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EXECUTIVE SUMMARY

Endecon Ubuntu (Pty) Ltd was appointed by The Richards Bay Industrial Development Zone Company SOC Ltd (RBIDZ) as Professional Engineering Consultant for the execution of a preliminary design of all civil infrastructure and related works for the development of a new Phase of the Richards Bay Industrial Development Zone. This development is known as Phase 1F of the IDZ in Alton North, Richards Bay.

This preliminary design report addresses the civil engineering services required for the development, and in particular the following:

<u>Water supply:</u>

The provision of water to the development in the industrial development zone of Richards Bay will be based on the existing Municipal bulk water infrastructure in close proximity to the location of Phase 1F. No alternative source of water will be pursued and a water connection onto the existing large diameter pipelines will be applied for with the water services provider.

The existing infrastructure for this pressure zone consist of the following.

- 2 x 20Ml bulk water storage reservoirs north of the site.
- An 800mm diameter bulk water gravity distribution main from the reservoirs to the service area on the western boundary of the development.
- A 200mm diameter bulk water gravity distribution main from the reservoirs to the service area on the eastern boundary of the development.

The township will be interconnected to the existing 800mm and 200mm water mains as indicated on the water network layout. The available pressure in the proposed water distribution system is generally low. Pressure boosting in one form or another may be a requirement. The large diameter water mains contribute to reduce the loss in pressure from the reservoir to the end user. The pressure reduction is mostly noticeable when calculating the residual pressures for a fire flow scenario. It is a possibility that the existing 200mm diameter pipeline in Alumina Allee be replaced by a 300mm pipeline. This possible replacement must be pursued during the final design stage.

Pressure boosting can be achieved by the implementation of an elevated reservoir with 4 hours of storage volume and a booster pumping station at storage reservoir level to pump water into the elevated reservoir as and when required. The main storage volume will be on ground level in the existing reservoirs. Such a booster pump station will have the capacity of at least the free flow requirements (144Kl).

This is also a design option that must be pursued during the final design stage.

Sanitation:

The whole of the Phase 1F development can be drained under gravity. The larger Alton North Township has been divided into three major catchments draining southwards towards the two 30m service corridors in the existing Alton Township.

The design of the eastern and western collector sewers in the two corridors has a capacity of 60l/s and 30l/s respectively. This is the flow to be accommodated from Alton North Township. The spare capacities available can be utilised but is proposed that the capacities be verified for the purpose of the final design report.

Once the spare capacities are fully utilised, a new outfall sewer will have to be constructed.

Existing sewers to the south of Phase 1F will be utilised as connection of the internal sewers. The capacity of these sewers must be verified for the purpose of the final design. The design of these sewers has however included the inflow from the phase 1F development.

It must be emphasised that it is at this stage not known what type of industry will be established on the site. The following composition of industry is accepted for the purpose of the preliminary design of sanitation services:

- Dry industry (workshops, etc): 70%
- Dry industry (warehousing): 15%
- Wet industry 15%

This composition will be utilised to obtain an average flow from each sewer connection point on the development.

It must further be stated that the relief of the land is such that large diameter sewers will have to be designed at low gradients in order to achieve sewer flow. Various modifications to the sewer system may be required to achieve functionality such as flushing systems connected to the water mains, etc.

Stormwater management

The total length of stormwater pipework is approximately 2500 m. The pipe sizes vary between 450mm Ø and 1800mm Ø.

The bulk of the stormwater will discharge into two drainage systems, one located in the western quarter of the site and the second in the eastern quarter of the site. A watershed through the middle of the site divides the two systems. The both systems discharge into stormwater servitudes placed at the drainage lines.

The stormwater system is designed to handle the runoff generated by the developing stands only in the 1 in 5 year recurrence interval storm event. The total catchment area of the developing sites is approximately 120 hectare.

The erven are relatively large industrial zoned erven. Each erf has been provided with a connection point. These connection points will be constructed as field inlet structures for the interim period, before the erven are developed. In the pre-development phase gravel berms or stormwater gulleys are proposed on the boundary of each erf to collect overland runoff from the erf and to discharge into the connection points. In post-development phase the field inlet structures can be demolished and replaced with either kerb inlet or junction box structures (Whatever suits the need of the developer). The erven future internal stormwater scheme will therefore connect onto the connection points provided at this stage.

Subsoil drainage

The water table on site is generally very high and construction of an extensive and effective sub-soil system is encouraged. This is of utmost importance specifically for the protection of the structural layers in the roads.

Subsoil drains will generally be placed on both sides of the roads at a depth of approximately 1.5m. In the areas where ground water conditions warrant additional drains, these will be provided underneath the road layer works and even in the bedding of the pipework. The subsoil drainage system is not indicated on the layout as it will be determined on site based on the conditions.

Access road

The Traffic Impact Assessment (TIS) is not completed yet. The TIS will largely determine the design parameters of the access road. The roadway will be constructed in accordance with the requirements of the local authority, being for a 15.8m roadway (4 x 3,7m lanes + 2 x 0,5m shoulders with parking on either side of 2,5m wide).

Thus a 20,8m wide surfaced area if parking is included. A Kerb-channel combination will be installed on both sides of the road. The pavement structure will be designed for an "Industrial road" with a pavement class and design bearing capacity of ES10.

The layerworks will consist of the following:

- Surfacing : 30mm Asphalt
- Base : 150mm G1 Base (commercially obtained)
- Upper Subbase : 150mm C4 (imported from commercial sources)
- Lower Subbase : 150mm C4 (imported from commercial sources)
- Upper Selected : 150mm G7 (imported from commercial sources)
- Lower Selected : 150mm G9 (possibly obtainable on site, to be verified by Geotechnical investigation)
- Roadbed : Rip and re-compact insitu to 90%

Railway infrastructure

There will be a staging yard on the property's south eastern corner on the municipal servitude. The feeder line will run on the south eastern side of the industrial stands next to the entrance road. The stand sidings allows for approximately 30m of access but the main service railway line will have to be crossed by a level crossing should the road in future be constructed through.

The costing of the proposed infrastructure is contained in Chapter 11 of the report.

Geotechnical Investigation

A geotechnical investigation was conducted as part of the preliminary design procedure. The findings are briefly discussed.

The site is underlain by medium dense generally grading from about 1.0m into compressible silty to clean sands or fully expanded sandy clays with low shear strengths resulting in poor founding conditions due to lowered bearing capacity and increased settlement potential. The site will therefore require:-

- Good site drainage including both exclusion (perimeter interception drains) and storm water drainage on site.
- Modified construction techniques, such as compaction beneath footings and/or use of engineered fills, together with reinforcing and construction joints. Most are shown as having a deeper loose soil profile – then slightly more construction modifications, or deeper compaction will be required.
- Depending on foundation design pressures and allowable bearing pressures on the compacted foundation substrates, piling may have to be considered as an alternative to compaction, where heavier structures and/or tolerance to differential settlement is low. This will be a comparative cost analysis between importing fill material and/or stabilized substrate compaction, versus the cost of piles.

If piling is required for larger structures, or where designs cannot accommodate some tolerance to settlement, even after in situ compaction/soil improvement, then it is recommended that a DPSH (CPT) rig be used to check the deeper soil densities and cone penetration resistance for design of friction length and refusal depth of the piles.

1. INTRODUCTION

Endecon Ubuntu (Pty) Ltd was appointed by The Richards Bay Industrial Development Zone Company SOC Ltd (RBIDZ) as Professional Engineering Consultant for the execution of a preliminary design of all civil infrastructure and related works for the development of a new Phase of the Richards Bay Industrial Development Zone. This development is known as Phase 1F of the IDZ in Alton North, Richards Bay.

This preliminary design report addresses the civil engineering services required for the development, and in particular the following:

- Bulk water connection
- Internal sewer reticulation and bulk connection
- Internal streets and parking areas/bays
- Storm water drainage
- Intersection of access road
- Railway link connection and sidings

2. DESIGN CRITERIA

- The design criteria used as a guideline in the preliminary design of the services as mentioned above is the publication “Guidelines for the Provision of Engineering Services and Amenities in Township development”. This criterion is widely accepted as the guideline for the design of Municipal Services.
- TRH 4: Structural Design of Flexible Pavements for Interurban and Rural Roads.
- DRAFT UTG7: Geometric Design of Urban Local Residential Streets.
- TRH 3: Surfacing Seals for Rural and Urban Roads.

3. LOCALITY, TOPOGRAPHY AND BACKGROUND

General layout and locality

The site is situated on the northern boundary of the existing Industrial Township of Alton and approximately 1.5km to the west of the Central Business District of Richards Bay. It is bounded on the south by the TATA development and the future access road, on the east by the future Central Freeway, on the north by the cemetery and Mandlazini reservoirs and on the west by the railway marshalling yard and future Western Arterial. See enclosed Locality Plan. The developable land has a surface area of approx. 100 hectares. The area to be developed is approx. 60 hectares as a result of a 60% coverage factor.

The terrain is generally very flat with some large pans and slopes gradually toward the south at approximately 0.4%. There is a high lying area, a sand dune, on the northern portion of the site. The site falls from an elevation of approx. 67m ABSL in the north to approx. 42m -44m on the southern boundary.

Two natural watercourses traverse the site from north to south parallel to the eastern and western boundaries, abiding the site in approximate thirds.

Geology and topography

A geological investigation by Golder & Associated states the following in terms of Topography and geology. The topography of the area is characterized by three broad landforms. The coastal area comprise Neogene marine and coastal aeolian sediments; moving inland, and in a broad band parallel to the coastal sediments and curving around to incorporate the areas north of Empangeni comprises a section of Post-African surface (partly planed); the area south of Empangeni comprises dissected landforms of various ages, with major structural control common.
With reference to this plan, fifteen geotechnical zones were identified.

Vegetation

The Maputaland Coastal Plain is characterised by relatively flat to slightly undulating paeleodune fields. The majority of these areas feature approximately north/south orientated drainage systems linked with the dune-slacks and depressions created in these historic windblown landscapes.

Much of the hydrology is linked with the shallow water table of the coastal plain. Wetlands tend to be an expression of the groundwater / soil surface interface. The wetness regime varies seasonally with associated fluctuations in the local depth to the water table.

At a broad-scale, the project sites are characterised by a mosaic of indigenous coastal forest, Swamp Forest, grasslands, hygrophilous grasslands, reed swamps and areas of open water. Anthropogenic disturbances have resulted in areas of indigenous vegetation being replaced by alien plant encroachment, formal plantations, bare earth and infrastructure. The level of disturbance varies between sites.

4. TOWN PLANNING AND LAYOUT

The following table indicates the land use for Phase 1F. Annexure B to the report reflects the development layout plan and proposed site layout.

Table 4.1: Land use table

Stand No.	*Area m2*	*Description*	*FAR*	*Effective Area m^2*
16786	205696	Industrial	0.6	123417.6
16787	79654	Industrial	0.6	47792.4
16788	164750	Industrial	0.6	98850
16789	167275	Industrial	0.6	100365
16817	56970	Industrial	0.6	34182
16818	45514	Industrial	0.6	27308.4
16819	24179	Industrial	0.6	14507.4
16820	23375	Industrial	0.6	14025
17442	23375	Industrial	0.6	14025
17443	23375	Industrial	0.6	14025
17455	27692	Industrial	0.6	16615.2
17456	24377	Industrial	0.6	14626.2
Portion 1	50858	Industrial	0.6	30514.8
Portion 2	50005	Industrial	0.6	30003
portion 3	39529	Industrial	0.6	23717.4

The above table only represents stands that requires services. No roads or open areas are included into the table.

Various servitudes fall within the development. These servitudes are also indicated on the layout plan (attached as Annexure A) and are generally located on the boundaries for the following purposes as indicated:

- Electrical servitudes (35m wide) situated between Erf 17456 and Erf 17455;
- Stormwater servitude (30m wide) situated between Erf 17456 and Erf 17455;
- Stormwater servitude (30m wide) situated between Erf 16819 and Erf 16818;
- Road servitude (25m wide) situated on the western boundary of Erfs 16818 portion 1 and portion 2;
- Main entrance road servitude (40m wide).

5. ENVIRONMENTAL CONSIDERATIONS

The more important environmental consideration pertaining to the site is the wetland areas present. These wetland areas has resulted in specialist studies being undertaken on the site. The following are the abridged findings on the wetland delineation specifically for Phase 1F

Terrain indicators and hydro-geomorphology

Wetlands on the eastern portion have been influenced by the development of the channel. This system has resulted in notably drying of portions of the site. The flat nature of this portion of the property has added to the effect of the drain by influencing systems much further away from the channel. The second wetland system to the north-west remains largely intact. This system is likely to have had little disturbance and it is only the reduced water availably linked with the gum trees that is likely to be influencing this system

Soil indicators

The majority of the eastern wetlands on site were characterised by semi-permanent and seasonally waterlogged hydric soils linked with the pans and the lower lying depressions on this portion of the site. Some temporary waterlogged hydric soils occurring within and around the project site. The north western pan was as above but also included elements of permanent wetland. Soils were generally sandy in nature with increasing organic levels linked with increase moisture levels. Mottling was limited due to the generally inert nature of these regic sands and quarts derived soils.

Vegetation indicators

The wetlands on the eastern portion maintained a diverse and valuable range of plant communities. Many of the systems were characterised by smaller areas of reed swamp surrounded by extensive hygrophilous grassland The Pan systems were dominated by Phragmites reed beds in the wetter portions, with a shift in community composition as one moved away from the pan onto the drier portions of the site. Seasonal and temporary wet areas were characterised by low sedge and hygrophilous grass communities. encroachment by Eucalyptus poses the largest threat to these systems with respect to alien plants.

The wetland delineation of the different IDZ sites identified a number of systems in various states of health and functionality. Many of the wetland still had a notable PES and performed many of the ecosystem services to a high degree. However, two sites were of distinctly higher quality and this is where we believe some element of compromise needs to be reached between the IDZ Company and the conservation authorities. IDZ 1C and the western portion of IDZ 1D have very high conservation significance and it is felt that these areas should be excluded from any development planning for the area. Rather, development should be focused on IDZ 1A, 1B, 1F and the eastern portion of 1D, including encroachment or loss of some of the smaller wetlands on these portions, in compensation for ensuring protection of the high value systems.

6. GEOTECHNICAL ASSESSMENT

6.1 Background

A geotechnical investigation was undertaken by Golder & Associates in 2005.

This strategic level assessment of the geotechnical conditions in the uMhlatuze Municipal Region (KZ282) has been undertaken to provide a supporting document and guidance for future industrial and other developments in respect of geotechnical conditions and associated development categories. The aim is to use the information to classify areas for the purpose of identifying constraints to development as well as difficult founding conditions, or other geotechnical factors affecting urban development. This document has been prepared by Golder Associates Africa, (GAA). The assessment was carried out from January to December 2004 and involved both desk studies and field investigations. The document provides a description of the results obtained together with recommendations concerning geotechnical management within the project area. This report supplements the database and accompanying atlas of maps.

The terms of reference for the investigation are as follows:

- to establish the nature and relevant engineering properties of the upper soil and rock strata underlying the UMA;
- to present foundation options and recommendations for typical developments;
- to comment on suitable excavation procedures for the installation of foundations and services;
- to present comments on the use of the on-site soils in the construction of bulk fill and roads;
- To comment on site water management aspects, particularly pertaining to shallow groundwater or seepage;
- to comment on any other geotechnical aspects that may affect the development

The investigation presents the findings of the investigation taking into account inter alia the following:

- Compressible soils
- Erodability of soils
- Expansive clay soils

- Shallow groundwater or seepage
- Difficulty of excavation to 3,0m depth
- Areas below floodlines
- Stability of temporary excavations

The study was undertaken in the following phases:

- **Desk study**: This comprised the retrieval and compilation of available geological and geotechnical data from existing information within the Umhlathuze archives, as well as the consultation of published geological maps.
- **Air Photo interpretation**: this entailed a detailed air photo interpretation of stereo pair air photographs. This will be used to identify and demarcate land facets that represent the different geotechnical conditions expected. The results of the desk study will be used to help calibrate the air photo mapping.
- **Field Investigation**: The field investigation comprised a field visit to inspect the available soil exposures, e.g. road cuttings, borrow pits, erosion gullies etc. A number of control test pits (new and existing) were used to confirm and characterise the geotechnical land facets.

The investigation done by Golder & Associates is considered very broad and not site specific.
A more comprehensive geotechnical investigation is being undertaken for phase 1F in particular.

6.2 Site specific Geotechnical investigation (Engeolab)

Terms of Reference

This report presents the results of an engineering geological investigation into the foundation conditions for a proposed industrial development Phase 1F, Alton, Richards Bay.

Mr. G.S. Engelbrecht *Pr.Eng.* of Messrs. ENDECON UBUNTU (Pty) LTD Engineering Consultants requested the site investigation on behalf of the Richards Bay Industrial Development Zone Company SOC Ltd (RBIDZ). The appointment is contained in a letter from the acting CEO, Mr. M. Nkopane, dated 18 April 2013, Reference RBIDZ – 1F/IPD/001/12. The fieldwork was carried out on the 9th and the 14th May 2013. The site investigated is indicated on the Locality Plan, Figure 1 in the beginning of the report.

The objectives of the geotechnical investigation on were to:-

i) Determine the soil profile across the site and evaluate its engineering properties and influence on the design of light single storey structures;

ii) Assess the groundwater conditions, including surface run-off, ponding, seepage, and perched or permanent water tables;
iii) Evaluate the workability of the site materials with regard to their excavatability and compactability and comment on other nearby sources of suitable stable fill materials;
iv) Comment on predicted safe bearing capacity values, expected heave and settlement of the different potential founding horizons and recommend founding depths.

Site Details and Assumptions

It is understood that the proposed structures will comprise elevated single and double storey structures for industrial purposes with foundation pressures not exceeding 150KPa.

The 159ha undulating site is poorly drained, with surface runoff draining into gulleys and low lying areas, forming local wetlands which slowly seep into a central canal draining southwards to Lake Nsezi located to the south of the terrain.

Available Information

Information was obtained from the following sources:-
i) A site plan, provided by the Client showing the existing and proposed township layouts;
ii) The 1:250,000 scale geological map 2732 St. Lucia was also consulted.

A site plan showing the layout and the position of the test holes is attached as Figure 2 on the opposite page.

6.2.1 <u>Method of investigation</u>

Test Pitting and Profiling
Thirty five test pits were excavated around the site using a tractor-loader-backhoe (TLB) provided by Leomat Plant Hire of Richards Bay.

The test pits were excavated to the maximum reach of the backhoe, some 2.5m to 3.0m below surface. The test pits were profiled from above in terms of moisture, colour, consistency, structure, soil type and origin.

The soil profiles are attached as **Appendix A**, and the positions of the test pits are indicated on the appended drawings.

Sampling and Laboratory Testing

Six undisturbed and twenty three disturbed samples representative of the various soil layers were selected for laboratory testing to confirm the in situ assessments of moisture, grading, plasticity, consistency, structure and to ascertain the engineering properties of each horizon.

The following tests were carried out on the samples:-

i) Two Foundation Indicator tests were done on clayey samples comprising particle size distributions analyses (sieve and hydrometer gradings) and Atterberg Limit tests. Twenty one road indicators were done, the hydrometer being excluded due to non-plasticity of the sandy samples.

These tests permit a basic classification of the soils and group them according to typical engineering properties.

ii) Six compaction tests comprising Modified AASHTO moisture/density relationships and California Bearing Ratio Values.

These tests evaluate the compaction characteristics of the site soils and permit an evaluation of their suitability of use as construction materials.

A number of undisturbed samples were taken but the material was so loose that although they were carefully taken and wrapped, the samples disintegrated and could not be tested. Copies of the laboratory tests are attached as **Appendix B** and a summary of the test data is attached for convenience.

Dynamic Cone Penetration Tests (DCP's)

Fifteen hand-held DCP's were carried out from surface to 2m below, adjacent to each test pit to verify the in situ described soil consistencies.

With some adjustments (partly after Terzaghi & Peck, 1967), the DCP test is regarded as being comparable with the SPT (Standard Penetration Test) test, for which N values have been correlated with soil consistencies for various soil types – that is mainly sand and clays.

These penetration tests are made with portable DCP where a 60° cone of compressed steel is forced into the ground, the impact provided by means of an 8kg sliding hammer falling 575mm. the penetration achieved per five blows are recorded and refusal is usually indicated as soon as negligible penetration (<1mm) is recorded after 10 blows.

The DCP results and interpretations are attached in **Appendix C** at the back of the report and the average N-values of the soil profiles where undisturbed samples were taken (TP) were noted on the profile descriptions.

6.2.2 Site soils and Geology

The site is underlain by redistributed Quaternary sandy and clayey soils that blanket the coastal section of Richards Bay.

The average soil profile on the site comprises very loose, grey-brown sand grading through light brown to yellowish brown with a loose to medium dense consistency, mainly silty to clean, equigranular sand (that is sand with no binding material) down to an average depth of some 2.5m below surface.

Without exception, the site materials pass the 2.0mm (sand-grade) sieve, recording an average grading modulus of 0.98 with little binding material (small silt and clay fractions), hence the low plasticity and negligible clay fractions. However, two samples taken from the deeper clayey profile exposed in TP11, recorded plasticity values ranging from 12 – 16 and an average grading modulus of 0.62 (note that the clayey horizon is generally present from 1.3m below surface and was recorded in TP's 10,11,12,16 and 17. In terms of pavement construction material, the site soils generally comply with the operational requirements of G8 and G9 class pavement construction material.

6.2.3 Groundwater

Slight seepage was encountered in 50% of the test pits excavated at a depth of approximately 2.0m below surface. Although no seepage was encountered in any of the other test pits, the soils were profiled as being very moist at the base of the test pits indicating that the groundwater is just below the base of the excavation. Seepage is evident in the lower lying portions of the site, covering about 59% of the site. The only 'dry' test pits were those excavated in the northern portion of the site on some 41% near the reservoirs where cover soils are much thicker.

6.2.4 Foundation assesment

The soil strata have been examined and tested to determine their suitability as founding horizons for the proposed structures according to the following criteria:-

- Strength of the founding materials indicated by the N values and estimated allowable bearing capacities.
- Compressibility of the founding materials measured in terms of their estimated deformation moduli.
- Potential heave in the residual soils.
- Predicted settlement/heave from the above factors.

A summary of the foundation assessment is provided below.

Bearing Capacity

The site is underlain by equigranular Quaternary sand with clayey lenses in places, generally silty to clean sands, with a medium dense consistency, becoming loose with increasing depth. The estimated allowable bearing capacities are provided in the table below, based on in situ profile and DCP observations.

Table 6.1 : Estimated Allowable Bearing Capacity

Soil Description	Average Layer Depth (m)	Estimated Allowable Bearing Capacity (kPa)
Grey sandy cover soil	1.0	<25
Beige- brown lightly coloured sandy soils with clayey lenses in places	1.0 - 3.0	25 - 75

The estimated allowable bearing capacity of the foundation materials is a function of the angle of internal friction (reflected in the grading), consistency (soil density) and degree of saturation (moisture content). The bearing capacity is only a guide to the maximum load that can be placed on the soil without shear failure, and as such is not an indicator of the settlement that will occur at foundation pressures up to the bearing capacity of the soil. Consequently the allowable bearing capacities tabulated above include a factor of safety on the foundation loads to reduce the likelihood of shear failure, which helps to reduce settlement, not eliminate it – refer following paragraphs on settlement.

Note also that the aforementioned bearing capacities are based on the materials exposed in situ in the test pits and ignore improvement, which may be gained by compacting, or treating the site soils.

Estimated Compressibility

Based on the assessment of the soil profiles, the soil strength descriptions are given in the table below.

