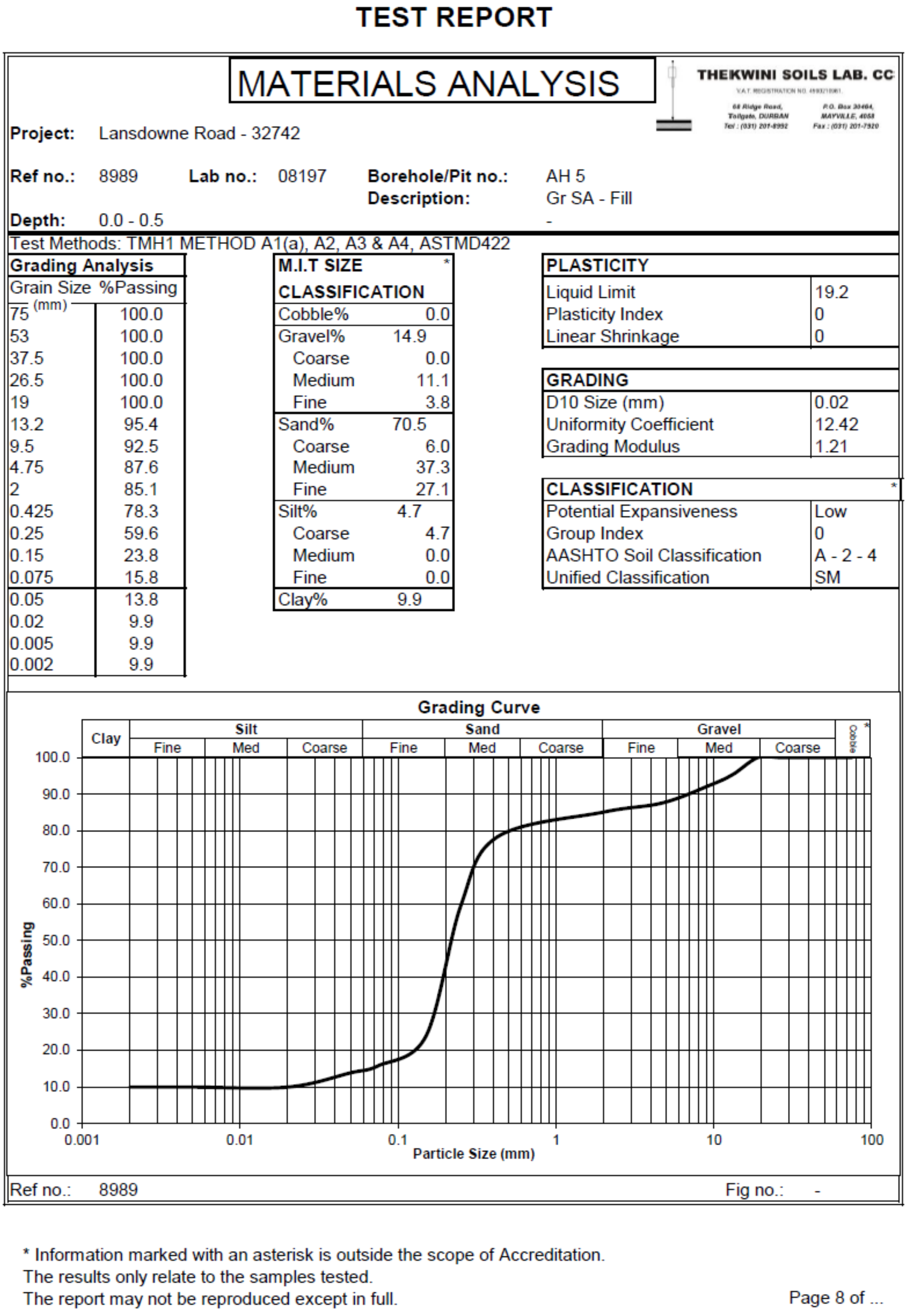

TEST REPORT

MATERIALS ANALYSIS

THEKWINI SOILS LAB. CC
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68 Ridge Road, Tollgate, DURBAN
Tel : (031) 201-8992
P.O. Box 30464, MAYVILLE, 4058
Fax : (031) 201-7920

Project: Lansdowne Road - 32742

Ref no.: 8989 **Lab no.:** 08197 **Borehole/Pit no.:** AH 5
Description: Gr SA - Fill
Depth: 0.0 - 0.5 -