Table 6.2 : Estimated Compresssibility/Deformation

Soil Description	Consistency	Average Depth (m)	Estimated (Ed) MPa	Estimated Settlement	NHBRC Classification
Grey sandy cover soils	Medium dense	0 - 1.0	12 – 20	5 – 10	C1
Beige-brown lightly coloured sandy soils with clayey lenses in places	Loose	1.0 - 3.0	10 – 20	10- 20	C2, S2

**Estimate based on a foundation pressure of approximately 50kPa.*

Predicted Settlement

Total settlement where Quaternary soils form the foundation substrate and are shallower than 1000mm – is expected to be between 10 and 20mm at a foundation pressure of 45kPa. Although the collapse potential of these soils is expected to be low because of the high moisture content of the in situ soils, minor collapse settlement could cause the total settlement to exceed 20mm at 45kPa, if the foundation soil were to become further saturated. The overall settlement in these in situ ground conditions classifies as C2/S2 in the SAICE building recommendations (Reference 15 & 16, Table 1 in the Appendix), requiring modified construction techniques, including but not limited to:

- Reinforced strip footings and reinforcement of masonry;
- Articulation joints at some internal & all external doors;
- Remove in situ material below foundation to a depth and width 1.5 times the foundation width for spread footings and to 2.5 times the foundation width for strip footings, or remove down to a competent horizon and replace with suitable selected material compacted to 93% Modified AASHTO density at -1% to +2% of optimum moisture content;
- Site drainage, impermeable concrete/brick apron around the structures to limit ingress of water;
- Additional service/plumbing precautions relevant to accommodating movement – such as flexible joints;
- Foundation pressures not to exceed 50kPa on compacted substrate without further analysis.

Total settlement where the sandy cover soils are thicker than 1000mm is expected to exceed 20mm at a foundation pressure of 45kPa. The overall settlement in these in situ ground conditions classifies as C2/S2 in the SAICE building recommendations, requiring modified construction techniques with additional stiffening of footings and/or raft foundations and fabric reinforcement in floor slabs.

Where foundation pressures are expected to exceed 50kPa, or where the design of the structure cannot accommodate the loads, or where tolerances to differential movement are low, then piles (or mini-piles), or deep dynamic compaction should be considered – refer to Table 1 in the Appendix.

Excavatibility

Based on the excavation depths achieved with the backhoe, the excavatability of the site materials is classified according to the SABS 1083 as tabulated in Table 6.3.

Table 6.3 : Excavation Characteristics

Soil Description	Average Depth (m)	Excavatability SABS 1083	Excavation Class *	Excavation Characteristics
Loose to medium dense (silty) fine sand and clayey sand	Surface to 3.0m	Soft	1	Hand/machine excavation

**Kirsten, HAD. A classification system for excavation in natural materials. The Civil Eng. In SA, 82.*

Note that the sandy soil has relatively good shear strength (Ø' = 15° and cohesion C' = 16kPa) and vertical excavations deeper than 2.5m, or below the perched water table will need supporting and/or cutting back to a stable slope of between 45° and 60°.

Note also that where very moist or wet soils were encountered, the stand-up time of excavations were less than 1.0m and in several instances, the sidewalls slumped into the test pit excavation.

Compactibility of Materials

The foundation soils only comply with G7 to G9 construction materials – and as such are poor in terms of their compactability, although with the addition of cement stabilization higher unconfined compressive strength(s) will be obtained.

Table 6.4 : Summary of compaction results

Hole No.	Depth (m)	Material Description	GM	PI	Mod. AASHTO	CBR @ 93%	G Class
TP4	0.1 - 0.8	Sand	1.3	NP	1678	9	G8
TP4	0.8 - 3.0	Sand	0.96	NP	1721	8	G9
TP9	0.1 - 1.1	Sand	1.2	NP	1718	11	G8
TP11	0.1 - 0.9	Sand	1.06	NP	1779	9	G9
TP11	1.9 - 2.3	Sand	0.98	NP	1741	11	G8
TP12	0.1 - 1.3	Sand	1.15	NP	1798	12	G8

Note: PI – Plasticity Index GM – Grading Modulus CBR- California Bearing Ration at 93% Mod.

Based on the results of the compaction tests, it is evident that these Quaternary sands may be suitable for use as general fill underneath surface beds and for possible use as lower subbase layers. Selected layers or subgrade in road construction, after carefully removing all organic material.

Chemical stabilization and/or mechanical modification of the foundation soils is expected to considerably improve the compacted strength of the in situ soils. The average maximum Modified AASHTO density in these soils is approximately 1720kg/m³ without stabilization.

Cognizance should be taken of the potentially compressible and collapsible nature of the upper insitu soil horizons and the consequent need for adequate subgrade preparation. Upper subbase and basecourse quality material will have to be imported for construction purposes.

The above compaction figures assume good pre-site drainage since all densities and CBR figures are quoted at optimum moisture content, which moisture level cannot be achieved when the site conditions are at, or near saturation.

Active Clays

The equigranular sandy cover soils are either non-plastic or slightly plastic with the sandy/clayey lenses exposed >1.3m in TP’s 10, 11, 112, 16 and 17 being more plastic. Generally the sandy soils are non-active with the medium active plastic soils fully expanded due to the high in situ moisture content.

6.2.5 Conclusions

The site is underlain by medium dense generally grading from about 1.0m into compressible silty to clean sands or fully expanded sandy clays with low shear strengths resulting in poor founding conditions due to lowered bearing capacity and increased settlement potential. The site will therefore require:-

- Good site drainage including both exclusion (perimeter interception drains) and storm water drainage on site.
- Modified construction techniques, such as compaction beneath footings and/or use of engineered fills, together with reinforcing and construction joints. Most are shown as having a deeper loose soil profile – then slightly more construction modifications, or deeper compaction will be required.
- Depending on foundation design pressures and allowable bearing pressures on the compacted foundation substrates, piling may have to be considered as an alternative to compaction, where heavier structures and/or tolerance to differential settlement is low. This will be a comparative cost analysis between importing fill material and/or stabilized substrate compaction, versus the cost of piles.

If piling is required for larger structures, or where designs cannot accommodate some tolerance to settlement, even after in situ compaction/soil improvement, then it is recommended that a DPSH (CPT) rig be used to check the deeper soil densities and cone penetration resistance for design of friction length and refusal depth of the piles.

6.2.6 Zonal Demarcation

The site has been demarcated into three development zones as follows:-

Zone 1 - 64.44 ha; NHBRC Class C2, S2

Main geotechnical constraints recorded within Zone 1 are compressible and potentially collapsible soils with low bearing capacity, requiring modified construction techniques.

Zone 2 - 73.7 ha; NHBRC Class P, C2, S2

As above but this zone is subject to seepage and seasonal ponding which will require adequate drainage.

Zone 3 - 19.59 ha; NHBRC Class P

Zone 3 comprises 5 areas with recognized wetlands – no development recommended.

6.2.7 General

While every effort has been made to ensure that representative test pitting and sampling has been undertaken on-site, it is impossible under the constraints of an investigation of this nature to guarantee that isolated zones poorer foundation conditions have not been identified. The investigation has sought therefore to highlight general foundation, construction and excavation problems, and to provide early warning to the design engineers. In view of the variability inherent in soils, a competent person must inspect all foundation excavations.

The placement of the engineered fills must be controlled with suitable field tests to ensure that the required densities are achieved during compaction, and that the quality of fill material is within specification.

All appeddixes referred to in this Chapter is contained in Annexure G

7. TRAFFIC IMPACT ANALYSIS

Endecon is in the process of conducting a Traffic Impact Assessment for the IDZ. The information and subsequent guidelines will be available for the final design stage.

8. EXISTING INFRASTRUCTURE

The following existing infrastructure will be utilised in order to provide engineering services to the Phase 1F development:

Water:

- Existing Mandlazini 20Ml bulk water storage reservoirs north of the site
- An 800mm diameter bulk water gravity distribution main from the reservoirs to the service area on the western boundary of the development.
- A 600mm diameter bulk water gravity distribution main from the reservoirs to the service area on the eastern boundary of the development.

Sanitation:

- Existing collector sewers exist. eastern and western collector sewers in the two corridors has a capacity of 60l/s and 30l/s respectively.
- New link sewers on the southern boundary of the development will be utilised.

Roads infrastructure:

- The existing road infrastructure on the eastern perimeter will act as the access road into the site. A connection of the proposed new road onto Alumina Alley street will be implemented.

Railway infrastructure

- The existing railway infrastructure and railway servitudes to the west of the site will be utilised.

Stormwater drainage

- The bulk of the stormwater will discharge into two drainage systems, one located in the western quarter of the site and the second in the eastern quarter of the site. A watershed through the middle of the site divides the two systems. The both systems discharge into stormwater servitudes placed at the drainage lines.

9. DESIGN PARAMETERS AND PHILOSOPHY

9.1 WATER SUPPLY

The provision of water to the development in the industrial development zone of Richards Bay will be based on the existing Municipal bulk water infrastructure in close proximity to the location of Phase 1F. No alternative source of water will be pursued and a water connection onto the existing large diameter pipelines will be applied for with the water services provider.

The existing infrastructure for this pressure zone consist of the following. Please refer to the water supply scheme indicated in Annexure A to the report.

- 20Ml bulk water storage north of the site
- An 800mm diameter bulk water gravity distribution main from the reservoirs to the service area on the western boundary of the development.
- A 600mm diameter bulk water gravity distribution main from the reservoirs to the service area on the eastern boundary of the development.

Connections onto existing services:

A general layout of the scheme is contained in *Annexure B* of this report. The township will be interconnected to the existing 800mm and 200mm water mains as indicated on the water network layout.

Available pressure:

The available pressure in the proposed water distribution system is generally low and pressure boosting in one form or another may be a requirement. The reservoir water level at 70% full is 72m AMSL. Table 9.1 reflects the available static pressure at the stand connection points. The large diameter water mains contribute to reduce the friction loss in pressure from the reservoir to the end user. The pressure reduction is mostly noticeable when calculating the residual pressures for a fire flow scenario. This is discussed under paragraph 9.1.1 and is reflected in Table 9.3

Table 9.1: Water pressure calculation

Stand no take-off	Elevation m AMSL	Static Head (m)
16786	43	24
16787	48	21
16788	51	23
16789	49	31
16817	41	31
16818	41	31
16819	42	29
16820	43	29
17442	44	28
17443	44	28
17455	44	28
17456	44	28
Portion 1	41	31
Portion 2	41	31
portion 3	39	33

Elevation of reservoir at 70% full is 72m AMSL

<u>Pressure boosting:</u>

Pressure boosting is not a requirement as the available pressure in the system during peak domestic water withdrawal is above the minimum allowable pressure of 2.4Bar except for stands 16786, 16787 and 16788. The pressures are however very close to the minimum allowable minimum pressure. It may be a municipal requirement to provide a pressure boosting facility. This feature will be finalised during the final design stage of the project.

Pressure boosting can be achieved by the implementation of an elevated reservoir with 4 hours of storage volume and a booster pumping station at storage reservoir level to pump water into the elevated reservoir as and when required. The main storage volume will be on ground level in the existing reservoirs. Such a booster pump station will have the capacity of at least the free flow requirements (144Kl).

9.1.1 WATER DEMAND CALCULATIONS:

Table 9.2 depicts the water demand figures for Phase 1F of the Richards Bay IDZ.
It must be emphasised that it is at this stage not known what type of industry will be established on the site. The following composition of industry is accepted for the purpose of the preliminary design of sanitation services:

- Dry industry (workshops, etc): 70%
- Dry industry (warehousing): 15%
- Wet industry 15%

Table 9.2: Water demand calculation

Stand No.	*Area m2*	*FAR*	*Effective Area m^2*	*Water demand @ 225l/100m^2 (Kl AADDg)*	*Peak seasonal demand (AASD)*	*Peak daily domestic flow (Kl/day)*
16786	205696	0.6	123417.6	319.3	479.0	1676.6
16787	79654	0.6	47792.4	123.7	185.5	649.2
16788	164750	0.6	98850	255.8	383.7	1342.8
16789	167275	0.6	100365	259.7	389.5	1363.4
16817	56970	0.6	34182	88.4	132.7	464.3
16818	45514	0.6	27308.4	70.7	106.0	371.0
16819	24179	0.6	14507.4	37.5	56.3	197.1
16820	23375	0.6	14025	36.3	54.4	190.5
17442	23375	0.6	14025	36.3	54.4	190.5
17443	23375	0.6	14025	36.3	54.4	190.5
17455	27692	0.6	16615.2	43.0	64.5	225.7
17456	24377	0.6	14626.2	37.8	56.8	198.7
Portion 1	50858	0.6	30514.8	79.0	118.4	414.5
Portion 2	50005	0.6	30003	77.6	116.4	407.6
portion 3	39529	0.6	23717.4	61.4	92.1	322.2

Abbreviation legend:

AADD = Annual average daily demand, for the purpose of determining the annual water demand.

AADDg = Annual average daily demand inclusive of system transition losses. The losses are taken at 15% of AADD.

AASD = Annual average seasonal demand, for the design of reservoirs and bulk infrastructure.

Domestic water demand:

Annual average daily demand **(AADDg)**	3,195 Kl/day (18 l/s)
Seasonal Peak Factor	1.5
Seasonal Peak **(AASD)**	4,793 Kl/day
Instantaneous Peak Factor	2.4
Instantaneous Peak	11,502 Kl/day
With a 24 hour day withdrawal period	133 l/s (Design Flow) for internal water network as well as the gravity main from the reservoir.

Normally the larger figure of the water demand or the fire water supply demand would take preference when designing the water supply/distribution system.

Fire water demand:

The provision of water for fire fighting should comply with the requirements as described in SANS code of practice 090:1972 – Community protection against fire.

The design of the fire water mains will be based on the rational fire design that will be submitted for Council approval by each individual stand developer. This rational fire design will be based on the type of building and the type of industry that will be practised. Our aim in this report is to apply the general requirement rule as indicated in the publication "Guidelines for the Provision of Engineering Services and Amenities in Township development".

The preliminary fire design is based on the following criteria.

"Guidelines for the Provision of Engineering Services and Amenities in Township development" Fire flow calculation criteria:

• Fire risk category:	Moderate	(Chapter 9, p33)
• Design fire flow:	6,000 l/minute	(Table 9.18, p35)
• Number of hydrants:	All within a 270m radius	
• Duration of fire flow:	4 hours	(Table 9.19, p35)
• Minimum residual head:	15m	(Table 9.20, p36)

- Location of hydrants: 180m (Table 9.21, p36)

A determining factor in the volume of water to be available at the point of discharge is the pressure within the pipe reticulation system. The available static pressure in the system at the lower erven is approx. 29m. The dynamic pressure with each individual stand discharging water at a time is in the region of 15m. This residual pressure can be achieved with only one hydrant discharging at 25l/s.

If the fire rational design requires a flow higher than 25l/s for each stand then on-site fire storage and pressure boosting be considered.

Table 9.3: Water demand and related residual pressures *(Note: 1 bar = 10m pressure)*

Stand no.	Peak domestic water demand (2.4 x AASD)			Fire option 1		Fire option 2	
	Kl/Day	Flow l/s	Residual Pressure (bar)	1 Hydrant + Peak flow	Residual Pressure (bar)	2 x hydrants only	Residual Pressure (bar)
16786	2350.4	27.2	2.29	52.2	1.72	50	1.78
16787	910.2	10.5	2.38	35.5	2.10	50	1.55
16788	1882.5	21.8	2.24	46.8	1.66	50	1.57
16789	1911.4	22.1	2.43	47.1	2.10	50	2.06
16817	651.0	7.5	2.79	32.5	2.49	50	2.16
16818	520.1	6.0	2.78	31.0	2.26	50	1.63
16819	276.3	3.2	2.71	28.2	2.52	50	2.29
16820	267.1	3.1	2.82	28.1	2.64	50	2.41
17442	267.1	3.1	2.53	28.1	2.64	50	2.11
17443	267.1	3.1	2.45	28.1	2.64	50	2.05
17455	316.4	3.7	2.52	28.7	2.34	50	2.11
17456	278.5	3.2	2.47	28.2	2.34	50	2.17
Portion 1	581.1	6.7	2.73	31.7	2.02	50	1.19
Portion 2	571.4	6.6	2.78	31.6	2.07	50	0.90
Portion 3	451.7	5.2	2.78	30.2	1.57	50	0.82

9.1.2 WATER SOURCE:

The water source are Lake Mzingazi, Lake Nsesi and the Goedertrou Dam. Water is purified and pumped to the bulk water system of Richards bay.

A connection onto the Municipal water distribution system will provide water to the development. Extensive use is being made of the existing pipelines and in particular the 800mm industrial supply main on the western boundary of the development. The initial design of the pipeline provided for an industrial flow of 795l/s and a fire flow of 94.6l/s. Our preliminary design figures does not exceed this flow figures.

9.1.3 BULK WATER STORAGE:

The reservoir cluster to be utilised is the Mandlazini reservoirs with a full supply level of 77.10m AMSL and an outlet level of 61.55m AMSL. The reservoirs has a storage capacity of 40 Ml.

The bulk water storage capacity required is based on the Annual Average Daily demand over a 48hour period. The volume required is calculated as follows:

Domestic water demand:	3,195Kl/day x 2 =	6,390 Kl
Fire water:	600l/m x 60 x 4 =	144Kl
	Storage volume required	= 6,404 Kl (6.5Ml)

The existing storage volume for the IDZ area is 40Ml. It is assumed for the purpose of the preliminary design that the existing reservoirs has sufficient capacity for Phase 1F of the IDZ.

9.1.4 LINK PIPELINE DESIGN:

As per paragraph 9.1.1 the required peak demand is in the order of 133l/s. The maximum average design flow velocity is considered to be 0.7m/s to 1.0m/s. The link main is however fed from two ends (the 800mm and the 200mm connections). The actual preliminary design maksimum velocity is 1.2 m/s (Domestic peak flow + 1 x fire hydrant) and this reduces along the pipeline as the flow reduces to approx. 0.5m/s.

The existing 200mm pipeline along Alumina Allee indicates high flow velocities during the combined peak domestic flow and fire flow scenario with velocities reaching 1.9m/s. Replacing this existing 200mm pipeline with a 300mmwill be good practice. It is therefore recommended that this proposal be evaluated during the final design stage.

The new link design requirement is 400mm uPVC pipe in class 9 (PN 9). The low flow velocity reduces the friction loss in the link pipeline system which increases the residual pressure in the system.

9.1.5 ERF CONNECTIONS:

The sizing of the erf connections in an industrial area are complicated due to the unknown demand of each individual stand. The connection will largely be determined by the rational fire design. It is expected that 150mm water connection will be the norm. Certain industries may require one or two fire hydrants while other may require an internal sprinkler system in combination with hydrants. Possible on site fire storage and pressure boosting may be provided by the developer/s. This will largely determine the sizing of the connections to each site.

9.1.6 STANDARD APPLICATION AND DESIGN PRINCIPLES:

The distribution of water to the stands in the development will be based on the following criteria:

Table 9.2: General water distribution design principles

Loss Factor	15% (1.15)
AASD Factor	1.5
Instantaneous peak factor	2.8
Maximum velocity in pipes (m/s)	1.5
Maximum static head (m)	120
Minimum head under instantaneous peak demand (m)	24
Maximum head under instantaneous peak demand (m)	90

Materials and construction:

- All materials to be SABS approved
- All pipe specials to be flanged steel pipe to SABS 1123, Table 1600/3
- All corrosion protection to be copon coated or similar approved
- All construction will be done in accordance with SABS 1200.

9.2. SANITATION:

A general layout of the sewage system is contained in *Annexure C* to this report.

The whole of the Phase 1F development can be drained under gravity. The larger Alton North township has been divided into three major catchments draining southwards towards the two 30m service corridors in the existing Alton Township. The crossing of Bauxite Bay. Carbonode Cell and the railway line between the two streets were done by means of pipe jacking of sleeves to house the sewers.

The design of the eastern and western collector sewers in the two corridors has a capacity of 60l/s and 30l/s respectively. This is the flow to be accommodated from Alton North Township. The spare capacities available can be utilised but is proposed that the capacities be verified for the purpose of the final design report.

Once the spare capacities are fully utilised, a new outfall sewer will have to be constructed.

Existing sewers to the south of Phase 1F will be utilised as connection of the internal sewers. The capacity of these sewers must be verified for the purpose of the final design. The design of these sewers has however included the inflow from the phase 1F development.

It must be emphasised that it is at this stage not known what type of industry will be established on the site. The following composition of industry is accepted for the purpose of the preliminary design of sanitation services:

- Dry industry (workshops, etc): 70%
- Dry industry (warehousing): 15%
- Wet industry 15%

This composition will be utilised to obtain an average flow from each sewer connection point on the development.

It must further be stated that the relief of the land is such that large diameter sewers will have to be designed at low gradients in order to achieve sewer flow. Various modifications to the sewer system may be required to achieve functionality such as flushing systems connected to the water mains, etc.

9.2.1 SEWERAGE GENERATED BY THE DEVELOPMENT:

The Sewerage generated by the developments is estimated as 90% of the AADD before losses. The composition of the sewer flow is assumed as indicated the preceding paragraphs. This is a theoretical figure in terms of zoning contributions and this figure could also be as low as 60%, depending on the composition of the development.

Dry and wet Industrial developments:

Table 9.3: Water demand calculation

Stand No.	*Area m2*	*FAR*	*Effective Area m^2*	*Water demand @ 225l/100m^2 (Kl AADD)*	*AWWF*	*PWWF*
16786	205696	0.6	123417.6	567.7	587.6	1469.0
16787	79654	0.6	47792.4	219.8	227.5	568.8
16788	164750	0.6	98850	454.7	470.6	1176.6
16789	167275	0.6	100365	461.7	477.8	1194.6
16817	56970	0.6	34182	157.2	162.7	406.9
16818	45514	0.6	27308.4	125.6	130.0	325.0
16819	24179	0.6	14507.4	66.7	69.1	172.7
16820	23375	0.6	14025	64.5	66.8	166.9
17442	23375	0.6	14025	64.5	66.8	166.9
17443	23375	0.6	14025	64.5	66.8	166.9
17455	27692	0.6	16615.2	76.4	79.1	197.8
17456	24377	0.6	14626.2	67.3	69.6	174.1
Portion 1	50858	0.6	30514.8	140.4	145.3	363.2
Portion 2	50005	0.6	30003	138.0	142.8	357.1
portion 3	39529	0.6	23717.4	109.1	112.9	282.3

Abbreviation legend:

AADD = Annual average daily demand, for the purpose of determining the annual water demand.

ADWF = Average dry weather flow. A function of the percentage discharge from AADD

PWWF = Peak wet weather flow. A function of the dry weather flow including storm water ingress and a peak factor of approx. 2.5

Sewer flow:

Annual average daily demand **(AADD)**	2,778 Kl/day
Reduction factor (90%)	
Average dry weather flow **(ADWF)**	2,500 Kl/day
Storm water ingress	10%
Average wet weather flow	2,750 Kl/day
Peak factor	2.5
Peak wet weather flow	6,875 Kl/day
With a 24 hour day withdrawal period	79.6 l/s (Design Flow) for internal water network as well as the gravity main from the reservoir.

9.2.2 DESIGN CONSIDERATION

Existing Municipal infrastructure capacity

The capacity of the existing sewers will be determined post the completion of a topographical survey. This information will be contained in the final submission.

9.2.3 INTERNAL SEWER NETWORK DESIGN CRITERIA

The following design criteria are applicable to the design of the internal sewer network for the proposed Phase 1F sewer reticulation.

a) Mains:

Minimum cover:

- In servitudes 600mm

- In sidewalks 1,400mm below final kerb level
- In road carriageways 1,400mm below final road level

Trenching and bedding SABS 1200LB

Bedding Drawing LB-2 (SABS 1200LB, p 10)

b) <u>Collector sewers:</u>

Flow formulae: Friction loss factor (n) = 0.012 (Manning)

Minimum size of sewers: 100mm dia.

The minimum sewer will be 160mm PVC, 400kPa structured wall pipe.

Table 9.4: Limiting Gradients:

Minimum sewer gradients	
Sewer diameter (mm)	Minimum gradients (mm)
100	1 : 120
150	1 : 200
200	1 : 300
225	1 : 350
250	1 : 400
300	1 : 500

Velocities: Minimum (full bore): 0.7m/s

c) Manholes:

Position At all junctions, at all changes of grade

At all changes of direction

Maximum interval 150m

Minimum interval Determined by natural slope

Minimum manhole sizes:

Table 9.5: Minimum Gradients

Minimum internal dimensions of manhole chambers and shafts		
SHAPE	CHAMBER	SHAFT
Circular	1000mm	750mm
Rectangular	910mm	610mm

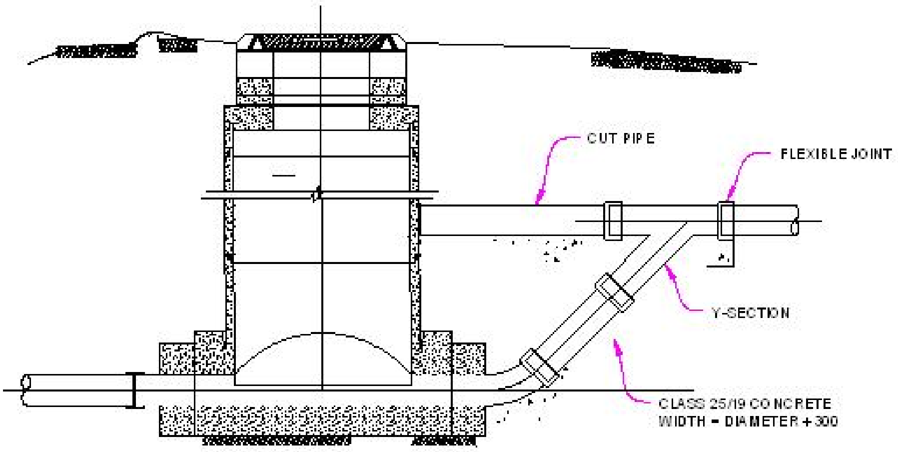
Slopes of benching in manholes:

Maximum 1 : 5

Minimum 1 : 25

Design requirements: SABS 1200LD

Steep drops:



Typical drop-section manhole:

Initial low velocities: Precautionary measures required

It is common practice to design sewer systems with slopes which will produce specified minimum velocities at maximum dry weather flows. These minimum velocities are such as to ensure self-cleansing of the sewer.

It must nevertheless be noted that the flow in the pipes, and consequently the velocities, will only build up to the design figures over a period of time, which is dependent on the rate of development of the township.

Certain measures do therefore become necessary in cases of large townships, such as this one, which by virtue of their size do generate considerable flows, but this only materializes at the end of the development. We consider that the design of flushing tanks or closing syphon's, is not warranted in this instance. However, it is essential that a regular inspection and maintenance programme be instituted for the safe, economical operation of the reticulation in the proposed township.

It is expected that occasional flushing of some of the lines will be required in the initial phases.

Erf Connections

Connections to the erven have been provided so that 100% of the erf area is drained at a nominal slope of 1:100 across the diagonal, with pipe cover of 500mm at the top end of the erf.
At least one 150mm diameter connection has been provided at each stand.

Construction

The general construction principles of SABS 1200 LD are endorsed and have been specified. Tolerances for pipe laying will however be more stringent than SABS 1200 LD and the use of a laser beam for pipe laying in the documentation will be made compulsory.

Except under circumstances contained in the following paragraph, the following are the minimum allowable values of the cover to the outside of the barrel of the pipe for sewers other than connecting sewers:

(a) In servitudes	600mm
(b) In Sidewalks	1.4 meters below final kerb level
(c) In road carriageways	1.4 meters below final constructed road level

Where the depth of cover in roads or sidewalks is less than 500mm or where the depth of cover in servitudes is less than 300mm, the pipe should be protected from damage by means of:

(a) The placement of cast-in-situ or precast concrete slab(s) over the pipe, isolated from the pipe crown by a soil cushion of 100mm minimum thickness. The protecting slab(s) to be wide enough to prevent excessive superimposed loads being transferred directly to the pipes.

(b) The use of structurally stronger pipes able to withstand superimposed loads at the depth concerned.

(c) The placement of additional earth filling over the existing ground level in isolated cases where this is possible.

The encasement of pipes in concrete is not recommended. Where encasement had to be employed these were made discontinuous at pipe joints so as to maintain joint flexibility.

Manholes are of precast concrete manufactured with high alumina cement. All in-situ concrete in the manhole that comes into contact with sewage will likewise have to be manufactured with high alumina cement.

Maximum spacing of manholes is 150m.

9.3 STORM WATER SERVICES

An urban stormwater network is made up of several types of structures that all interact and form an integral part of any urban development. The main function of a urban stormwater network is to remove runoff generated by the urban area as effectively as possibly to a natural stream or area from where the runoff would flow away naturally without disrupting or damaging any person or property within the urban and outlying natural area, whilst protecting the natural stream or area where to the runoff is discharged simultaneously from erosion or so called stormwater pollution. The misinterpretation or negligence of persons designing, installing or approving such urban stormwater networks can have devastating effects on property, the environment and human life.

The urban stormwater network is deemed to cover any of the following:

- Any structures that
  - Allow the inflow of stormwater runoff into a stormwater pipe;
  - Form the junction between more than 1 pipe;
  - Allow a stormwater pipeline to change size, gradient or direction;
  - Allow the outflow of stormwater runoff into a natural stream or area;
  - Any combination of the above.
- Any pipe of which the main function is to transport stormwater runoff;
- Any gravel or lined canal of which the main function is to transport stormwater runoff;
- Any gravel or concrete dam wall of which the main function is to attenuate stormwater runoff;
- Any kerb and road surface that contain water and transport runoff towards an inlet structure.

Storm water pipes control the flow path of storm water. The flow path of surface run-off as well as the characteristics of storm water flow is altered when surface runoff is allowed to enter a stormwater pipe. This is ideal for the prevention of the possible damages and effects by ponding or ingress of surface water.

The piping of storm water concentrates the water and therefore increases the instantaneous peak generated by a sub-catchment. Water usually arrives at a discharge point more vigorously than when allowed to flow freely.

Discharge points therefore have to be protected by means of energy dissipating structures and stilling basins to allow the flow to normalize to a laminar and non-turbulent flow condition. The reduction in flow velocity is very important in order to allow flow to be controlled. Uncontrolled storm water flow is a dangerous situation.

The purpose of storm water infrastructure is to manage overland storm water run-off during rainstorm events in order to protect the development. The design of proper storm water management is based on:

- The need to protect the health, welfare and safety of the public, and to protect property from flood hazards by safely routing and discharging storm water from developments
- The quest to improve the quality of life of affected communities
- The opportunity to conserve water and make it available to the public for beneficial uses
- The responsibility to preserve the natural environment
- The need to strive for a sustainable environment while pursuing economic development, and
- The desire to provide the optimum methods of controlling run-off

The area is very flat, making drainage challenging. There is however drainage lines intersecting the proposed access road. The stormwater outlets drain into these drainage lines, but deep outlet structures are however foreseen due to the flat ground slope. The flat ground however ensures that relatively low flow velocities can be expected at the outlets therefore minimising the amount of erosion that can be expected. Adequate erosion protection should however be provided to protective the in-situ sandy material from being washed out. The development is located near the coastline and sandy materials are found throughout the area.

The site slopes predominantly towards the south west.

9.3.1 DESIGN PHILOSOPHY

The development area is very flat with a gentle gradient falling towards the southern perimeter of the site. Storm water drainage on all the sites is considered to be a function of the development to be undertaken in future. The management of stormwater on each site may either be a closed pipe system or a surface flow system. It is therefore envisaged that each stand will receive a stormwater connection point.

Individual connections to erven

Previously, stands were allowed to discharge run-off across the sidewalk into the street and then collected at kerb inlets.

The alternative to this, especially where run-off was being concentrated, was to enforce developers to connect to the street/road piped system. This had the disadvantage that existing services had to be crossed on the sidewalks and connections points had to be broken in to the existing system.
An alternative system in which all erven will be provided with stormwater connections was investigated and found to be feasible. Where stands drain naturally towards the midblock boundary, connections provided as midblock open drains were not favoured due to possible maintenance problems and difficulties to control these.

A further advantage to individual connections lies in the design provisions.

The major part of the Township is very flat and sizes, slopes and velocities in gravity services are critical with connection points known, the design can be more specific.

The disadvantages, although outweighed by the advantages are as follows:

i) The design is prescriptive to developers. In each particular erf, internal stormwater reticulation will now have to drain to a pre-specified point, whereas a more economical, to the developer, alternative might have been possible.
ii) The connections will be sized on assumptions as to the development with the applicable run-off. Should erven however be developed to a lesser concentration, these could be slightly oversized.
iii) Initial capital investment is higher. The effect is however very small in comparison to later construction and the possible disruption of services.

Erven which naturally drain in a direction away from the street were mostly given a connection on the railway side. Where no railway facility is provided, a connection on the street side of the erf at a greater depth or alternatively a midblock drain was provided

9.3.2 DESIGN RUN-OFF CALCULATIONS

The management of storm water for the development was based on typical rain storm events for the area. The generation of a point rainfall maximum and the calculation of a design run-off was based on the guidelines as set in the planning and design manual for residential development.

Table 9.6: Flood frequencies – major system

Design flood frequencies for major systems	
Land use	Design flood recurrence interval
Residential	50 years
Institutional	50 years
General commercial	50 years
CBD	50 – 100 years

The hydraulic design of the development's storm water drainage in terms of a major system requirements is based on the following criteria:

- A minimum pipe size of 450 ø is the norm for sub-surface (pipe flow) drainage. The capacity of this system exceeds the actual 1:5 year storm event.
- The 1:50 year major storm analysis shows that the system can also accommodate the flood due to the minimum requirements.
- The available storage on the road surface, combined with the storm water system capacity is considered suitable to accommodate the 1:50 year rain storm event.

Table 9.7: Flood frequencies – minor system

Design flood frequencies for minor systems	
Land use	Design flood recurrence interval
Residential	1 - 5 years
Institutional	2-5 years
General commercial	5 years
CBD	5 – 10 years

The *Richard's Bay IDZ* is considered a mixed land use; the **5** and **50** year flood recurrence intervals should therefore be used to design the minor and major systems respectively.

The **time of concentration** for overland flow was calculated using the *Bransby-Williams formula*. For defined watercourse flow the *Empirical formula* developed by the US Soil Conservation service was used.

The Bransby-Williams formula is used to calculate the time of concentration for overland flow:

$$T_c = 0.604 \times \left[\frac{r.L}{\sqrt{s}} \right]^{0.467}$$

Where:

L = The length of the flowpath (m);

S = The slope of the waterpath (m/m);

T_c = The time of concentration (hrs);

r = Roughness coefficient, read from table 2 below:

Type of ground cover	**'r' Value**
Clean soil	0.10
Paved area	0.02
Sparse grass	0.30
Moderate grass	0.40
Thick bush	0.80

Table 1: Type of ground covers

The time of concentration for defined water course flow was calculated using the Empirical formula developed by the US Soil Conservation service:

$$T_c = \left(\frac{0.87L^2}{1000S_{AV}} \right)^{0.385}$$

Where:

L = The length of the flowpath (m);

S_{AV} = The average slope of the waterpath (m/m);

T_c = The time of concentration (hrs).

Overland flow concentrates into a defined watercourse over a minimum distance of approximately 150m, therefore where the length of the longest watercourse in a catchment exceeds 150m; concentrated (defined watercourse) flow was assumed. This reduces the time of concentration considerably.

The **runoff** was calculated using the rational method runoff calculation described in the “Road Drainage Manual” South African Roads National Roads Agency Limited”, 5th edition page 3-53 to 3-59. The “Utility Programs for Drainage” program developed by Sinotech CC as well as the design software “Civil Designer” compliments the mentioned Road Drainage Manual and was used to calculate the runoff.

9.3.3 <u>HYDROLOGY</u>

The study area falls within an average intensity, summer rainfall region, see figure 12 below:

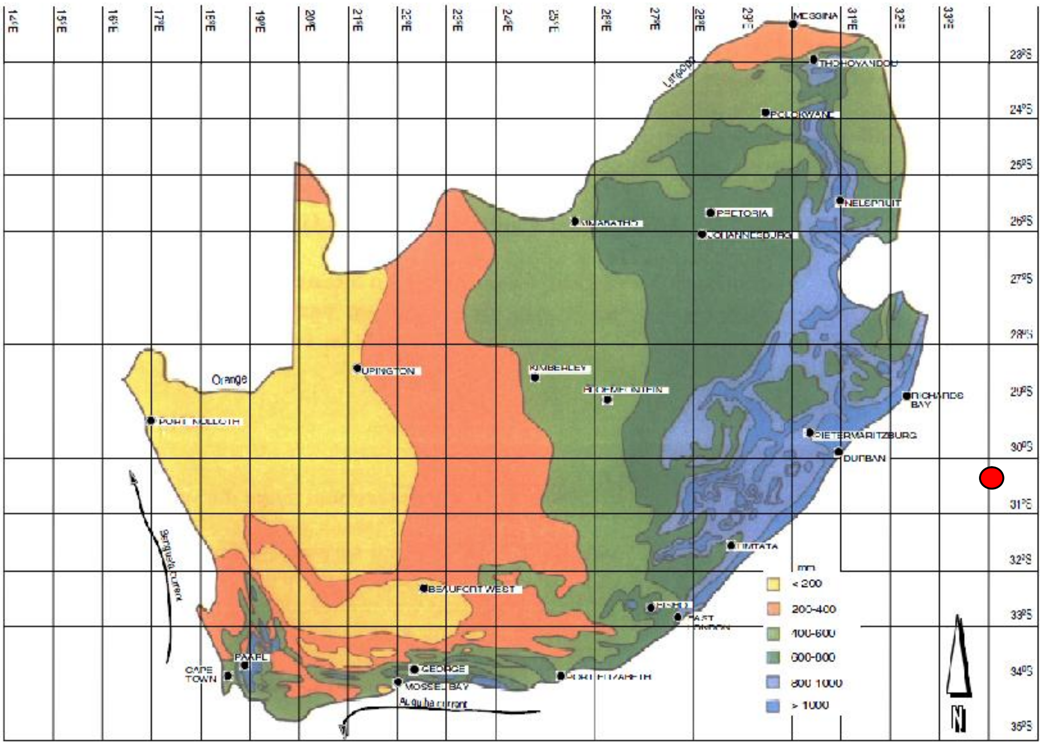


Figure 9.1: MAP of South Africa

The design rainfall is based on the following rainfall station information:

Station name	SAWS number	Record length (Years)	Latitude	Longitude	MAP (mm)
Richards Bay Municipality	0305167_W	53	28°47'	32°05'	1255

The 24 hour storm **point rainfall** depths extracted from the rainfall station data is as follows:

Return period (Years)	1 in 2	1 in 5	1 in 10	1 in 20	1 in 50	1 in 100
Rainfall depth (mm)	113.4	176.9	231.2	294.9	398.3	494.8

9.3.4 BRIEF DESCRIPTION OF THE DRAINAGE SCHEME

Stormwater drainage

The proposed layout of the drainage scheme is shown on the attached layout in ***Annexure D***. The total length of stormwater pipework is approximately 2800 m. The pipe sizes vary between 450mm Ø and 1800mm Ø.

The bulk of the stormwater will discharge into two drainage systems, one located in the western quarter of the site and the second in the eastern quarter of the site. A watershed through the middle of the site divides the two systems. The both systems discharge into stormwater servitudes placed at the drainage lines.

The stormwater system is designed to handle the runoff generated by the developing stands only in the 1 in 5 year recurrence interval storm event. The total catchment area of the developing sites is approximately 120 hectare.

Subsoil drainage

The water table on site is generally very high and construction of an extensive and effective sub-soil system is encouraged. This is of utmost importance specifically for the protection of the structural layers in the roads.

Subsoil drains will generally be placed on both sides of the roads at a depth of approximately 1.5m.

In the areas where ground water conditions warrant additional drains, these will be provided underneath the road layer works and even in the bedding of the pipework. The subsoil drainage system is not indicated on the layout as it will be determined on site based on the conditions.

9.3.5 DESIGN CONSIDERATIONS

Individual connection to erven

The erven are relatively large industrial zoned erven. Each erf has been provided with a connection point. These connection points will be constructed as field inlet structures for the interim period, before the erven are developed. In the pre-development phase gravel berms or stormwater gulleys are proposed on the boundary of each erf to collect overland runoff from the erf and to discharge into the connection points. In post-development phase the field inlet structures can be demolished and replaced with either kerb inlet or junction box structures (Whatever suits the need of the developer). The erf future internal stormwater scheme will therefore connect onto the connection points provided at this stage.

Inlets

Stormwater run-off has to enter the pipe network through an inlet structure. There are several types of inlet structures that can be utilized in this regard depending on the way the stormwater has to enter. Typical inlet structures include the following:

- Field inlet structures

  These are structures with relatively large concrete apron slabs placed in natural areas to allow stormwater runoff to flow into the structure from all four sides. The inlets orifices are relatively large to ensure larger material (Rubbish, leaves, branches etc.) do not block the entrances and is actually allowed to drop into the inlet structure. The junction box portion below the apron slab is also relatively large to ensure the majority of the material is trapped in this box without blocking the outflow from this box. The cover slab is held in place by four short stub columns, but can easily be removed for maintenance purposes. Maintaining these structures is very important. They have adequate capacity to store debris, but they should be cleared regularly to prevent the structure from blocking. See details below:

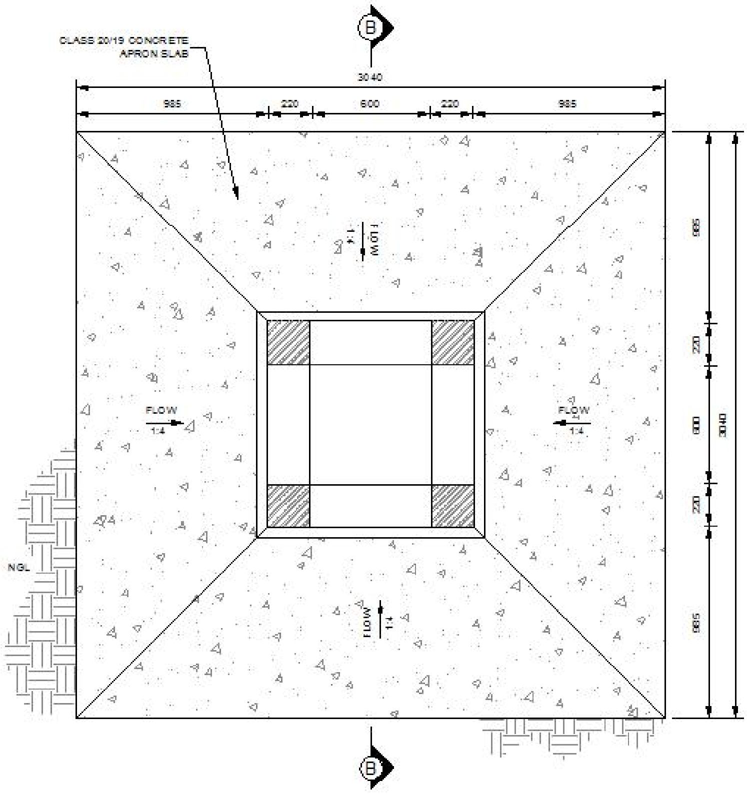


Figure 9.2: Field inlet structure - Plan view

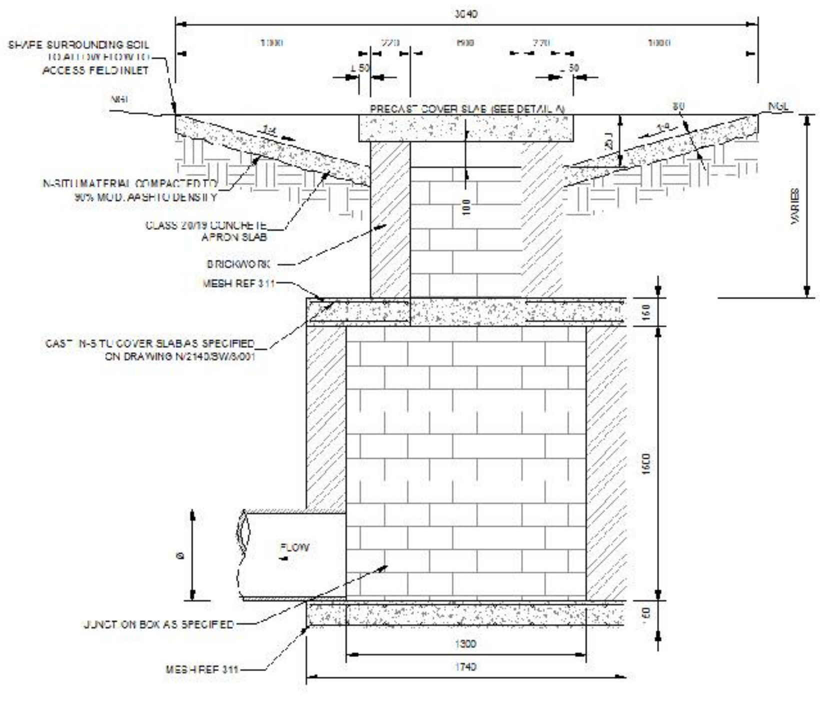


Figure 9.3: Field inlet structure - Sectional view

- <u>Kerb inlet structures</u>

  Kerb inlet structures are placed next to roads to remove runoff from the roadway. They are normally recessed from the kerb line. The kerb line also assists with the conveying of the stormwater runoff towards the inlet portion of the kerb inlet structure. The inlet orifice of the kerb inlets are located on only one side of the structure and are relatively large to ensure the orifice has adequate capacity (Preventing runoff ponding in the roadway). The large orifices also ensure that rubble or debris transported by the runoff passes through the inlet portion, therefore not blocking the inlet. The structures should also be maintained regularly to remove rubble and debris that were allowed to pass into the structure. See details below:

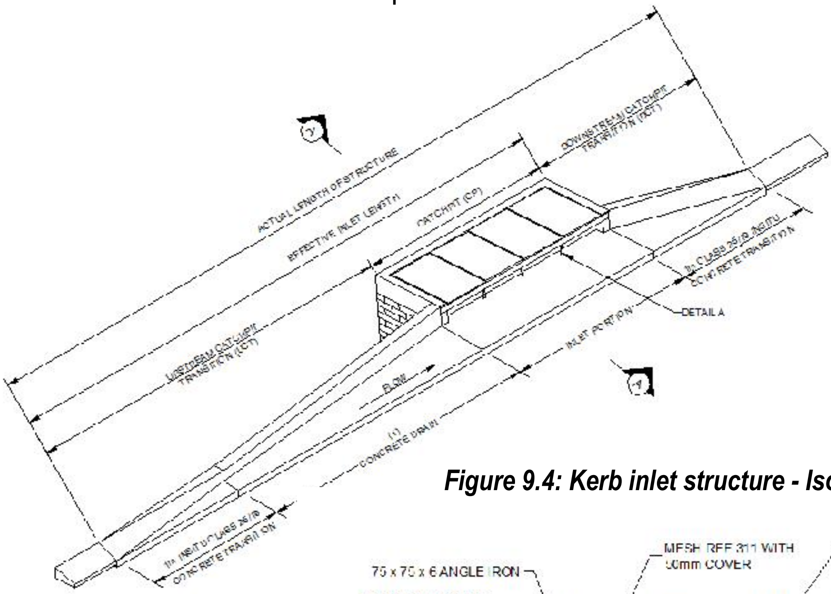


Figure 9.4: Kerb inlet structure - Isometric view

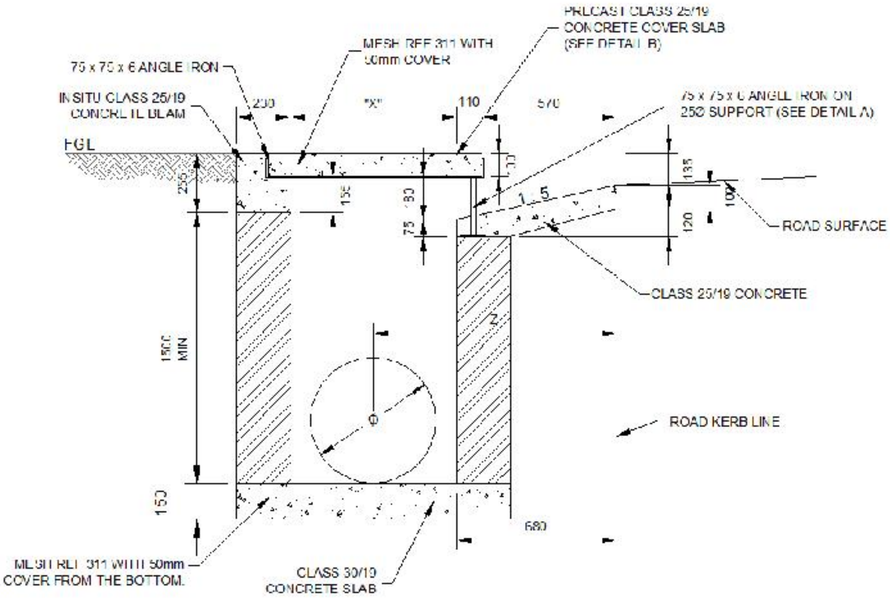


Figure 9.5: Kerb inlet structure - Sectional view

Kerb inlets at strategic positions will drain water from the road surface into the nominal storm water system.