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	95.4
9.5	92.5
4.75	87.6
2	85.1
0.425	78.3
0.25	59.6
0.15	23.8
0.075	15.8
0.05	13.8
0.02	9.9
0.005	9.9
0.002	9.9

M.I.T SIZE CLASSIFICATION *

Cobble%		0.0
Gravel%	14.9	
Coarse		0.0
Medium		11.1
Fine		3.8
Sand%	70.5	
Coarse		6.0
Medium		37.3
Fine		27.1
Silt%	4.7	
Coarse		4.7
Medium		0.0
Fine		0.0
Clay%	9.9	

PLASTICITY

Liquid Limit	19.2
Plasticity Index	0
Linear Shrinkage	0

GRADING

D10 Size (mm)	0.02
Uniformity Coefficient	12.42
Grading Modulus	1.21

CLASSIFICATION *

Potential Expansiveness	Low
Group Index	0
AASHTO Soil Classification	A - 2 - 4
Unified Classification	SM

Grading Curve

Ref no.: 8989 Fig no.: -

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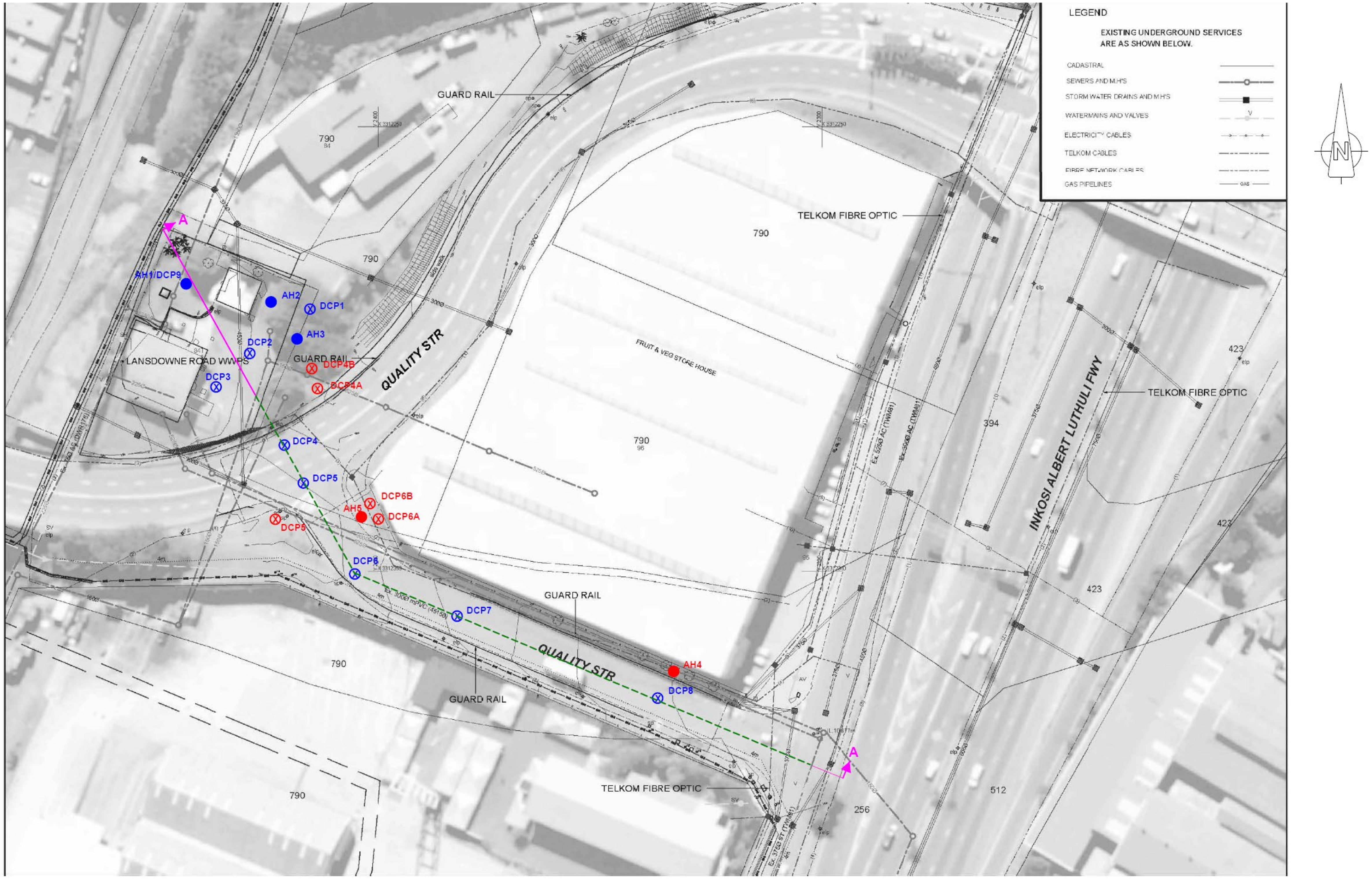
DRAWING 32742-01
Site Plan



KEY

DCP1 ⓧ ORIGINAL POSITION OF DYNAMIC CONE PENETROMETER TESTS PROPOSED BY ETHEKWINI ENGINEERS

AH1 ● ORIGINAL POSITION OF HAND AUGERS PROPOSED BY ETHEKWINI ENGINEERS

DCP6A ⓧ ORIGINAL ALTERNATE CONE PENETROMETER TESTS ON CONDUCTED ON ORIGINAL PROPOSED POSITIONS

AH4 ● ADDITIONAL HAND AUGERS POSITIONS

-------------- APPROX. PROPOSED ALIGNMENT OF NEW GRAVITY SEWER

A A INFERRED GEOLOGICAL CROSS SECTION

DRENNAN MAUD (PTY) LTD.
Geotechnical Engineers & Engineering Geologists

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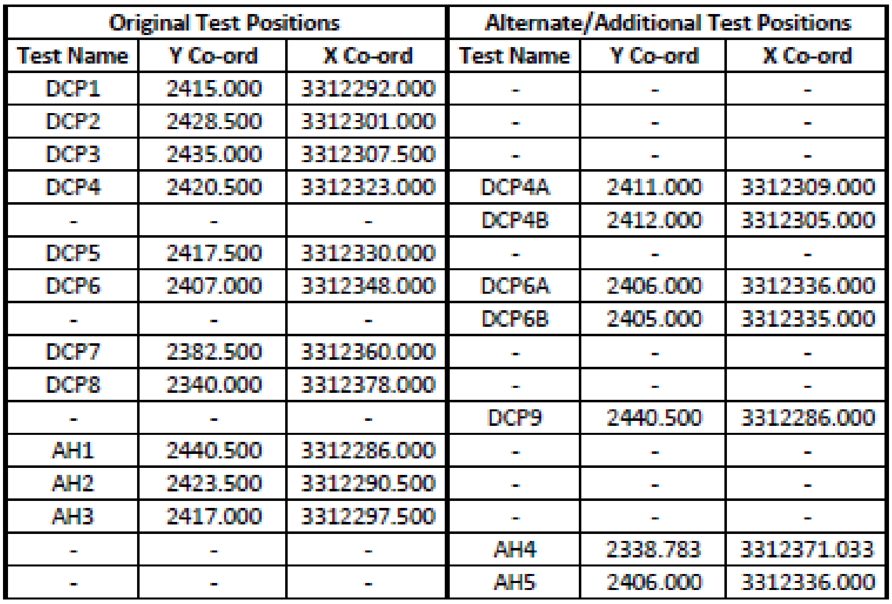
DESIGNED : A.J.
DRAWN : S.P.
DATE : 6/9/2019
SCALE : N.T.S.
CHECKED :