Kerb inlets are designed according to the Guidelines as stated in the Road Drainage Manual. The design of a kerb inlet is particular to the following aspects:

- Location of the kerb inlet
- Length of the transition from kerb to inlet opening
- The length of the actual inlet opening
- The above perimeters are determined by the following:
- The rain storm intensity
- The slope of the particular road section upstream of the kerb inlet
- The length between kerb inlets
- The kerb characteristics of the road in terms of overtopping capacity
- The shape of the road in cross section (cross fall or camber)

Outlets

The two outlets proposed in the stormwater servitudes will consequently be relatively deep (Approximately 3.5 to 4m deep) due to the flat terrain and the connection onto the existing drainage canals will have to be reviewed in the detail design stage.

Due to relatively high flow rates and velocities of runoff being discharged by the stormwater infrastructure at the outlet structures it is very important that adequate structural designs as well as erosion protection be implemented to prevent erosion and/or damage to property. Energy dissipation structures are therefore paramount in all critical areas. Chief among these are outlets that discharge into natural streams and rivers where wetlands and sensitive ecological areas exist. These natural areas should not be disrupted by uncontrolled stormwater discharges. The primary aim of downstream erosion protection is to release water into the natural channel at a velocity no greater than the original velocity and in the same direction. See examples of dissipation structures below:

Typical headwall outlet structures recommended for relatively lower flow rates are as shown below:

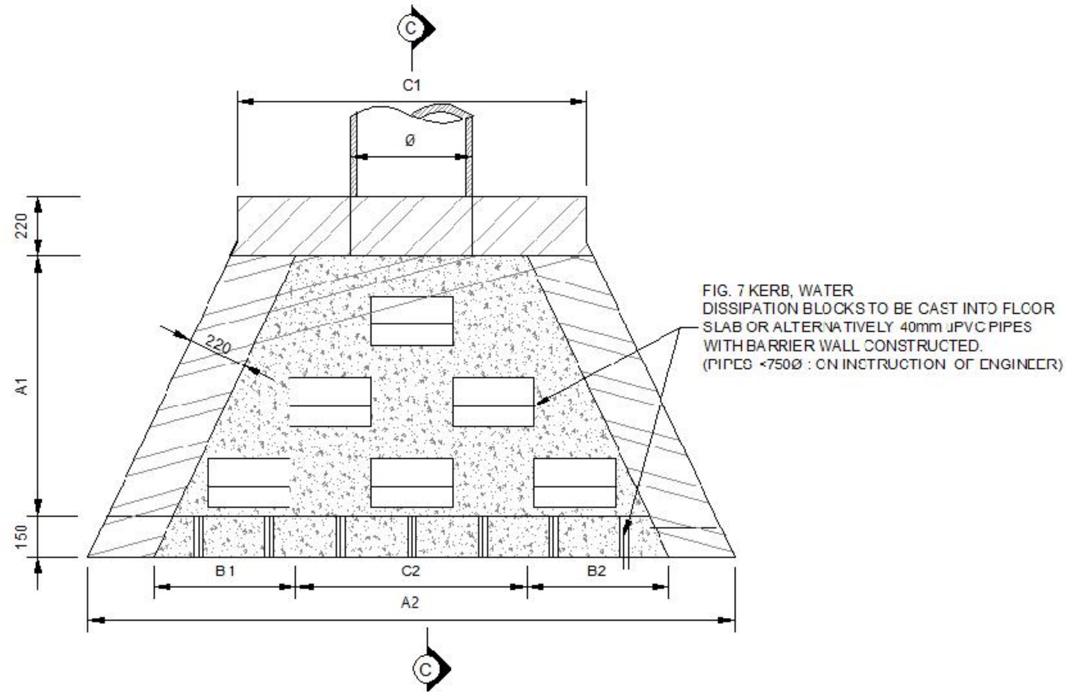


Figure 9.6: Typical headwall structure - Plan view

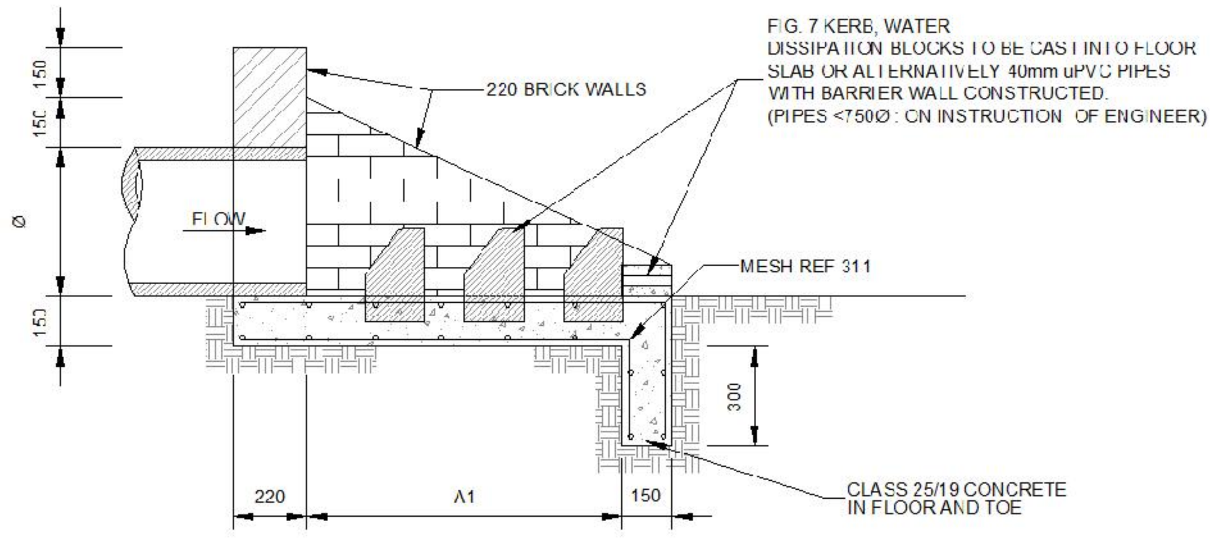
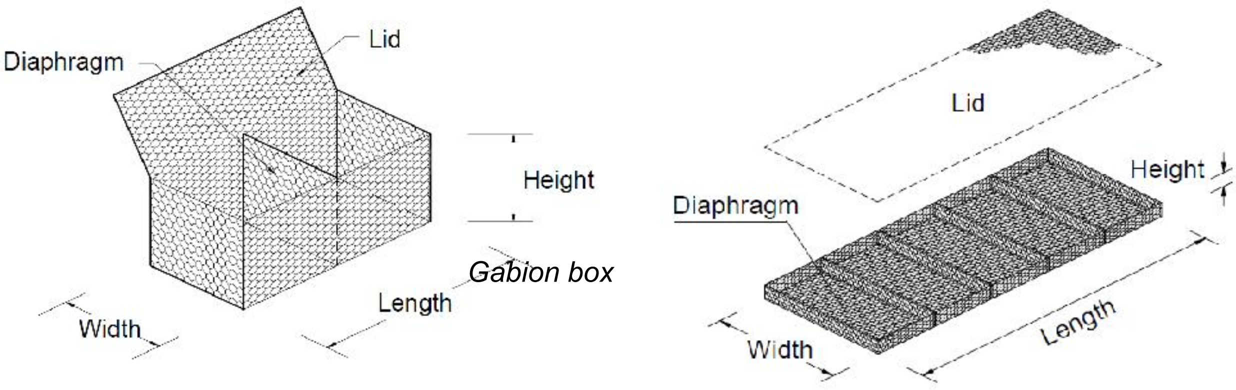


Figure 9.7: Typical headwall structure - Sectional view

Erosion is generally found where:

- Water velocities are high and;
- The direction of flow changes rapidly.

Erosion is also prevented by installing reno mattresses and gabion boxes in the transition zone from the outlet structures to natural ground. The reno mattresses and gabion boxes are hexagonal woven wire mesh filled with rock to form flexible, permeable, monolithic structures. See figures below:



Reno mattress

Figure 9.8: Gabions and reno matresses

<u>Pipe slopes</u>

The suggested minimum grades for pipes are prescribed in the “Guidelines for human settlement planning and design” and are tabled below:

Table 9.8: Minimum Gradients

SUGGESTED MINIMUM GRADES FOR PIPES		
Pipe diameter (mm)	Desirable minimum gradient (1 in …..)	Absolute minimum gradient (1 in …..)
300	80	230
375	110	300
450	**140**	**400**
525	170	500
600	**200**	**600**
675	240	700
750	280	800
825	320	900
900	**350**	**1000**
1 050	440	1 250
1 200	520	1 500

Pipe Construction

The precast culvert bedding will predominantly be the standard type B bedding, see detail below:

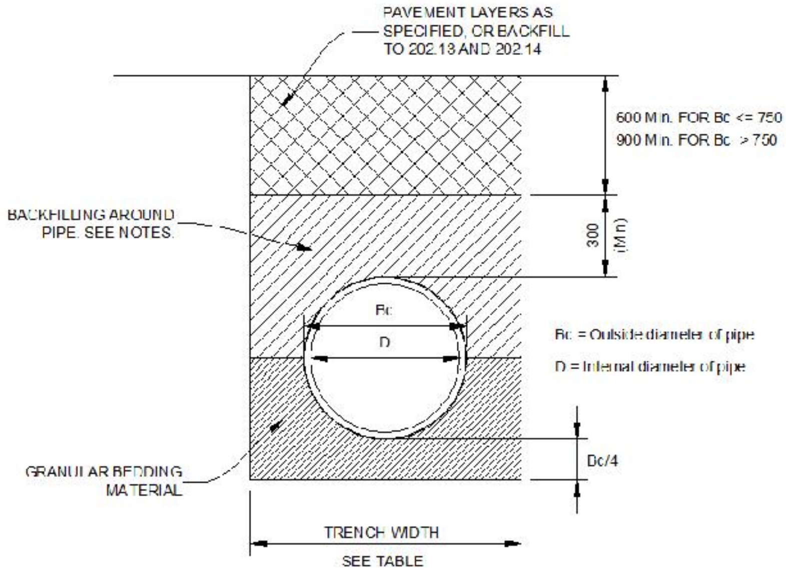


Figure 9.9: Typical pipe bedding

9.4 ROADS INFRASTRUCTURE

9.4.1 DESIGN CONSIDERATIONS

The document *"Guidelines for Human Settlement Planning and Design"* was used as basis of the design standards adopted in this project in conjunction with the following design documentation / guidelines:

- TRH 4: Structural Design of Flexible Pavements for Interurban and Rural Roads.
- DRAFT UTG7: Geometric Design of Urban Local Residential Streets.
- TRH 3: Surfacing Seals for Rural and Urban Roads.

From the *"Guidelines for Human Settlement Planning and Design"* the following street characteristics were adopted:

TABLE 9.9: PROPOSED DESIGN STANDARDS
(based on Table 8.1: Typical street characteristics *"Guidelines for Human Settlement Planning and Design")*

STREET / ROAD CATEGORY:	**UA**	**UB**	**UC**	**UD**
	ARTERIAL STREETS		ACCESS STREETS	
Description & function	Vehicles only	Higher order mixed pedestrian & vehicle	Lower order mixed pedestrian & vehicle	Pedestrian & vehicle access
Level of Service (LOS)	High	Moderate	Moderate to Low	Low
Traffic (vehicles / day)	>600	<600 (paved) & <350 (unpaved)	>75	<75 <5 heavy vehicles
Traffic (no of E80's)				
a) If street carries construction vehicles	1 – 50 x10^6 E80's/lane	0,3 – 3 x10^6 E80's/lane	<0,3 x10^6 E80's/lane	
b) If street does not carry construction vehicles	1 – 50 x10^6 E80's/lane	0,03 x10^6 E80's/lane	<0,03 x10^6 E80's/lane	
Pavement Class & Design Bearing Capacity (indicative only)	ES3 to ES30	ES1 to ES3	ES0,1 to ES0,3	
Standard	Most likely paved	Paved / unpaved	Most likely unpaved or paved for reasons other than traffic	Paved for reasons other than traffic
Pedestrian traffic	None	Very little, controlled	High, controlled	High, uncontrolled

9.4.2 LAYOUT, ZONING AND COVERAGE

A Layout plan of the proposed development is attached under Annexure D.

The following assumptions were made and incorporated in the design approach.

- Erven 16817 & 16818 is excluded on the assumption that they are serviced directly of Alumnia Allee.
- A GLA of 711500m².
- Road servitude of 5,086 hectares.
- Trip generation based on the abovementioned information is 2135 trips (1707 in/427 out).

9.4.3 GEOTECHNICAL ASSESSMENT

Refer to Chapter 6.2.

9.4.4 TRAFFIC IMPACT ANALYSIS

There is no Traffic Impact Analysis for this area that includes this development, therefore a Traffic Impact Study (TIS) needs to be done as soon as possible, as to determine the number of incoming and outgoing lanes into the development. The TIS will also provide the upgrades required (if any) at the intersection into the development as well as other possible upgrades upstream from the development.

A preliminary desktop analysis indicates that the access road should have two lanes in, and two lanes out of the development. This will be finalised once the TIS is completed.

9.4.5 DESIGN STANDARDS

Design standards utilized in the preliminary road design are in essence extracted from the following:

- Guidelines for human Settlement Planning and Design;
- TRH 14: Guidelines for Road Construction Materials, 1985;
- TRH 15: Guidelines for Sub-Surface drainage for roads, 1994;

- TRH 4: Structural Design of Flexible Pavements for Inter Urban and Rural Roads, 1996.

9.4.6 DESIGN PHILOSOPHY

The new street will have a singular access point directly from Alumina Allee Street.

The new street will provide access to each stand bordering the street, and is therefore designed as a road used for industrial purposes.

The roadway will also form the basis of the stormwater disposal system for the "minor storm" (see discussion on stormwater in this report), with kerb inlets placed strategically for optimum stormwater drainage.

All stands bordering the road will have field inlets that also discharge into the stormwater system which ultimately discharges into the two main stormwater canals crossing the development.

9.4.7 DESIGN PARAMETERS FOR ROAD WAYS

New road:

- Classification :Industrial road;
- Roadway width :15.8m (4 x 3.7m lanes & 2 x 0.5m shoulders);
- Pavement class : ES 10 (3 – 10 x 10^6 80kN axles/lane);
- Horizontal alignment : The horizontal alignment is fixed, but the intersection layout has the possibility of changing once the TIA is complete;
- Vertical alignment : A vertical alignment has not been finalised yet, due to shortage of various critical elements on the existing survey. (detail design can be done once a complete survey have been received);
- Road edging : Fig 3 kerb-channel combination both sides;
- Parking : 2.5m wide parallel parking both sides.

9.4.8 PROPOSED INFRASTRUCTURE

Roadways

Road works layouts and details are contained in ***Annexure E*** to the report. The roadway will be constructed in accordance with the requirements of the local authority, being for a 15.8m roadway (4 x 3,7m lanes + 2 x 0,5m shoulders with parking on either side of 2,5m wide).

Thus a 20,8m wide surfaced area if parking is included. A Kerb-channel combination will be installed on both sides.The pavement structure will be designed for an “Industrial road” with a pavement class and design bearing capacity of ES10.

The layerworks will consist of the following:

- Surfacing : 30mm Asphalt
- Base : 150mm G1 Base (commercially obtained)
- Upper Subbase : 150mm C4 (imported from commercial sources)
- Lower Subbase : 150mm C4 (imported from commercial sources)
- Upper Selected : 150mm G7 (imported from commercial sources)
- Lower Selected : 150mm G9 (possibly obtainable on site, to be verified by Geotechnical investigation)
- Roadbed : Rip and recompact insitu to 90%

The layout of the roadway is indicated on the layout drawing attached under Annexure A.
A typical cross section is indicated below:

9.4.9 PEDESTRIAN ACCESS

A Pedestrian walkway of 1,2m wide will be provided along the entire length of the road on both sides.

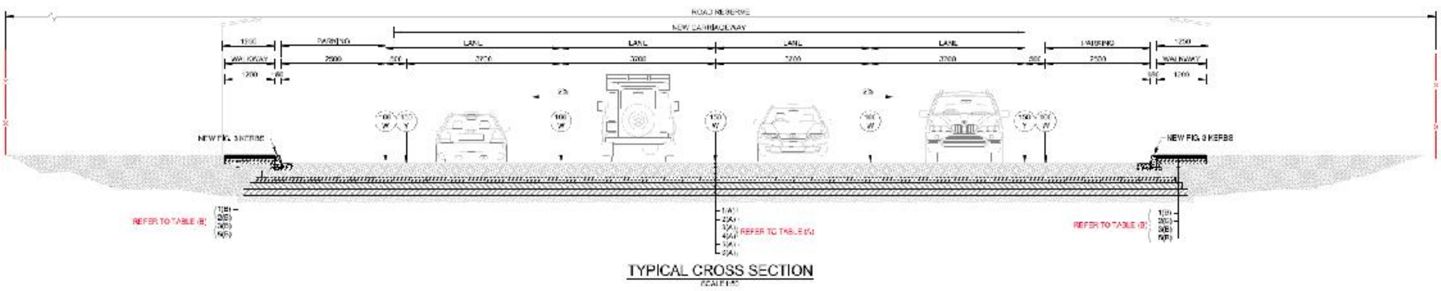


Figure 9.10: Road cross section

9.4.10 TAXI AND BUS LAYBYES

Two taxi and bus laybyes will be provided of 27m long excluding tapers. These laybyes will be situated approximately in the middle of the street at chainage 750.

The location of all services to be installed within the road servitude have been indicated on the section below.

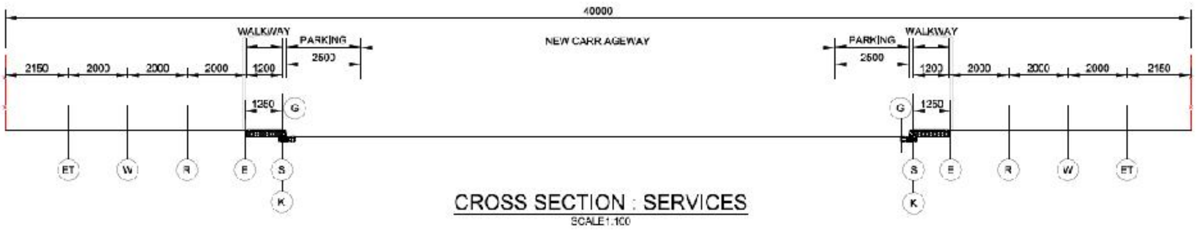


Figure 9.11: Typical road width and shoulders

LEGEND:

- ET ELECTRICAL & G.P.O
- W WATER
- R SEWER
- S STORMWATER
- E STREETLIGHTS & SUBSOIL
- K KERBING
- G GUTTER

9.4.11 PROPOSED SEAL

The seal of a road / street is the final lasting impression of the quality and workmanship of a road. It is thus important to ensure that quality and workmanship on the seal layer is not compromised. The “look” of a township and aesthetics of the streetscape is all dependant on the visual effect. The only visible parts of civil infrastructure within a township are the surfacing and kerbing of the roads.

In determining a seal for the roads the following considerations played a major role:

- Quality of base and ability of contractor – an uneven base would reflect poorly if surfaced using single / double seals versus the traditional good image reflected by asphalt seals.

- Construction traffic of housing infrastructure after completion of seal. Construction traffic is associated with physical damage, wheel turning movements, spillages, etc, to which an asphalt seal performs well, versus a thin layered seal.
- Establishment of different paving units (asphalt unit / chip and spray units) are expensive, thus one tries to only accommodate one type of surfacing team.
- Ability of contractor to construct a final base to strict criteria – a roughly finished base will show surface irregularities through chip and spray seal. Asphalt will not show irregularities
- Life span of a seal – a single / double seal has a much shorter life cycle than an asphalt seal
- Ability of local authority to maintain – it is a well known fact that very few municipalities have proper maintenance structures in place. Thus the less maintenance required on a surfacing seal would play a large role on the seal selection. Asphalt requires much less maintenance.
- Turning movements at cul-de-sacs and intersections, as well as pulling away movements at intersections, prone thin chip and spray seals to be damaged, especially in early life of the seal
- Gradients – on steep gradients chip and spray seals are not recommended asphalt seals are recommended for slopes steeper than 12%.

It is proposed to utilise an asphalt seal of thickness 30mm on all roadways. An asphalt layer of less than 30 mm is not recommended due to:

- Optimum compaction not being reached as the asphalt layer cools down too quickly (compaction temperature to be above ±70°C). Compaction window time is critical.
- Due to the long haulage of the asphalt from the manufacturing plants the delivery temperature will be well below the manufacturing temperature.
- Heat transfer from the mat to the base is the greatest cause of heat loss. The thinner the mat the greater the loss.
- One can expect density failures on a mat with thickness ≤25mm - *SARF course on Compaction of Building Materials*.

9.5 RAILWAY SERVICES

This chapter focus on the findings from a preliminary investigation on options to supply railway siding services to the new development of Phase 1F. The components of the investigation and the various aspects to consider is briefly summarised as follows:

- Bulk Earthworks
    - Optimising cut and fill requirements
- Storm water Drainage
    - Shaping of earthworks to allow drainage under track
    - Shaping of loading platforms (Excluded. Assumed to be for the owners account upon sale of the industrial stand.)
    - Storm water drainage pipes and culverts
- Railway Track
    - Formation
    - Turnouts
    - Rails
    - Fishplates
    - Rail-to-Sleeper Fastenings
    - Stop blocks
    - Derail Devices
    - Insulated Rail Joints (Block Joints) and Rail Bonds.
    - Ballast

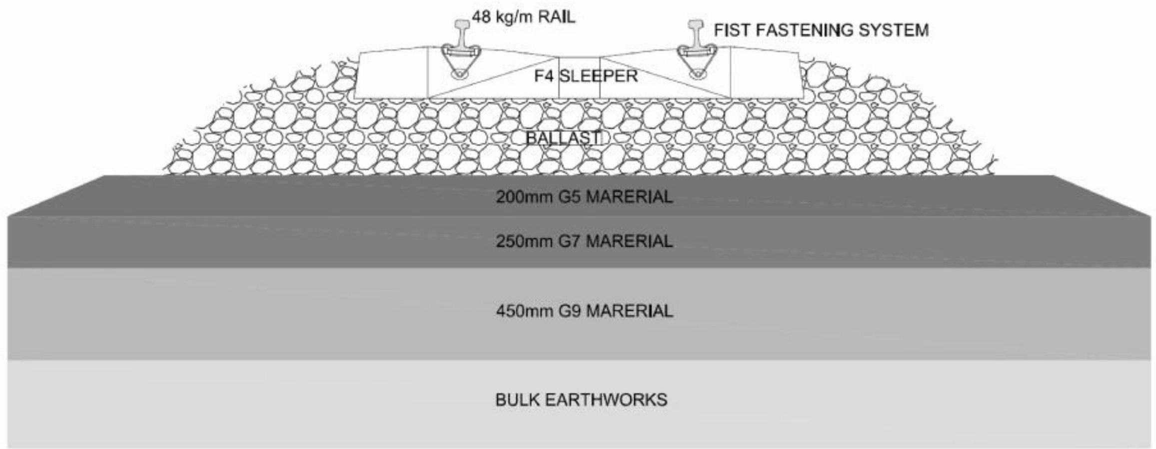


Figure 9.12: Track Components

The components where possible was assumed to be second hand because of the cost effectiveness thereof and the long lead times which could be as much as 12 to 24 months.

9.5.1 DESIGN CONSIDERATIONS

Bull earthworks

Bulk earthworks estimates were done for the two options investigated. We used market related rates to determine the expected costs in order to compare the options.

The following were assumed to calculate the quantities:

Cut and Fill slopes:	1:1.5
Top Soil stripping:	0.2m
Compaction:	90% Mod AASHTO

The above is based on the assumption that the cut material is suitable to be used for the construction of the fill embankments. (According to Specification for Railway Earthworks S410) This must be confirmed through a geotechnical investigation.

Drainage

Storm water drainage estimates was done to obtain a budget cost for each option. The water runoff will be diverted through culverts. The following were assumed to calculate the quantities:
Slope under track: 2%

The disposal of the water must be integrated into the Environmental Plan and there may be additional requirements.

Track Formation

The formation design will be according to Transnet's Specification for Railway Earthworks S410.

Top 200mm layer:	G5 Material
Min Grading Modulus:	1.8
Plasticity index:	3 – 10
Minimum Compaction:	95% Modified AASHTO

Second layer 250mm:	G7 Material
Min Grading Modulus:	1.0
Plasticity index:	< 12
Minimum Compaction:	95% Modified AASHTO
Third layer 450mm:	G9 Material
Min Grading Modulus:	0.5
Plasticity index:	< 17
Minimum Compaction:	93% Modified AASHTO
Bulk Earthworks:	G10 Material
Plasticity index:	< 25
Minimum Compaction:	90% Modified AASHTO

The availability of the above mentioned materials must be confirmed through a geotechnical investigation. We assumed that the G9 and G10 layer material will be available from the cutting material. All materials will be imported.

Turnouts

Second hand 1:9 rail bound turnouts and hand tumblers shall be supplied on wooden sleepers due to the cost effectiveness thereof and the long lead times to order new turnouts and wooden sleepers. New hand tumbler rods shall be supplied.

Turnouts shall be supplied complete with crossing bearers of steel or wood and the appropriate rail - to - bearer fastenings.

The supply of turnouts shall also include stocks and guards, stocks and switches, soleplates, closure rails, lead rails, internal fishplates, fish bolts and nuts, connecting rods, pull rods, steel plate foot guards, switch boxes, and all other fittings, fastenings, and accessories and, where applicable, notched soleplates and gauge plates.

Rails

Second hand 48kg/m rails shall be supplied due to the cost effectiveness thereof and the long lead times to order new rails.

Rails shall be of Class B or better and of standard section. They shall be sound, free from flaws, cracks, splits, and corrosion, and shall have at least one good running edge.

- Rails shall be at least 6 m in length and shall preferably be of nominal length 12 m or 18 m.
- Rails shall be at least 3 m from fish plated or welded joints.
- Rail ends shall not be battered in excess of 2 mm
- A length of at least 300 mm shall be cropped to remove damaged rail ends
- Rails shall not have more than the maximum wear given in Table 1 appropriate to the mass per meter of the rails
- Rails shall have no holes through the web other than fish bolt holes, and fish bolt holes shall not be flame cut.

Nominal	Height of rail, mm. min.			Side wear.
kg/m	Class			Kg / m. max.
mass of rail	A	B	C	All classes
48	142,6	139,3	136,2	5,2

Table 9.10 - Maximum Permissible Wear of 48kg/m Rails

Fishplates

Second hand fishplates shall be supplied due to the cost effectiveness thereof and the long lead times to order new fishplates.

Fishplates shall be supplied in pairs and shall be of the correct design for the type of rail being laid. Second-hand fishplates shall be in such condition that the remaining draw is at least 5 mm when the fish bolts are pulled up to a torque of 340 N.m.

Fish bolt holes shall not be worn or deformed.

Rail-to-Sleeper Fastenings

New Rail-to-Sleeper Fastenings for the concrete sleepers shall be supplied. We included the first type with is similar in price to the Pandrol fastening system. This can be finalised in the design phase. New B1B2 Gauge plates, new F4 pins and new F4 clips shall be supplied. Rail-to sleeper fastenings for the turnouts shall be second hand and be supplied with the turnouts.

Soleplates shall be of a type to suit the rail sections specified. Second-hand soleplates shall be in sound condition with no noticeable ridges on the rail flange seat.

Rail chairs and check-rail chairs shall be suitable for use with the rail sections specified, and shall be supplied complete with rail-to-chair fastenings. Second-hand rail chairs and check-rail chairs shall *be* in sound condition.

Coach screws for hardwood sleepers shall be of size M22 Type B galvanized provided that Type A shall be used where the nominal thickness of the base plate is 30 mm. Type B galvanized coach screws shall be used with softwood and laminated softwood sleepers.

Stop blocks

Second hand stop blocks shall be supplied due to the cost effectiveness thereof and the long lead times to order new stop blocks. One stop block per siding termination point shall be installed.

Stop blocks, of the fixed type shall be constructed with Class B (See table 1) or better second-hand rails of the same mass per metre as that of the rails to which they are connected, and shall include buffer beams, hardwood bumping blocks, running rails and fastenings, accessories, and bearers.

Derail Devices

Second-hand derail devices shall be supplied due to the cost effectiveness thereof and the long lead times to order new derail devices.

Derail Devices shall be placed to protect potential traffic on the mainline from runaway trucks.
They shall be a standard scotch block manufactured from 30 kg/m rail or from any suitable rolled steel section that is not too heavy to handle. Each scotch block shall be supplied with the additional sleepers and fastenings required and fitted with a stout chain and eyebolt to ensure locking in either the on or the off position.

Insulated Rail Joints (Block Joints) and Rail Bonds.

New block joints shall be supplied to isolate the siding from the mainline's traction electrical supply or return current.

Insulated rail joints shall consist of laminated wood fishplates, approved insulation, and insulating shims of thickness at least 5 mm between rails.

Traction bonds shall be braided copper wire of cross-sectional area at least 20 mm^2, bonded to the rails in an acceptable manner.

Ballast

Ballast shall consist of approved hard broken hornfels, dolerite, tillite, quartzite, or similar crushed stone that complies with requirements for single-sized crushed stone for roads of nominal size 53.0 mm given in SABS 1083, except that the nominal aperture size of the largest sieve used for the grading shall be 63.0 mm (instead of 75.0 mm) and, instead of the applicable ACV specified in SABS 1083, the stone shall have an abrasion resistance of 34 as determined by the Los Angeles abrasion test. Samples and a grading analysis of stone that the Contractor proposes to use as ballast shall be submitted to the Engineer for examination and approval.

Clearance markers and sign boards

Clearance markers shall consist of lengths of rail or other approved material of nominal length 1.7 m placed at the safe clearance point behind turnouts.

Whistle boards, siding number boards, and other railway signboards required shall be of standard Transnet pattern, shall be painted in accordance with standard Transnet practice, and shall comply with the regulations of Transnet.

In the case of level crossings, road signs shall conform to the requirements of the applicable Road Traffic Ordinance.

9.5.2 PROPOSED RAIL INFRASTRUCTURE

The most cost effective way to supply railway trucks to the development was to construct a staging yard for 50 trucks. These trucks can then be shunted as per the individual owner's requirements to a maximum of 10 trucks per stand. The costs will be to supply the supporting infrastructure only (Tracks on stand owners' properties excluded. Only turnouts provided). The owner of each stand will have to build their own siding line with stop block.

There will be a staging yard on the property's south eastern corner on the municipal servitude. The feeder line will run on the south eastern side of the industrial stands next to the entrance road. The stand sidings allows for approximately 30m of access but the main service railway line will have to be crossed by a level crossing.

Staging Yard Capacity:	50 Trucks
Staging Capacity per Stand:	10 Trucks (Stand layout dependant. Some stands have more capacity)
Total length of rails required:	8 300m
Total number of turnouts required:	10
Quantity of soil to cut:	16 400 m³
Quantity of soil to fill:	1 800 m³
Deepest cut:	± 1.7 m
Highest fill:	± 2 m

The cost estimate for this option is some R 28,300,000 Excl VAT

Figure 9.13: Proposed railway layout

The area is flatter and therefore the cost of bulk earthworks is significantly lower. The line will be parallel with the access road and therefore a level crossing per stand will be required.

9.5.3 CONCLUSIONS AND RECOMMENDATIONS

We recommend the approval of Option 2 due to the following reasons:

- Cut and fill dimensions is favourable and practical
- There should be enough cut material to construct the fill banks.
- More stand area will be available for development.
- Financially it is a better option

10. COST ESTIMATE

The following cost estimate is based on first order calculations. It is therefore suitable for high level budgetary purposes only.

10.1 Summary of costs:

PHASE 1F - RICHARDSBAY IDZ - 20 MAY 2013	
Construction costs	**R 55 912 583.17**
Contingency of 10%	R 5 591 258.32
Escalation of 5%	R 2 795 629.16
Total Construction Estimate	R 64 299 470.65
Professional costs	R 5 582 132.49
Bulk services contribution costs	0
TOTAL PROJECT COSTS - EXCL VAT	**R 69 881 603.13**

10.2 Cost breakdown:

PHASE 1F - RICHARDSBAY IDZ - 20 MAY 2013		
1	**WATER CONNECTION**	**R 3 483 844.53**
	Preliminary and General (12%)	R 373 269.06
	Earthworks	R 278 491.00
	Pressure pipe and fittings	R 2 047 972.48
	Thrust blocks and miscelanous	R 14 112.00
	Stand connections and meters	R 720 000.00
	Municipal connection	R 50 000.00
2	**STORMWATER DRAINAGE**	**R 6 647 200.00**
	Preliminary and General (12%)	R 712 200.00
	Earthworks	R 1 645 000.00
	Pipe materials:	R 1 255 000.00
	Manholes, kerb inlets, junction boxes, etc	R 1 885 000.00
	Accecories: inlets and grids	R 380 000.00
	Gabions and Pitching	R 220 000.00
	Road crossing culvert	R 450 000.00
	Open drains	R 100 000.00

3	**SUBSOIL DRAINS**	**R 540 000.00**
	Earthworks	R 120 000.00
	Piping	R 180 000.00
	Accessories	R 240 000.00
4	**SEWERS**	**R 2 514 305.20**
	Preliminary and General (12%)	R 269 389.84
	Earthworks	R 748 638.00
	Pipe materials	R 948 908.40
	Manholes & Rodding eyes	R 447 368.96
	Connection onto existing sewers	R 40 000.00
	Drainage layer	R 60 000.00
5	**ACCESS ROAD AND INTERSECTION**	**R 19 917 233.45**
	Preliminary and General (10%)	R 1 810 657.59
	Earthworks	R 2 534 904.90
	Layerworks	R 9 747 765.76
	Surfacing and paving	R 4 359 943.00
	Kerbing and Channeling	R 583 620.00
	Signage and road markings	R 880 342.20
6	**RIALWAY INFRASTRUCTURE (Option 2)**	**R 22 810 000.00**
	Preliminary and General (10%)	Incl
	Earthworks	R 7 220 000.00
	Rails	R 4 140 000.00
	Concrete sleepers and fastening	R 3 960 000.00
	Turn-out 1:9 wood complete	R 2 000 000.00
	Ballast	R 450 000.00
	Ancillary works	R 3 190 000.00
	Drainaige	R 1 850 000.00
	TOTAL CONSTRUCTION COSTS	**R 55 912 583.17**
7	**BULK SERVICE CONTRIBUTION**	**R 0.00**
	Bulk water infrastructure	R 0.00
	Bulk sewer infrastructure	R 0.00
	TOTAL BSC COSTS	**R 0.00**
8	**PROFFESIONAL COSTS**	**R 5 582 132.49**
	Professional fee (Approx)	R 5 032 132.49
	Disbursement	R 150 000.00
	Pre development costs	R 400 000.00
	TOTAL PROFESSIONAL COSTS	**R 5 582 132.49**

11. CONCLUSION AND RECOMMENDATIONS

The following recommendations are to be noted:

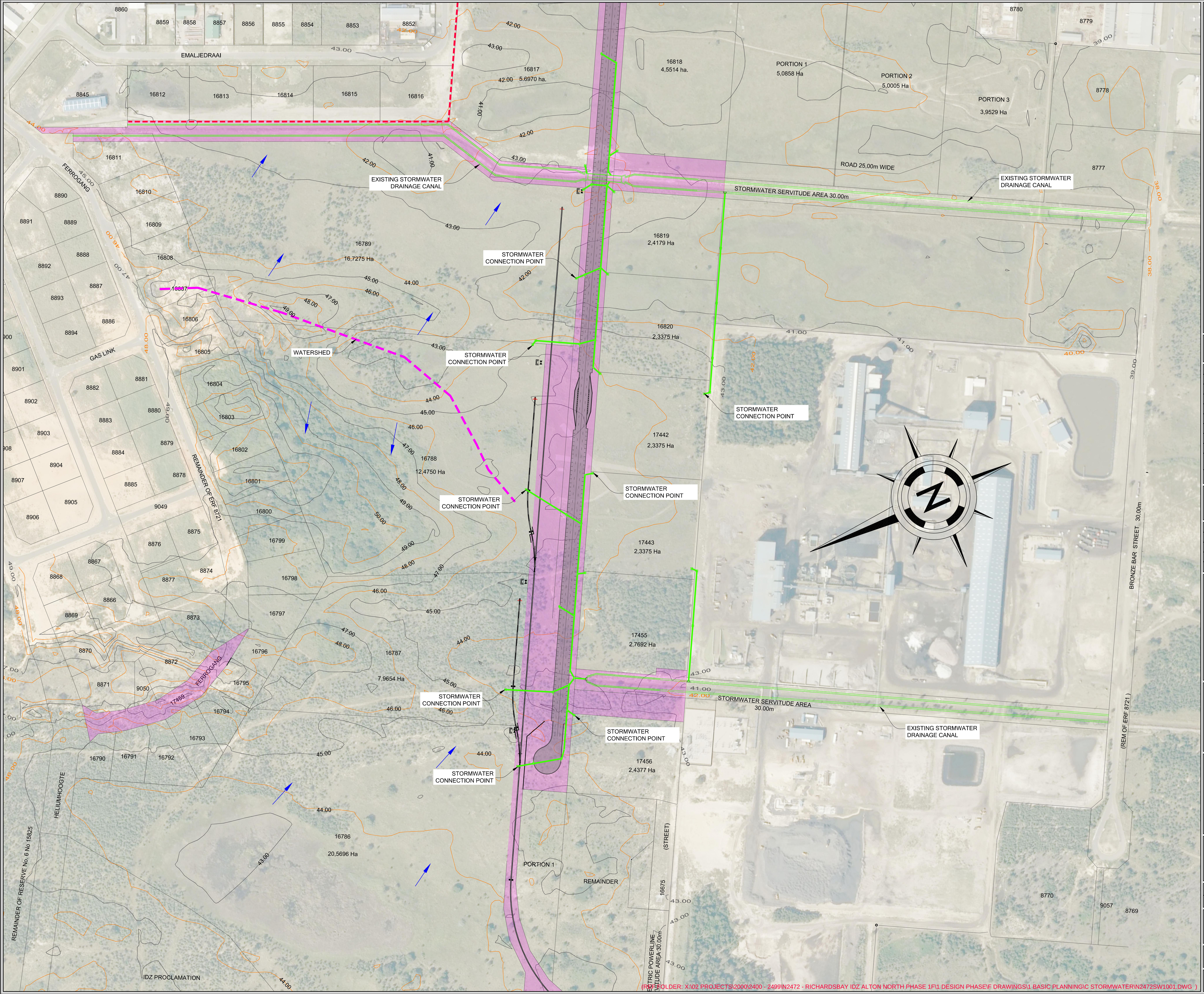
- A detailed Traffic impact assessment be undertaken to obtain figures for road design
- A decision be taken on the option to be followed in terms of the railway infrastructure
- A detailed survey be undertaken to obtain the sewer infrastructure capacities and levels
- The geotechnical investigation be completed and included into this report.
- A road route centre line survey to be undertaken
- A detail design be undertaken in order to determine the final costs of the project.
- The bulk services cost contributions to be finalised with the local authority
- The option of an elevated reservoir to increase pressure boosting be discussed. It is currently not included into the cost estimation.

ANNEXURE A: GENERAL LAYOUT OF THE SITE

ANNEXURE B: WATER SERVICES LAYOUT AND DETAILS

ANNEXURE C: SANITATION INFRASTRUCTURE LAYOUT AND DETAILS

ANNEXURE D: STORMWATER SERVICES LAYOUT AND DETAILS



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LEGEND:
- RAILWAY LINE
- PROPOSED STORMWATER
- WATERSHED

FOR INFORMATION PURPOSES ONLY

Rev. Her.	Description Beskrywing	Date Datum	By Deur

AMENDMENTS/WYSIGING

SERVICE / DIENS

GENERAL

DESIGNED/ONTWERP	N. V.D. MERWE
DRAWN/GETEKEN	N. V.D. MERWE
CAD FILE NAME/CAD LÊER NAAM	N2472SW1001.dwg
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APPROVED/GOEDGEKEUR	M.J.M.H VENTER

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CLIENT / KLIËNT

RICHARDS BAY INDUSTRIAL DEVELOPMENT ZONE

PROJECT / PROJEK

ALTON NORTH PHASE 1F

TITLE / TITEL

STORMWATER - GENERAL LAYOUT

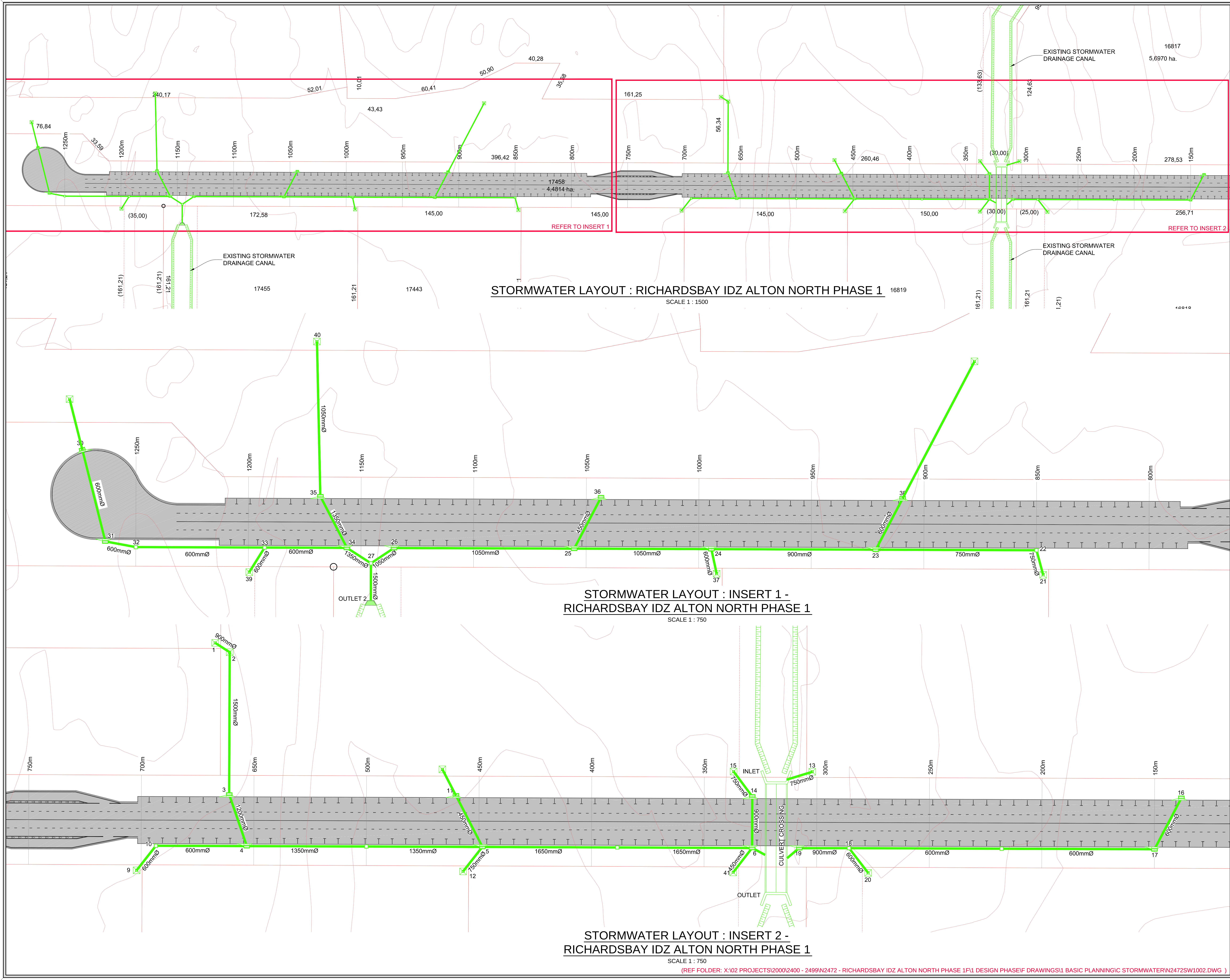
DATE/DATUM	MAY 2013
SCALE/SKAAL	N.T.S.

SCALE REF. / SKAAL VERWYS.

CONSULTANT DWG. NO. / KONSULTANT TEK. NR.

N/472/SW/1/001

CLIENT DWG. NO. / KLIENT TEK. NO.



LEGEND:

- STORMWATER PIPE
- KERB INLET STRUCTURE
- FIELD INLET STRUCTURE
- ELECTRICAL SLEEVE

FOR INFORMATION PURPOSES ONLY

Rev. Her.	Description Beskrywing	Date Datum	By Deur

AMENDMENTS/WYSIGING

SERVICE / DIENS

STORMWATER

DESIGNED/ONTWERP	A.J. NEL
DRAWN/GETEKEN	J.M. VENTER
CAD FILE NAME/CAD LÊER NAAM	N2472SW1002.dwg
CHECKED/NAGESIEN	A.J. NEL
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CLIENT / KLIËNT

RICHARDS BAY INDUSTRIAL DEVELOPMENT ZONE

PROJECT / PROJEK

ALTON NORTH PHASE 1F

TITLE / TITEL

STORMWATER - GENERAL LAYOUT: NEW ROAD

DATE/DATUM	MAY 2013
SCALE/SKAAL	AS SHOWN

SCALE REF. / SKAAL VERWYS.