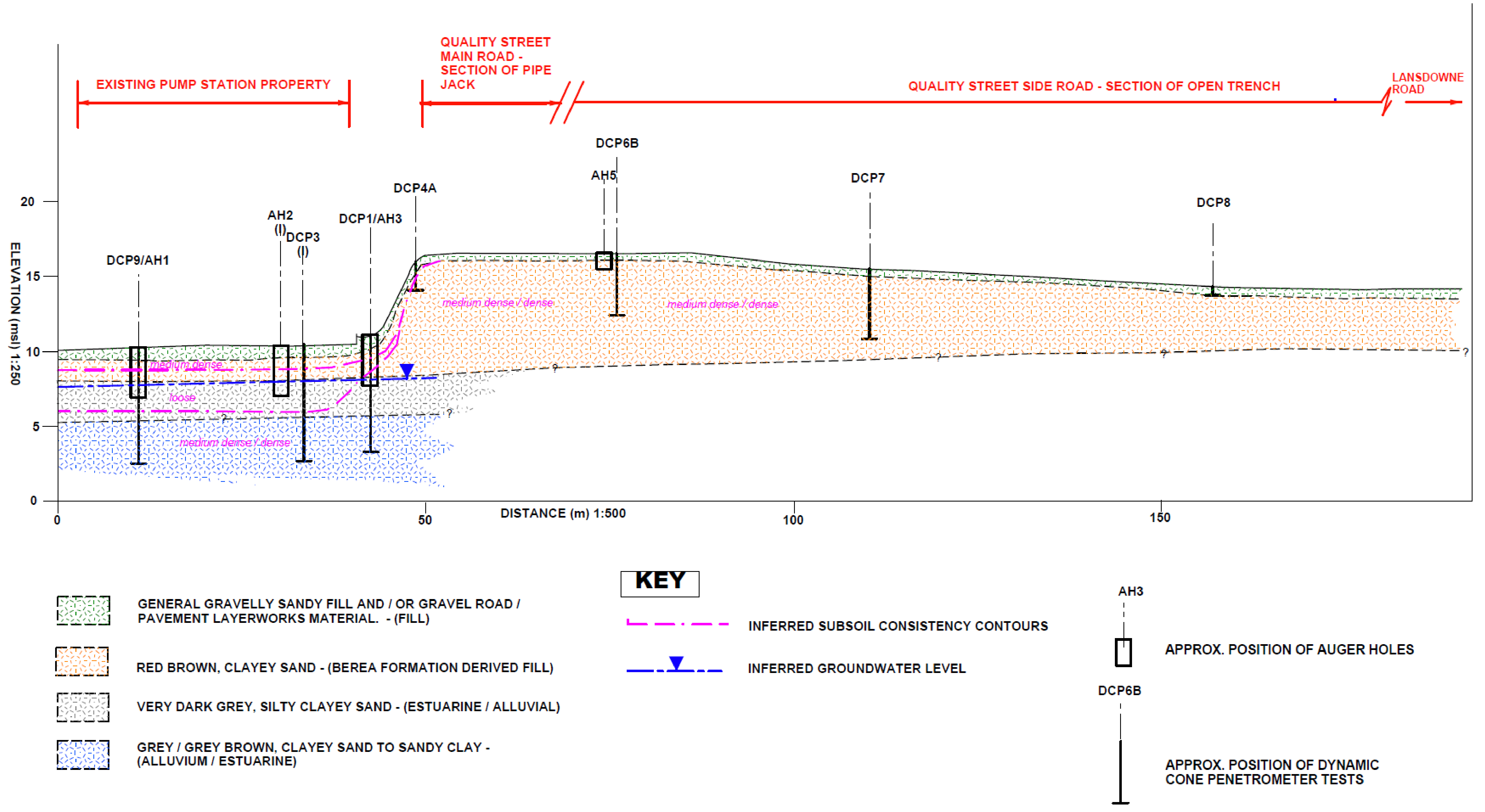
GEOTECHNICAL INVESTIGATION
FOR THE CONSTRUCTION OF A NEW SEWERAGE PUMP STATION
AND GRAVITY TRUNK SEWER AT LANSDOWNE ROAD
SITE PLAN

DRAWING NO.
32742-01



Original Test Positions			Alternate/Additional Test Positions		
Test Name	Y Co-ord	X Co-ord	Test Name	Y Co-ord	X Co-ord
DCP1	2415.000	3312292.000	-	-	-
DCP2	2428.500	3312301.000	-	-	-
DCP3	2435.000	3312307.500	-	-	-
DCP4	2420.500	3312323.000	DCP4A	2411.000	3312309.000
-	-	-	DCP4B	2412.000	3312305.000
DCP5	2417.500	3312330.000	-	-	-
DCP6	2407.000	3312348.000	DCP6A	2406.000	3312336.000
-	-	-	DCP6B	2405.000	3312335.000
DCP7	2382.500	3312360.000	-	-	-
DCP8	2340.000	3312378.000	-	-	-
-	-	-	DCP9	2440.500	3312286.000
AH1	2440.500	3312286.000	-	-	-
AH2	2423.500	3312290.500	-	-	-
AH3	2417.000	3312297.500	-	-	-
-	-	-	AH4	2338.783	3312371.033
-	-	-	AH5	2406.000	3312336.000

DRAWING 32742-02
Inferred Geological Section
A - A



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DESIGNED : A.J.
DRAWN : S.P.
DATE : 13/9/2019
SCALE : AS SHOWN
CHECKED :

GEOTECHNICAL INVESTIGATION
CONSTRUCTION OF NEW SEWERAGE PUMP STATION &
GRAVITY TRUNCK SEWER AT LANSDOWN ROAD
INFERRED GEOLOGICAL SECTION

DRAWING NO.
32742-02