CONSULTANT DWG. NO. / KONSULTANT TEK. NR.: N/2472/SW/1/002

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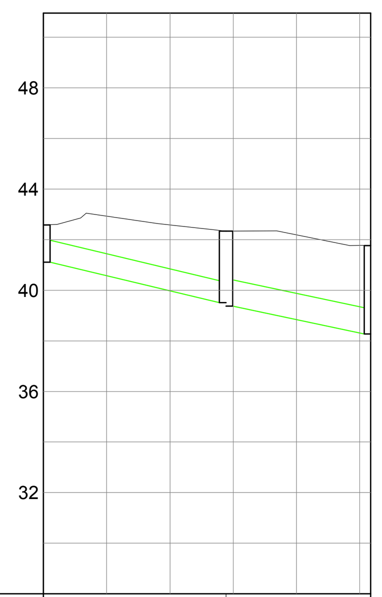

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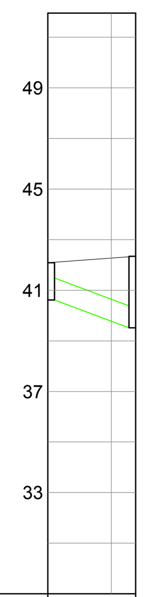
28.000

Chainage	0.000	28.853	51.733
Manhole Name	13	14	6
Ground Level	42.577	42.337	41.761
Depth	1.416	2.881	4.291
Inlet / Outlet	41.161	39.563 / 39.456	38.357 / 37.471

Length	28.853 m	22.880 m
Link Type / Link Size	50D Concrete 750 mm	100D Concrete 900 mm
Slope	5.538 %	4.805 %
Flow / Capacity	1.515 m³/s / 2.733 m³/s	3.126 m³/s / 3.614 m³/s
Velocity	6.148 m/s	6.856 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 13 TO 6



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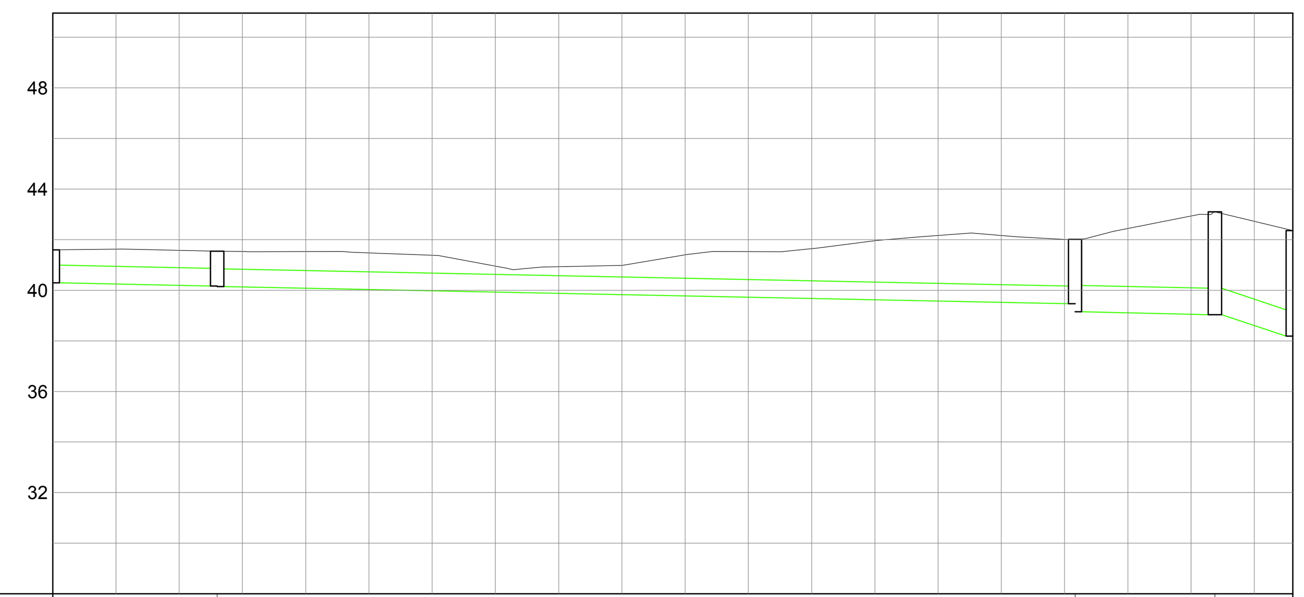
29.000

Chainage	0.000	13.853
Manhole Name	15	14
Ground Level	42.085	42.337
Depth	1.416	2.881
Inlet / Outlet	40.669	39.563

Length	13.853 m
Link Type / Link Size	50D Concrete 750 mm
Slope	7.984 %
Flow / Capacity	1.990 m³/s / 3.282 m³/s
Velocity	7.540 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 15 TO 14



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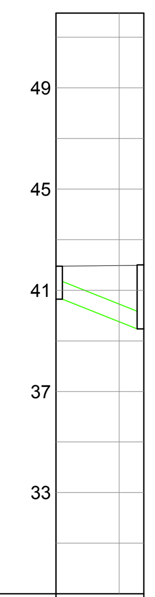
Vertical Scale :250

28.000

Chainage	0.000	25.992	161.654	183.765	196.069
Manhole Name	16	17	18	19	7
Ground Level	41.590	41.539	42.005	43.099	42.360
Depth	1.242	1.347	2.794	3.997	4.952
Inlet / Outlet	40.348	40.218 / 40.192	39.514 / 39.212	39.101 / 39.101	38.250

Length	25.992 m	135.662 m	22.111 m	12.304 m
Link Type / Link Size	100D Concrete 600 mm	50D Concrete 600 mm	50D Concrete 900 mm	50D Concrete 900 mm
Slope	0.500 %	0.500 %	0.500 %	6.920 %
Flow / Capacity	0.221 m³/s / 0.406 m³/s	0.221 m³/s / 0.456 m³/s	1.264 m³/s / 1.330 m³/s	1.384 m³/s / 4.948 m³/s
Velocity	1.542 m/s	1.543 m/s	2.310 m/s	6.487 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 16 TO 7



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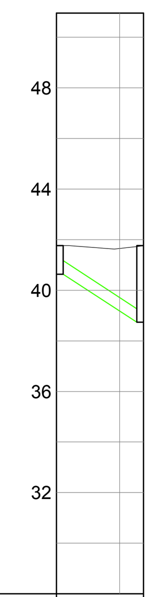
29.000

Chainage	0.000	13.892
Manhole Name	20	18
Ground Level	41.945	42.005
Depth	1.255	2.794
Inlet / Outlet	40.690	39.514 / 39.212

Length	13.892 m
Link Type / Link Size	50D Concrete 600 mm
Slope	8.467 %
Flow / Capacity	1.131 m³/s / 1.875 m³/s
Velocity	6.693 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 20 TO 18



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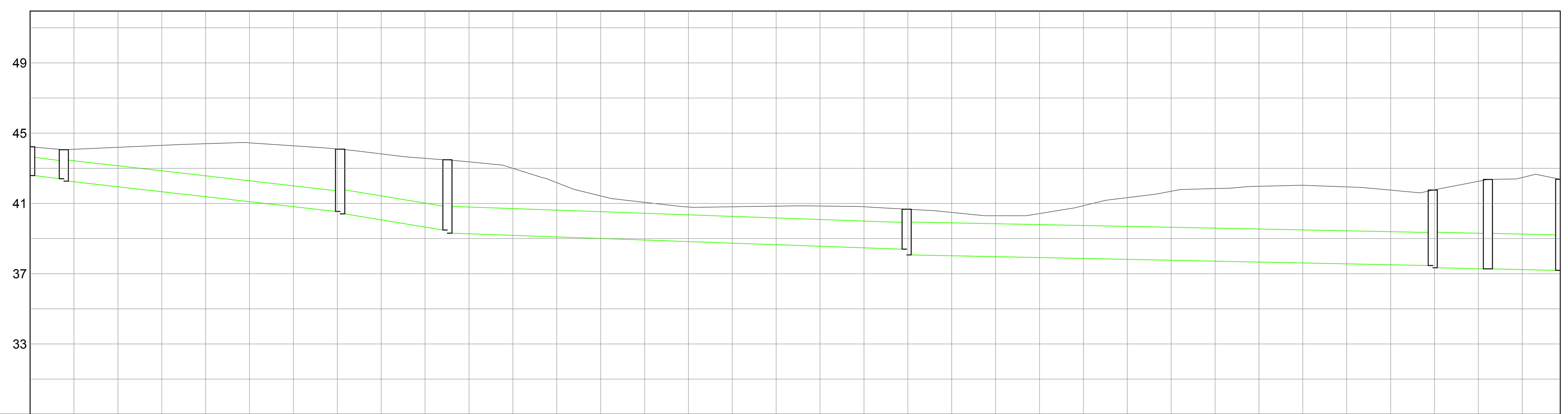
28.000

Chainage	0.000	13.760
Manhole Name	41	6
Ground Level	41.766	41.761
Depth	1.089	4.291
Inlet / Outlet	40.677	38.781 / 37.471

Length	13.760 m
Link Type / Link Size	50D Concrete 450 mm
Slope	13.782 %
Flow / Capacity	0.474 m³/s / 1.027 m³/s
Velocity	6.475 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 41 TO 6



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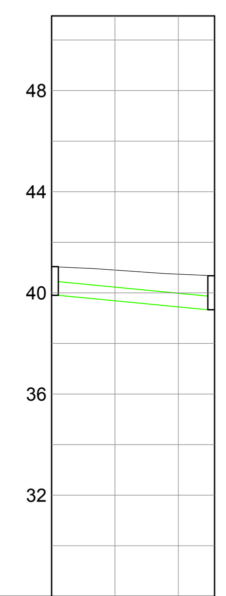
29.000

Chainage	0.000	7.673	70.648	95.081	199.732	319.615	332.185	348.663
Manhole Name	1	2	3	4	5	6	7	OUTLET
Ground Level	44.227	44.061	44.084	43.483	40.672	41.761	42.360	42.378
Depth	1.577	1.714	3.557	4.078	2.487	4.291	4.952	5.053
Inlet / Outlet	42.650	42.468 / 42.347	40.620 / 40.527	39.606 / 39.405	38.497 / 38.185	37.586 / 37.471	37.408 / 37.408	37.325 / 37.325

Length	7.673 m	62.975 m	24.433 m	104.651 m	119.883 m	12.570 m	16.478 m
Link Type / Link Size	50D Concrete 900 mm	50D Concrete 1050 mm	100D Concrete 1200 mm	50D Concrete 1350 mm	50D Concrete 1650 mm	50D Concrete 1800 mm	50D Concrete 1800 mm
Slope	2.372 %	2.742 %	3.769 %	0.868 %	0.500 %	0.500 %	0.500 %
Flow / Capacity	2.661 m³/s / 2.897 m³/s	3.970 m³/s / 4.341 m³/s	3.970 m³/s / 6.403 m³/s	4.403 m³/s / 4.758 m³/s	4.836 m³/s / 6.341 m³/s	7.597 m³/s / 7.597 m³/s	7.597 m³/s / 7.597 m³/s
Velocity	5.018 m/s	5.861 m/s	6.761 m/s	3.899 m/s	3.305 m/s	3.141 m/s	3.141 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 1 TO OUTLET



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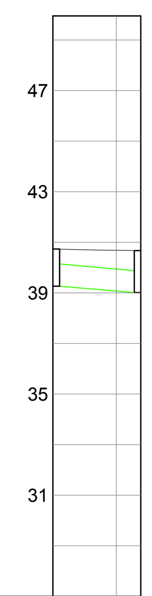
Vertical Scale :250

28.000

Chainage	0.000	25.699
Manhole Name	11	5
Ground Level	41.039	40.672
Depth	1.089	2.487
Inlet / Outlet	39.950	39.380 / 38.185

Length	25.699 m
Link Type / Link Size	100D Concrete 450 mm
Slope	2.218 %
Flow / Capacity	0.155 m³/s / 0.412 m³/s
Velocity	2.465 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 11 TO 5



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27.000

Chainage	0.000	13.860
Manhole Name	12	5
Ground Level	40.738	40.672
Depth	1.416	2.487
Inlet / Outlet	39.322	39.063 / 38.185

Length	13.860 m
Link Type / Link Size	50D Concrete 750 mm
Slope	1.869 %
Flow / Capacity	0.578 m³/s / 1.588 m³/s
Velocity	3.209 m/s

LONGITUDINAL SECTION THROUGH STORMWATER 12 TO 5

FOR INFORMATION PURPOSES ONLY

Rev. Her.	Description Beskrywing	Date Datum	By Deur

AMENDMENTS/WYSIGING

SERVICE DIENS

STORMWATER

DESIGNED/ONTWERP	A.J. NEL
DRAWN/GETEKEN	J.M. VENTER
CAD FILE NAME/CAD LÊER NAAM	N2472SW2001-2.dwg
CHECKED/NAGESIEN	A.J. NEL
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CLIENT KLIËNT

CLIENT

PROJECT PROJEK

RICHARDSBAY IDZ ALTON NORTH PHASE 1

TITLE TITEL

STORMWATER - LONGITUDINAL SECTIONS: SHEET 1 OF 2

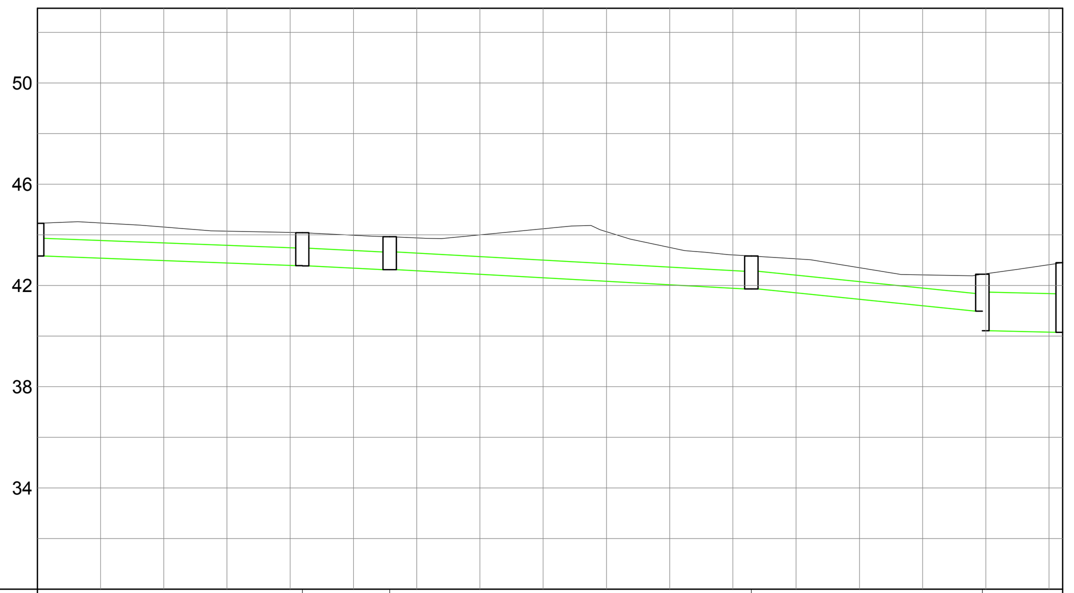
DATE/DATUM	MAY 2013
SCALE/SKAAL	AS SHOWN

SCALE REF. SKAAL VERWYS.
80 40 0 40 80
60 20 20 60
meter

CONSULTANT DWG. NO. KONSULTANT TEK. NR.
N/2472/SW/2/001

CLIENT DWG. NO. KLIËNT TEK. NO.

(REF FOLDER: X:\02 PROJECTS\2000\2400 - 2499\N2472 - RICHARDSBAY IDZ ALTON NORTH PHASE 1F\1 DESIGN PHASE\F DRAWINGS\1 BASIC PLANNING\C STORMWATER\N2472SW2001-2.DWG)



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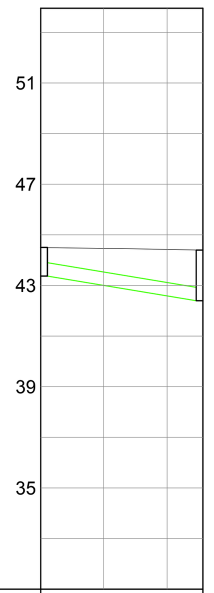
30.000

Chainage	0.000	41.913	55.731	112.920	149.436	162.148
Manhole Name	30	31	32	33	34	27
Ground Level	44.460	44.086	43.922	43.161	42.442	42.897
Depth	1.242	1.268	1.255	1.255	2.135	2.787
Inlet / Outlet	43.218	42.844 / 42.818	42.667	41.906	41.024 / 40.307	40.243 / 40.110

Length	41.913 m	13.818 m	57.189 m	36.516 m	12.712 m
Link Type / Link Size	100D Concrete 600 mm	50D Concrete 600 mm	50D Concrete 600 mm	50D Concrete 600 mm	50D Concrete 1350 mm
Slope	0.892 %	1.093 %	1.331 %	2.417 %	0.500 %
Flow	0.262 m³/s	0.262 m³/s	0.262 m³/s	0.658 m³/s	2.779 m³/s
Capacity	0.542 m³/s	0.674 m³/s	0.744 m³/s	1.002 m³/s	3.612 m³/s
Velocity	2.001 m/s	2.156 m/s	2.317 m/s	3.647 m/s	2.876 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 30 TO 27



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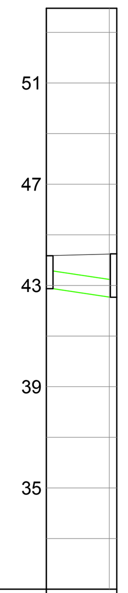
31.000

Chainage	0.000	25.695
Manhole Name	36	25
Ground Level	44.504	44.395
Depth	1.089	2.542
Inlet / Outlet	43.415	42.442 / 41.853
Length		25.695 m
Link Type / Link Size		100D Concrete 450 mm
Slope		3.788 %
Flow		0.313 m³/s
Capacity		0.539 m³/s
Velocity		3.593 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 36 TO 25



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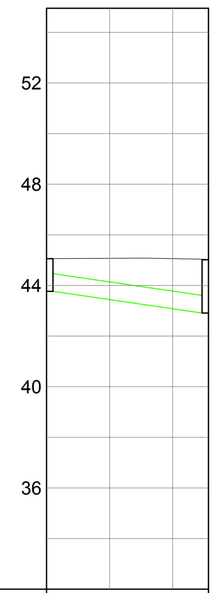
31.000

Chainage	0.000	11.190
Manhole Name	37	24
Ground Level	44.175	44.246
Depth	1.255	2.089
Inlet / Outlet	42.920	42.580
Length		11.190 m
Link Type / Link Size		50D Concrete 600 mm
Slope		3.036 %
Flow		0.556 m³/s
Capacity		1.123 m³/s
Velocity		3.820 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 37 TO 24



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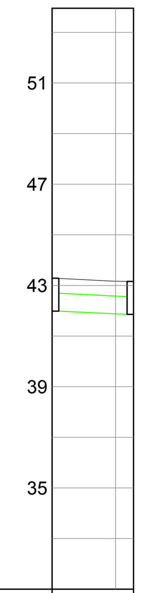
32000

Chainage	0.000	25.691
Manhole Name	38	23
Ground Level	45.062	45.022
Depth	1.242	2.378
Inlet / Outlet	43.820	42.971 / 42.643
Length		25.691 m
Link Type / Link Size		100D Concrete 600 mm
Slope		3.303 %
Flow		0.653 m³/s
Capacity		1.043 m³/s
Velocity		4.096 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 38 TO 23



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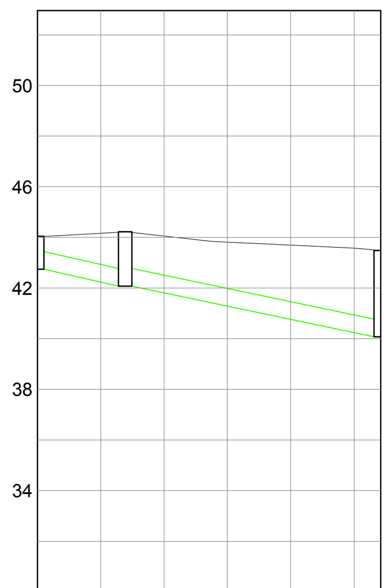
Vertical Scale :250

31.000

Chainage	0.000	12.862
Manhole Name	39	33
Ground Level	43.289	43.161
Depth	1.255	1.255
Inlet / Outlet	42.034	41.906 / 41.906
Length		12.862 m
Link Type / Link Size		50D Concrete 600 mm
Slope		0.995 %
Flow		0.527 m³/s
Capacity		0.643 m³/s
Velocity		2.446 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 39 TO 33



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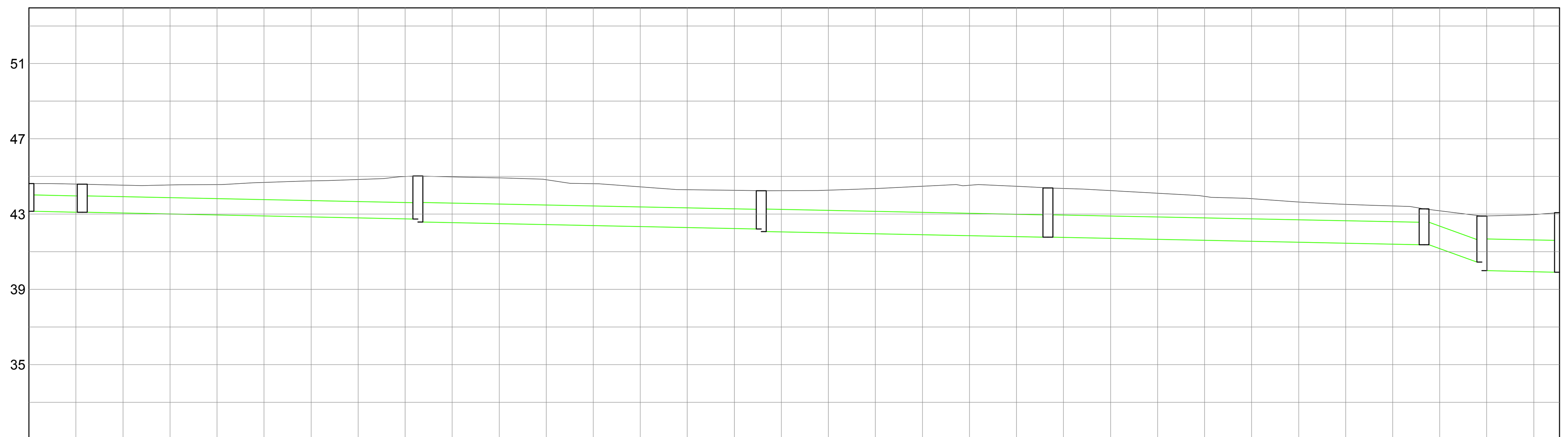
30.000

Chainage	0.000	13.864	54.209
Manhole Name	9	10	4
Ground Level	44.048	44.224	43.483
Depth	1.255	2.098	4.078
Inlet / Outlet	42.793	42.126 / 42.126	40.122 / 39.405

Length	13.864 m	40.345 m
Link Type / Link Size	50D Concrete 600 mm	50D Concrete 600 mm
Slope	4.811 %	4.967 %
Flow	0.577 m³/s	0.577 m³/s
Capacity	1.414 m³/s	1.436 m³/s
Velocity	4.578 m/s	4.633 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 9 TO 4



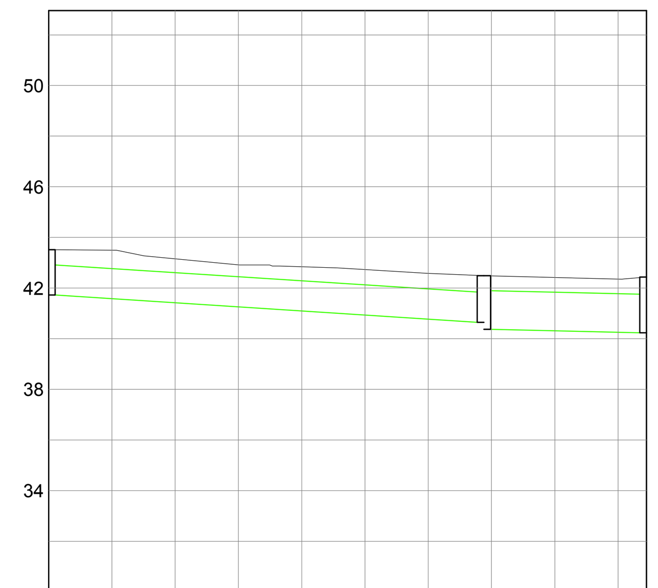
Horizontal Scale: 1000
Vertical Scale :250

31.000

Chainage	0.000	11.333	82.719	155.738	216.676	296.673	308.977	325.455
Manhole Name	21	22	23	24	25	26	27	OUTLET 2
Ground Level	44.624	44.596	45.022	44.246	44.395	43.282	42.897	43.083
Depth	1.416	1.444	2.378	2.089	2.542	1.830	2.787	3.055
Inlet / Outlet	43.208	43.151	42.794 / 42.643	42.278 / 42.157	41.853	41.453	40.537 / 40.110	40.028

Length	11.333 m	71.386 m	73.019 m	60.938 m	79.997 m	12.304 m	16.478 m
Link Type / Link Size	50D Concrete 750 mm	50D Concrete 750 mm	50D Concrete 900 mm	50D Concrete 1050 mm	50D Concrete 1050 mm	50D Concrete 1050 mm	50D Concrete 1500 mm
Slope	0.500 %	0.500 %	0.500 %	0.500 %	0.500 %	7.442 %	0.500 %
Flow	0.577 m³/s	0.577 m³/s	0.941 m³/s	1.458 m³/s	1.615 m³/s	1.880 m³/s	4.229 m³/s
Capacity	0.821 m³/s	0.821 m³/s	1.330 m³/s	1.853 m³/s	1.853 m³/s	7.151 m³/s	4.659 m³/s
Velocity	1.949 m/s	1.949 m/s	2.203 m/s	2.444 m/s	2.486 m/s	7.178 m/s	3.148 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 21 TO OUTLET 2



Horizontal Scale: 1000
Vertical Scale :250

30.000

Chainage	0.000	68.795	94.488
Manhole Name	40	35	34
Ground Level	43.519	42.494	42.442
Depth	1.714	1.993	2.135
Inlet / Outlet	41.805	40.729 / 40.501	40.373 / 40.307

Length	68.795 m	25.693 m
Link Type / Link Size	50D Concrete 1050 mm	100D Concrete 1350 mm
Slope	1.564 %	0.500 %
Flow	2.383 m³/s	2.383 m³/s
Capacity	3.278 m³/s	3.153 m³/s
Velocity	4.257 m/s	2.771 m/s

LONGITUDINAL SECTION THROUGH
STORMWATER 40 TO 34

FOR INFORMATION PURPOSES ONLY

Rev. Her.	Description Beskrywing	Date Datum	By Deur

AMENDMENTS/WYSIGING

SERVICE DIENS

STORMWATER

DESIGNED/ONTWERP	A.J. NEL
DRAWN/GETEKEN	J.M. VENTER
CAD FILE NAME/CAD LÊER NAAM	N2472SW2001-2.dwg
CHECKED/NAGESIEN	A.J. NEL
APPROVED/GOEDGEKEUR	J.L. VENTER

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CLIENT KLIËNT

CLIENT

PROJECT PROJEK

RICHARDSBAY IDZ ALTON NORTH PHASE 1

TITLE TITEL

STORMWATER - LONGITUDINAL SECTIONS: SHEET 2 OF 2

DATE/DATUM	MAY 2013
SCALE/SKAAL	AS SHOWN

SCALE REF. SKAAL VERWYS.
80 40 0 40 80
60 20 20 60
meter

CONSULTANT DWG. NO. KONSULTANT TEK. NR.
N/2472/SW/2/002

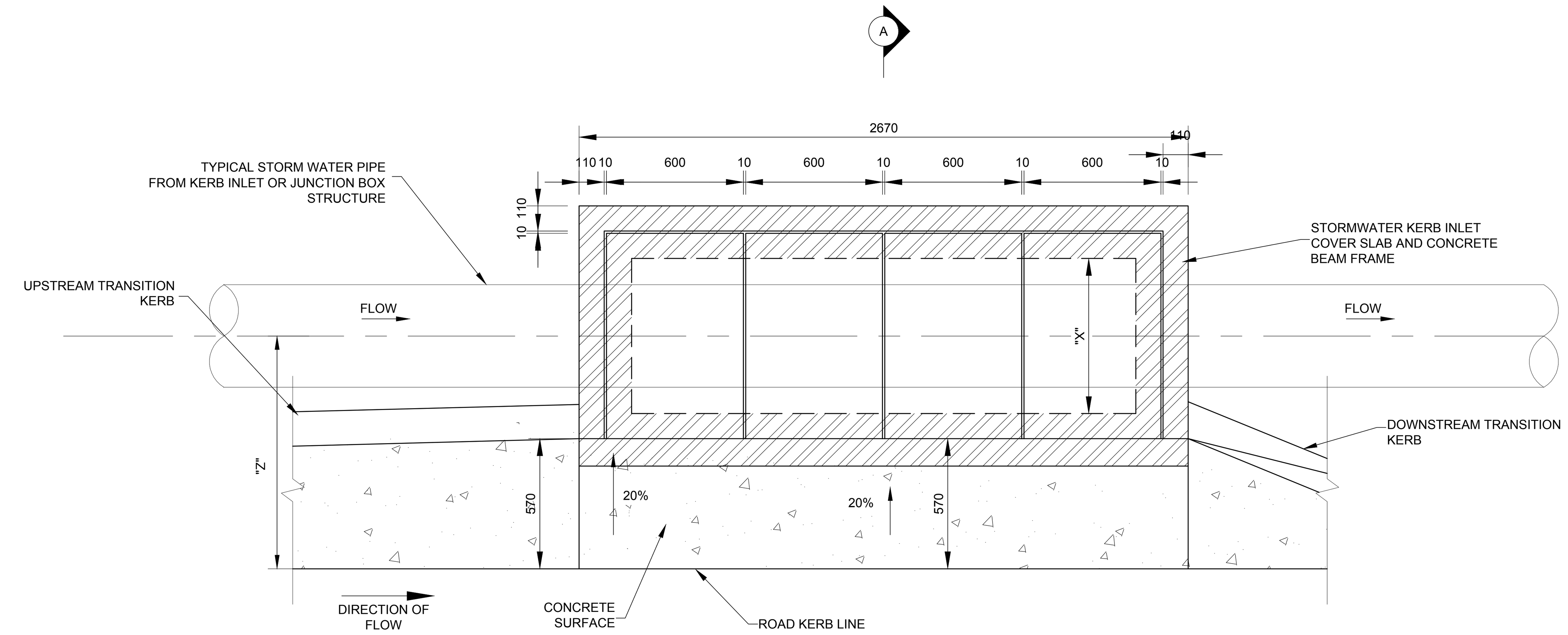
CLIENT DWG. NO. KLIËNT TEK. NO.

(REF FOLDER: X:\02 PROJECTS\2000\2400 - 2499\N2472 - RICHARDSBAY IDZ ALTON NORTH PHASE 1F\1 DESIGN PHASE\F DRAWINGS\1 BASIC PLANNING\C STORMWATER\N2472SW2001-2.DWG)

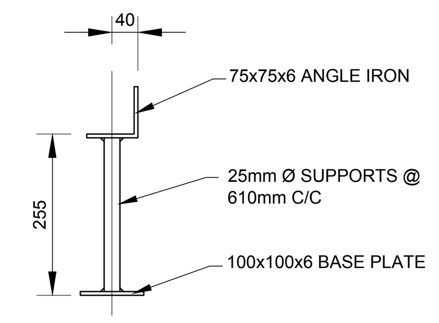
KERB INLET DIMENSIONS

Ø	WIDTH OF KERB INLET STRUCTURE "X"	PIPE CENTRE LINE OFF-SET FROM ROAD/KERB LINE "Z"
600mm Ø	1049mm	1204.5mm
750mm Ø	1220mm	1290mm
900mm Ø	1379mm	1369.5mm
1050mm Ø	1544mm	1452mm
1350mm Ø	1874mm	1617mm
1500mm Ø	2039mm	1699.5mm

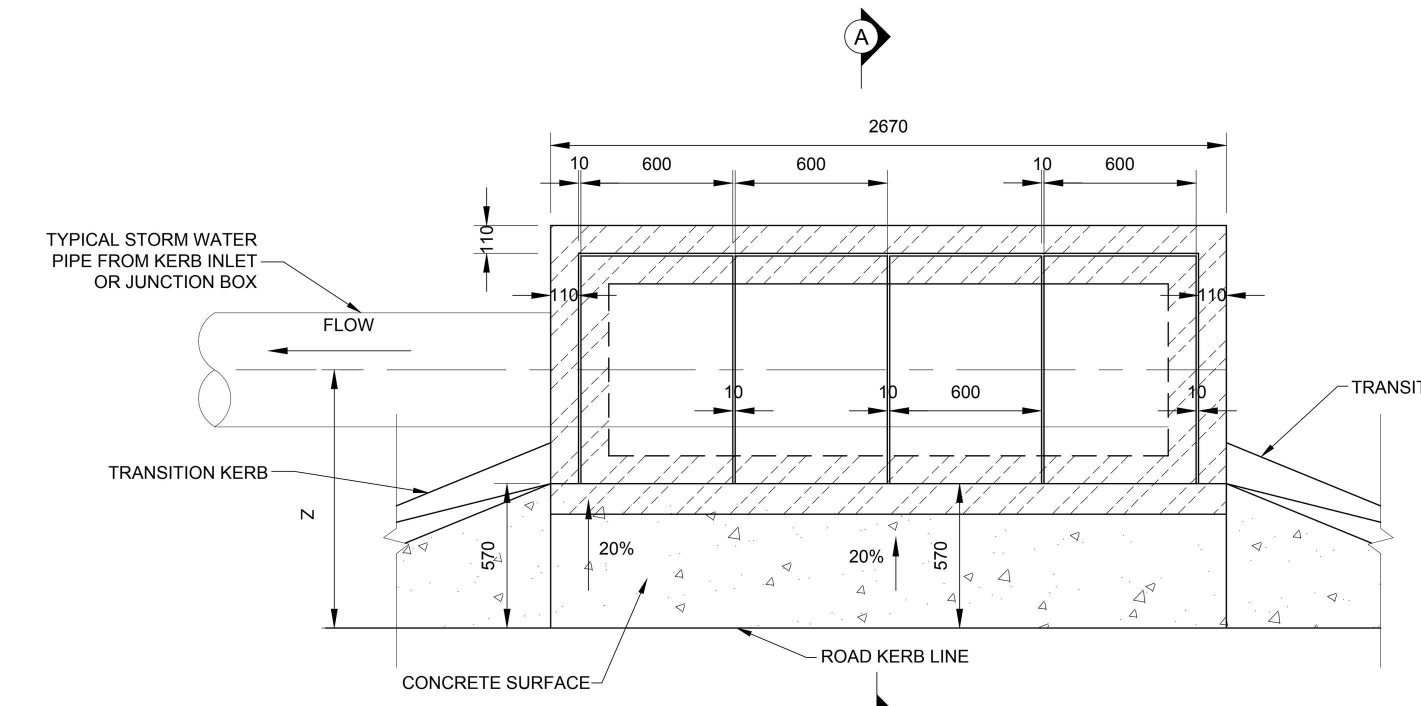
NOTE: THE DIMENSIONS FOR X, Y AND Z TO BE CONFIRMED PRIOR TO CONSTRUCTION OF MASONRY KERB INLET STRUCTURE AND PRECAST KERB INLET CONCRETE LID



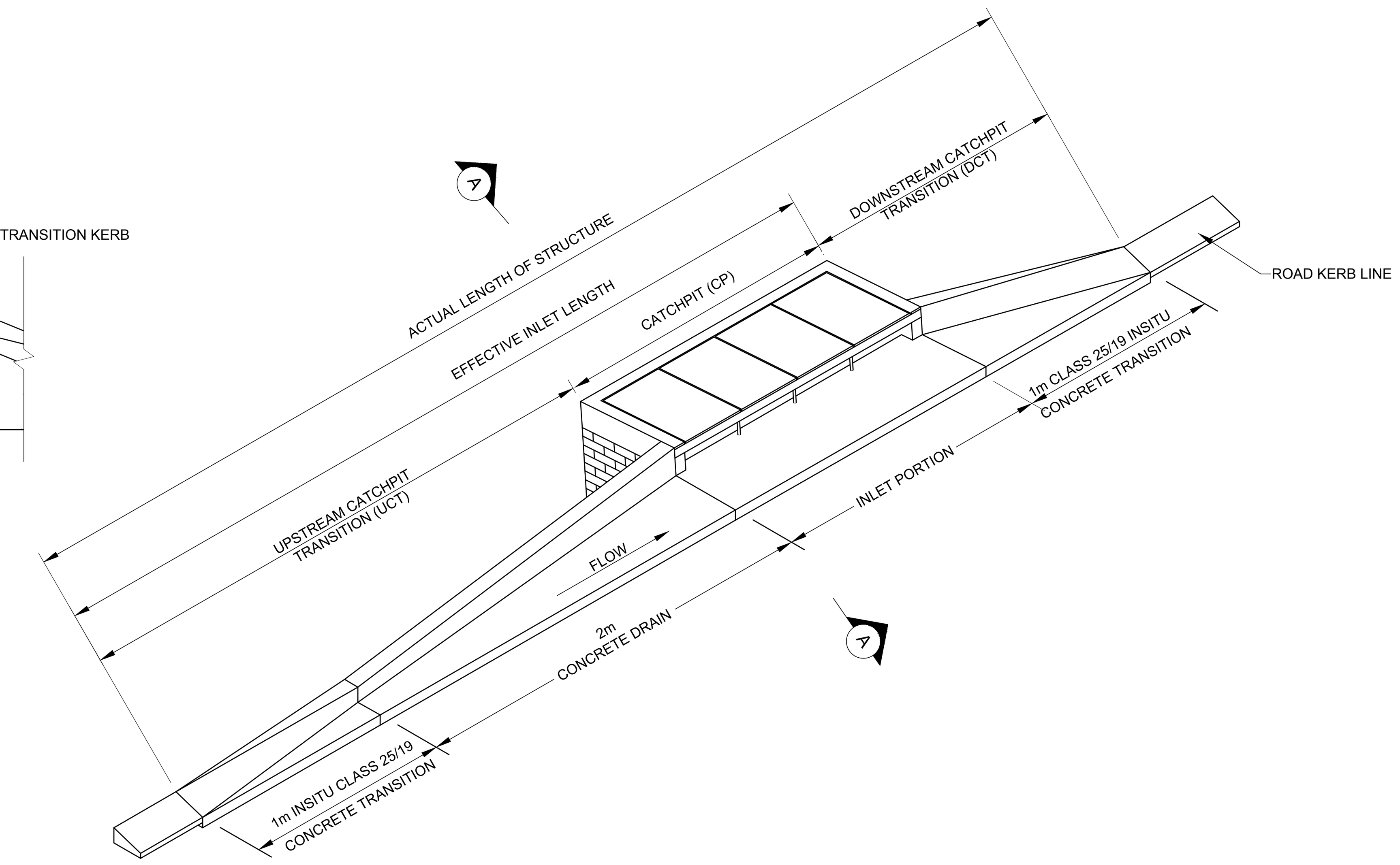
PLAN: KERB INLET
SCALE 1:20



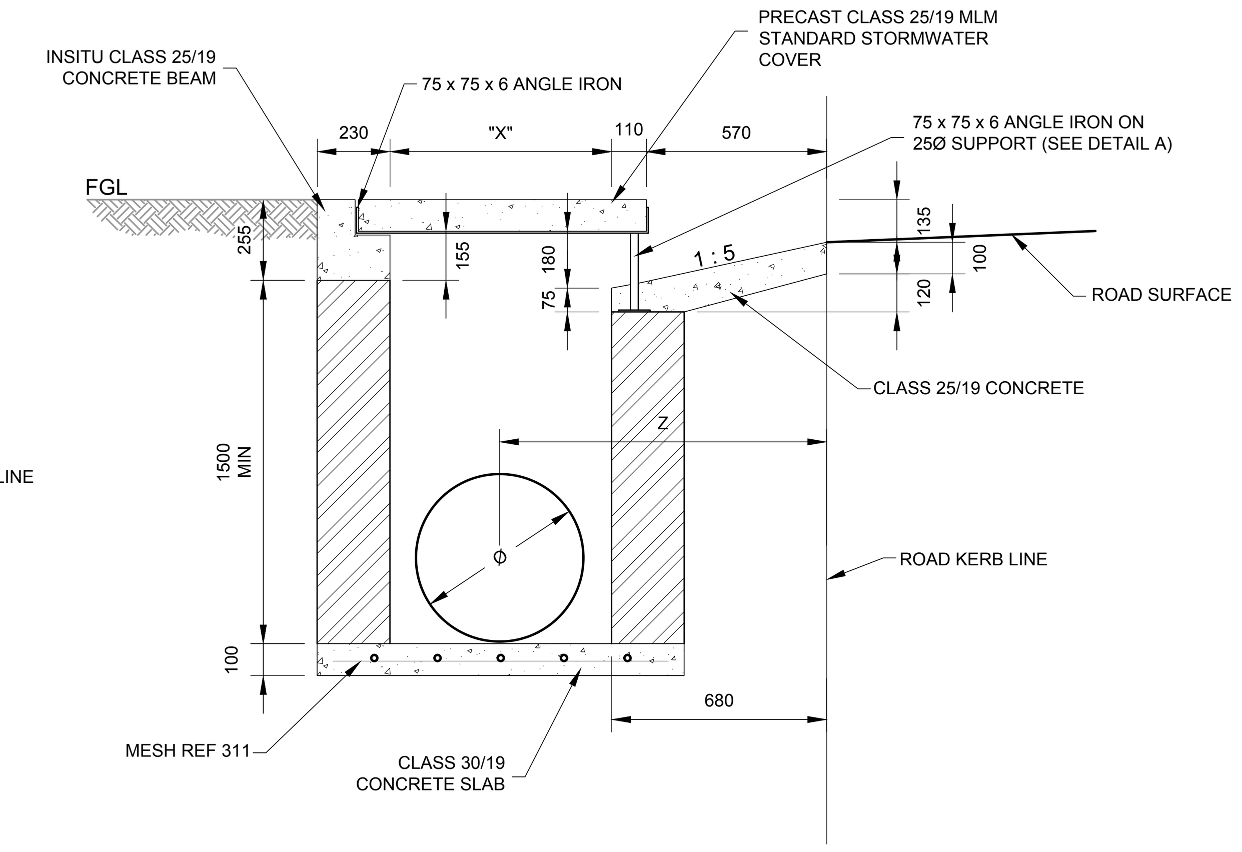
DETAIL A: SUPPORTS FOR CONCRETE PANELS
SCALE 1:10



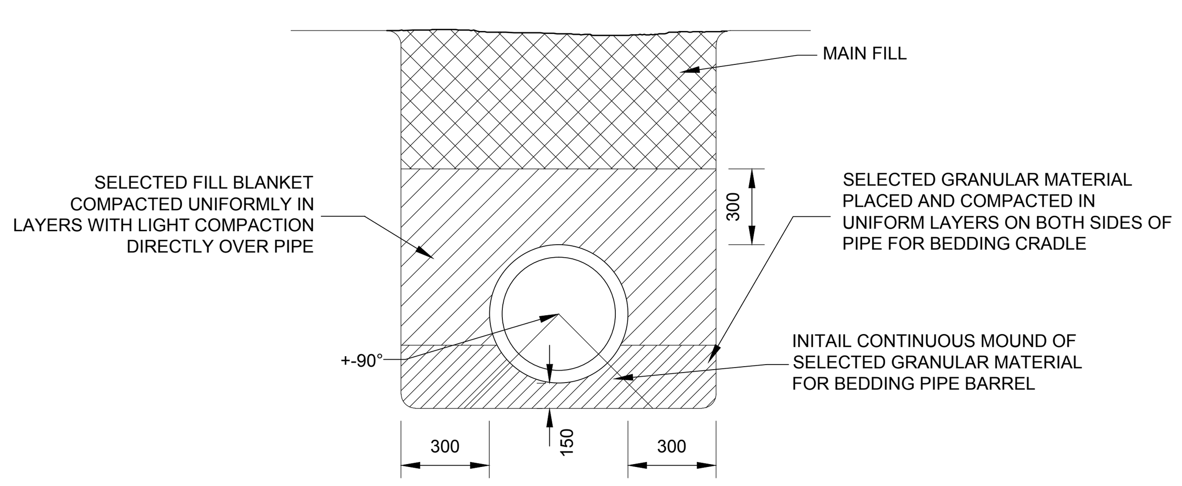
LOW POINT KERB INLET: PLAN
SCALE 1:25



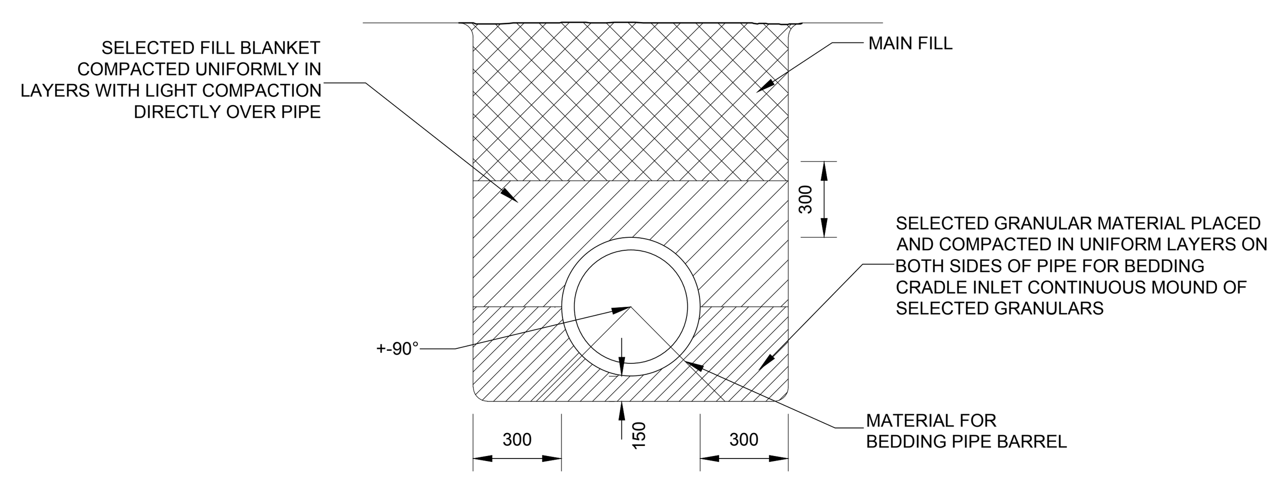
DETAIL OF TRANSITION INLET GUTTER (ISOMETRIC VIEW)
N.T.S



SECTION A - A
SCALE 1:20

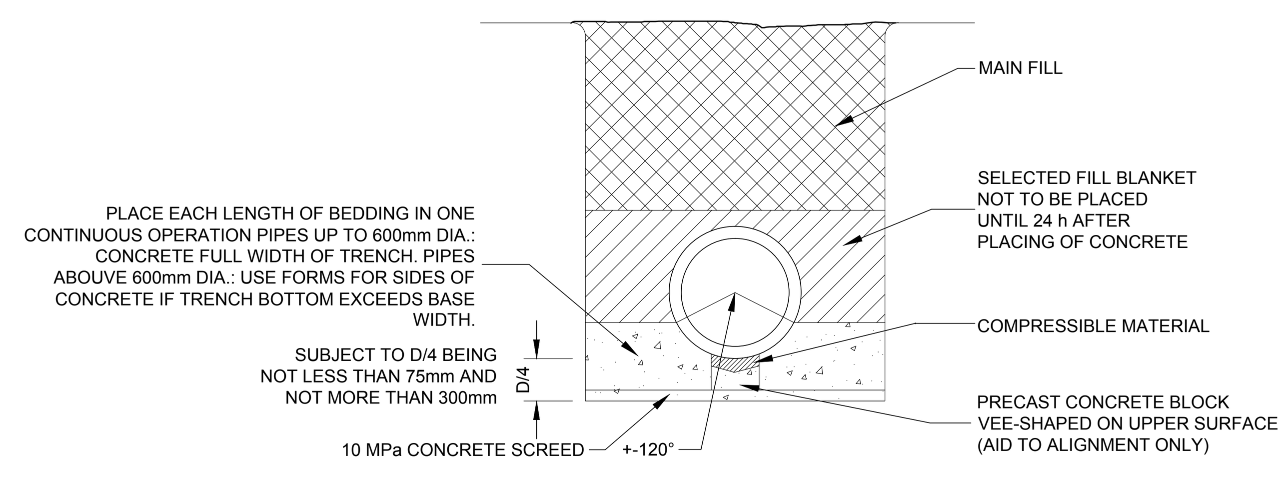


CLASS C BEDDING
N.T.S



CLASS B BEDDING
N.T.S

NOTE:
1. BEDDING SHALL BE OF AN APPROVED SELECTED GRAVEL MATERIAL WITH A PI NOT EXCEEDING 6.
2. BEDDING MATERIAL SHALL BE COMPACTED TO 90% OF MOD. AASHTO DENSITY.



CLASS A BEDDING
N.T.S

NOTE:
1. THE CLASS A IN SITU CONCRETE BEDDING SHALL BE IN ACCORDANCE WITH SABS 0102.

(REF FOLDER: X:\02 PROJECTS\2000\2400 - 2499\N2472 - RICHARDSBAY IDZ ALTON NORTH PHASE 1F\1 DESIGN PHASE\F DRAWINGS\1 BASIC PLANNING\C STORMWATER\N2472SW3001.DWG)

NOTES:

1. STORMWATER STRUCTURE IN ACCORDANCE WITH SABS 1200LE

FOR INFORMATION

Rev. Her.	Description Beskrywing	Date Datum	By Deur

AMENDMENTS/WYSIGING

SERVICE / DIENS

STORMWATER

DESIGNED/ONTWERP	A.J. NEL
DRAWN/GETEKEN	JM. VENTER
CAD FILE NAME/CAD LÊER NAAM	N2472SW3001.dwg
CHECKED/NAGESIEN	A.J. NEL
APPROVED/GOEDGEKEUR	J.L. VENTER

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CLIENT / KLIëNT

PROJECT / PROJEK

RICHARDSBAY IDZ ALTON NORTH PHASE 1

TITLE / TITEL

STORMWATER - DETAILS: KERB INLET

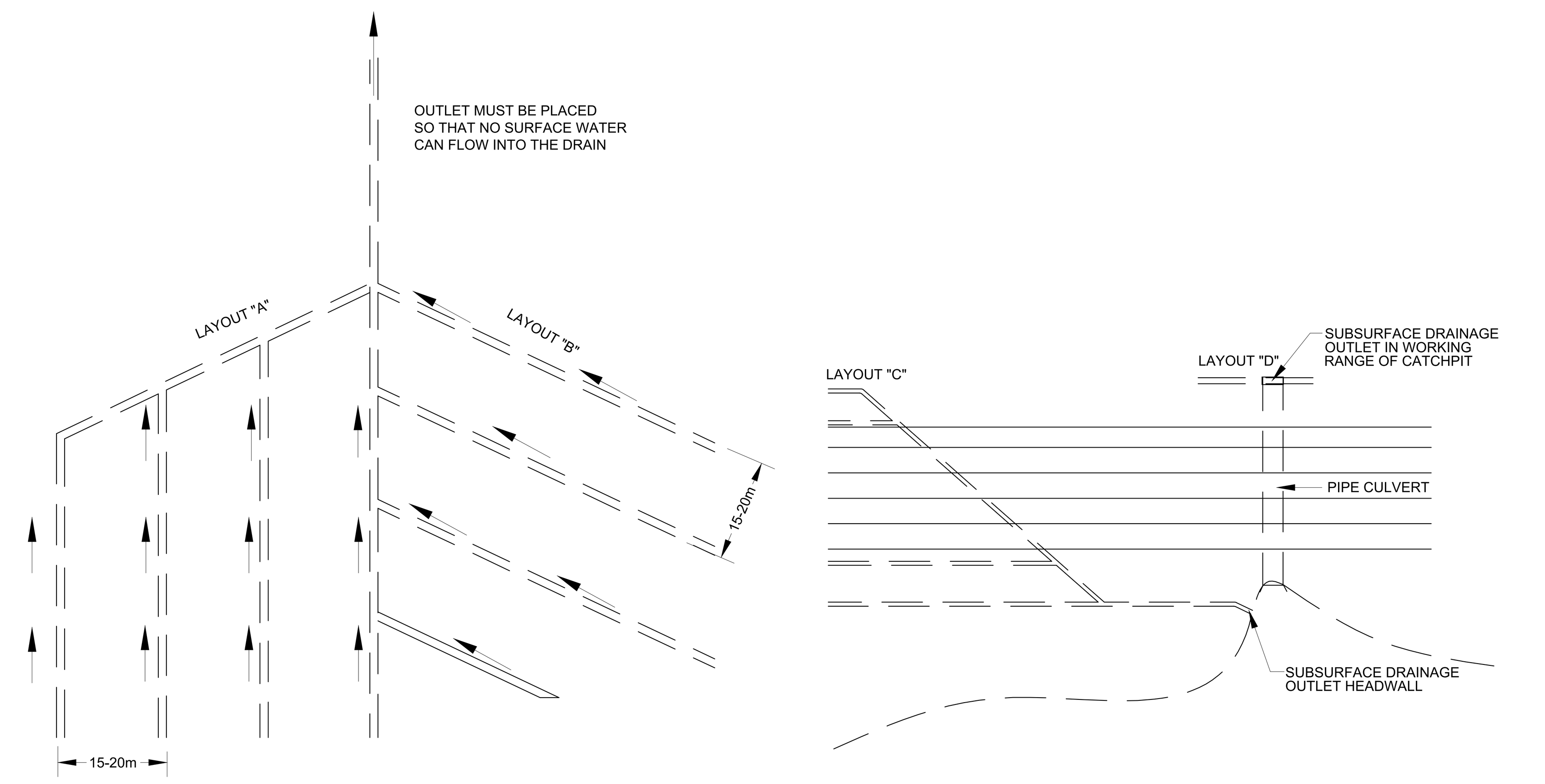
DATE/DATUM: MAY 2013
SCALE/SKAAL: AS SHOWN

SCALE REF. / SKAAL VERWYS.

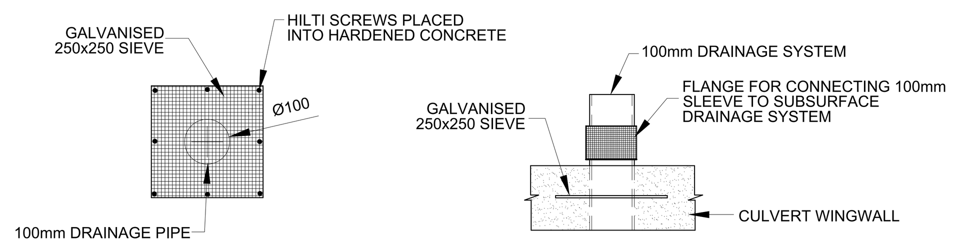
CONSULTANT DWG. NO. / KONSULTANT TEK. NR.

N2472SW3001

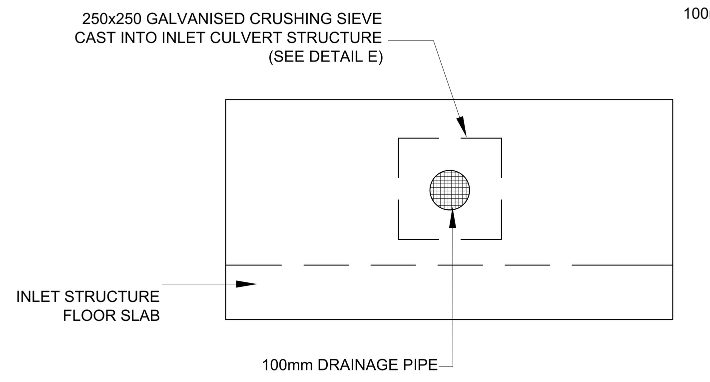
CLIENT DWG. NO. / KLIËNT TEK. NR.



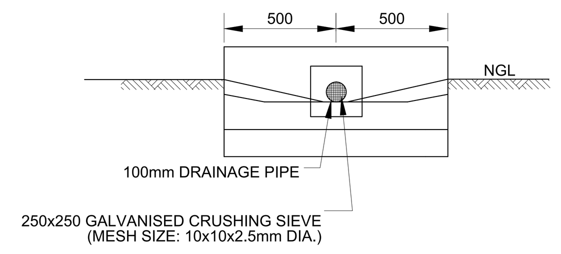
SUB SURFACE DRAINAGE LAYOUT UNDER ROAD LAYERS
SCALE 1:10



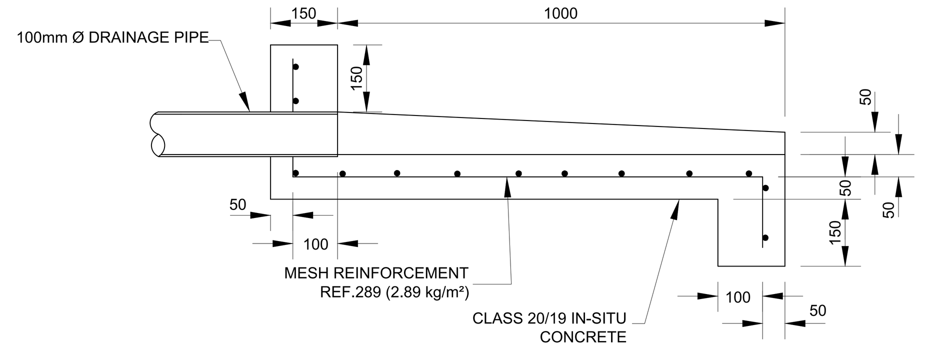
DETAIL E
SCALE 1:10



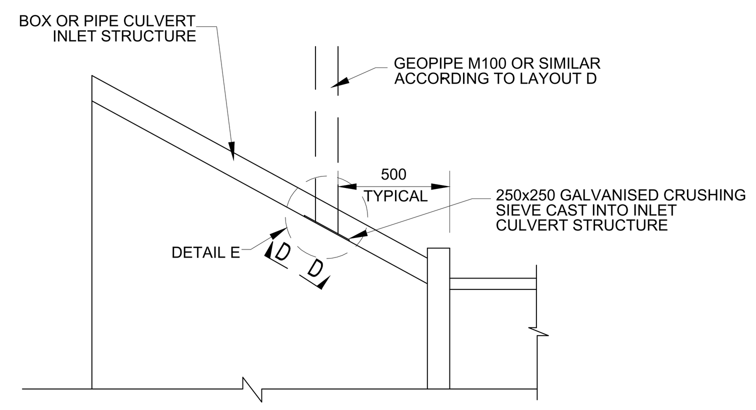
ELEVATION D - D
SCALE 1:10



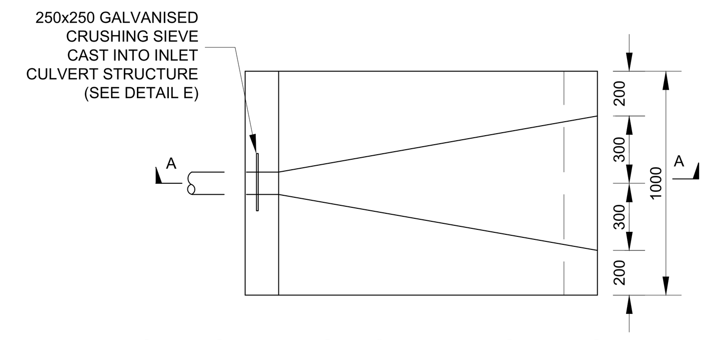
ELEVATION
SCALE 1:20



SECTION C - C
SCALE 1:10



PLAN : SUBSURFACE DRAINAGE OUTLET IN CULVERT WINGWALL
SCALE 1:20



PLAN: SUB SURFACE OUTLET STRUCTURE
SCALE 1:20

LEGEND

- IMPERMEABLE BACKFILL
- NATURAL PERMEABLE SAND FROM EITHER APPROVED SOURCES ON SITE OR COMMERCIAL
- GRADED CRUSHED STONE (FINE GRADE)
- GRADED CRUSHED STONE (COURSE GRADE)

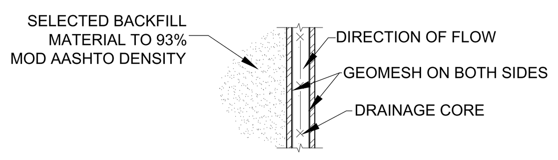
NOTES:

1. UNLESS OTHERWISE SPECIFIED, TYPICAL SUBSOIL DRAINAGE MUST BE APPLIED THROUGHOUT (TYPE 2)
2. WHERE A SUBSURFACE DRAINAGE CONDUIT IS PLACED IN A STORMWATER BEARING SIDE DRAIN, IT MUST BE PROTECTED BY LINING THE SIDE DRAIN WITH A CONCRETE LINING /STONE, PAVING/CORBELS OR SLURRY PAVING
3. FLOWNET AND PVC PIPE ONLY TO BE INSTALLED AT THE ENGINEER'S REQUEST
4. THE MINIMUM DEPTH OF SUBSOILS SHOULD BE AS DIRECTED BY THE ENGINEER ON SITE
5. USE BIDUM U24 IF SUBSOIL DRAIN IS TO BE SUBMERGED BY GROUNDWATER, OTHERWISE USE KAYTECH BSP 262 GEOMESH OR SIMILAR APPROVED
6. DESIGN 1 AND 2 CAN BE EXCHANGED WHEN INSTRUCTED BY THE ENGINEER

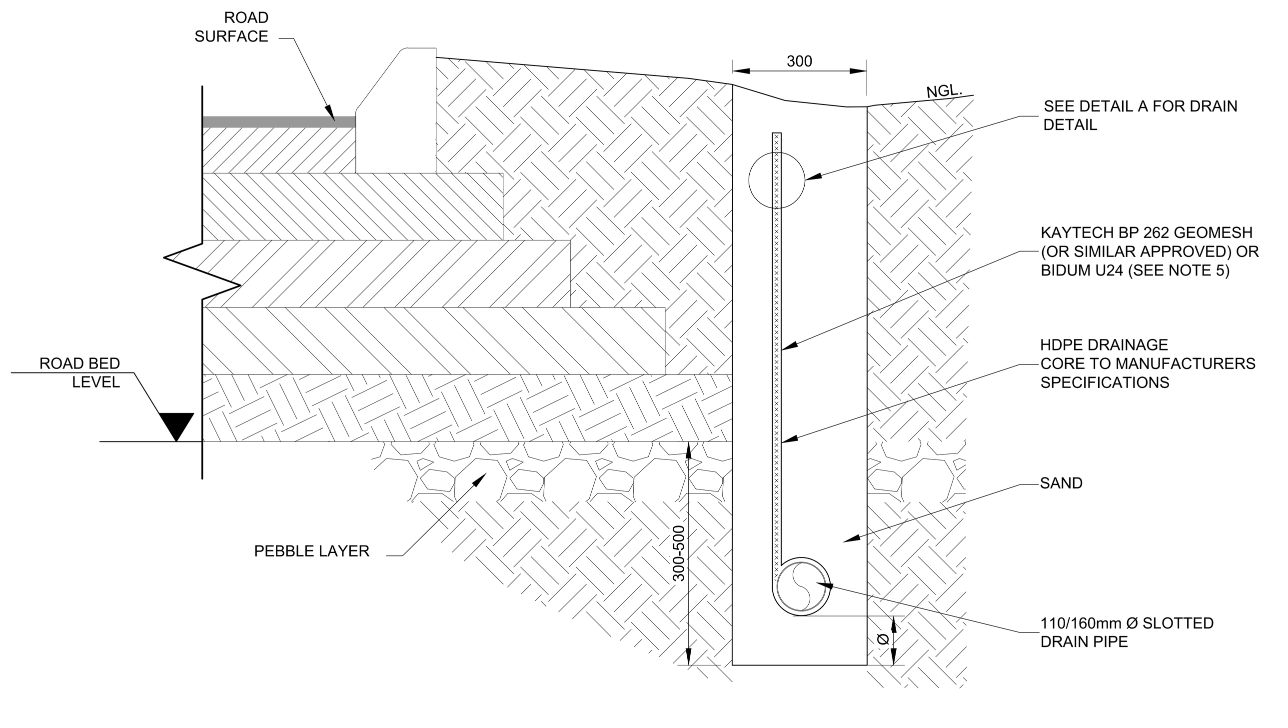
GRADING REQUIREMENTS OF CRUSHED STONE

- CRUSHED STONE SHALL BE GRADED
- CRUSHED STONE, WHICH CONFORMS TO THE FOLLOWING REQUIREMENTS:
- PERCENTAGE PASSING THROUGH A -

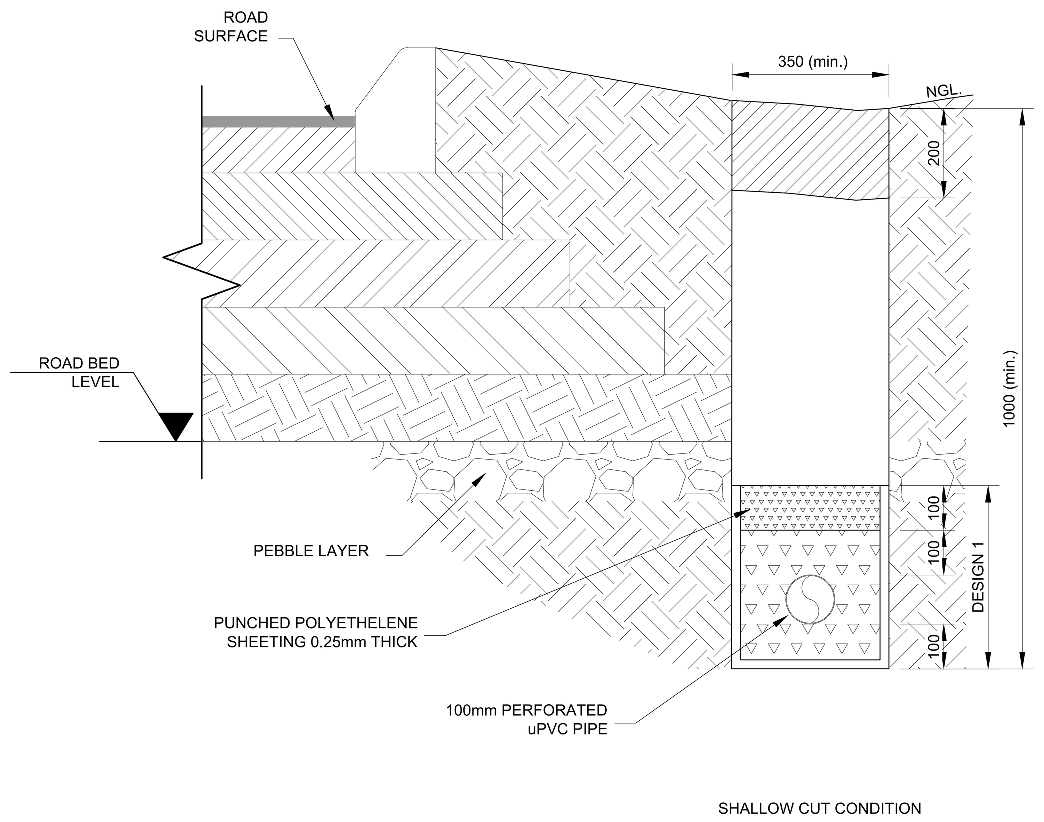
  26.6mm SIEVE.........................100
  13.2mm SIEVE......................60-85
- FINE GRADE STONE :
  A MINIMUM OF 15% PASSING THROUGH A 3.35mm SIEVE.
  A MAXIMUM OF 15% PASSING THROUGH A 1.18mm SIEVE.
- COURSE GRADE STONE :
  A MINIMUM OF 15% PASSING THROUGH A 6.70mm SIEVE.
  A MAXIMUM PF 15% PASSING THROUGH A 2.38mm SIEVE.
- IN THE CASE OF CRUSHED STONE NOT MORE THAN 5% OF THE MATERIAL SHALL PASS THROUGH THE 0.075mm SIEVE.



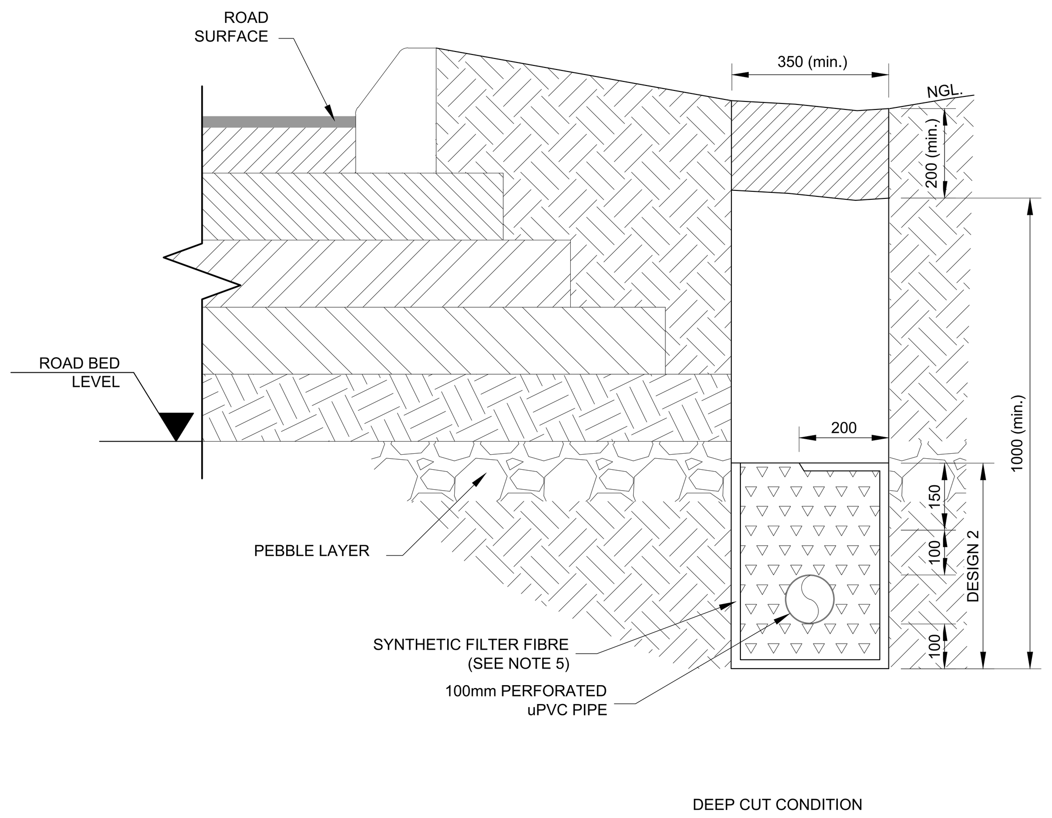
DETAIL A: DRAIN DETAIL
SCALE 1:5



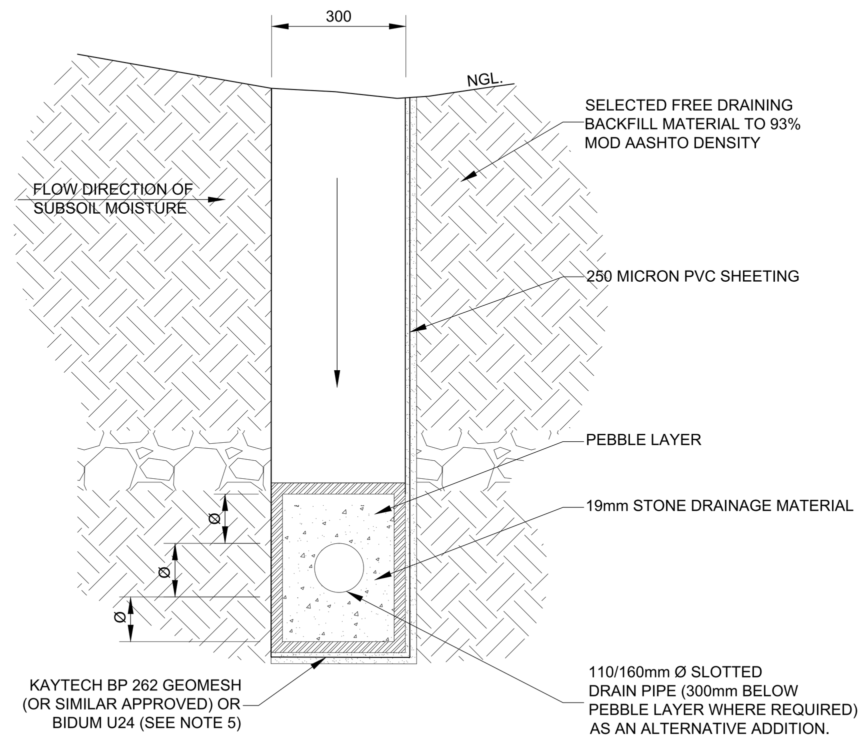
TYPICAL FIN-DRAIN DETAIL
SCALE 1:10



TYPE "1" SUB SURFACE DRAIN (SANDY MATERIAL)
SCALE 1:10



TYPE "2" SUB SURFACE DRAIN (SANDY MATERIAL)
SCALE 1:10



FLOW CUT-OFF SUBSOIL DRAIN
SCALE 1:10

FOR INFORMATION

(REF FOLDER: X:\02 PROJECTS\2000\2400 - 2499\N2472 - RICHARDSBAY IDZ ALTON NORTH PHASE 1F\1 DESIGN PHASE\F DRAWINGS\1 BASIC PLANNING\C STORMWATER\N2472SW3002.DWG)

No	DATE	AMENDMENTS	HEAD OF DEPT.	DATE

DEP DIR: TECH. SERVICES

DATE

ENDECON UBUNTU (PTY) LTD ENGINEERING CONSULTANTS
P.O. BOX 6215
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Fax:(013) 755 4904

DESIGNED BY	A.J. NEL
CHECKED BY	A.J. NEL
DRAWN BY	JM. VENTER
CHECKED BY	J.L. VENTER

DESIGN APPROVED — CONSULTING ENGINEER — DATE:

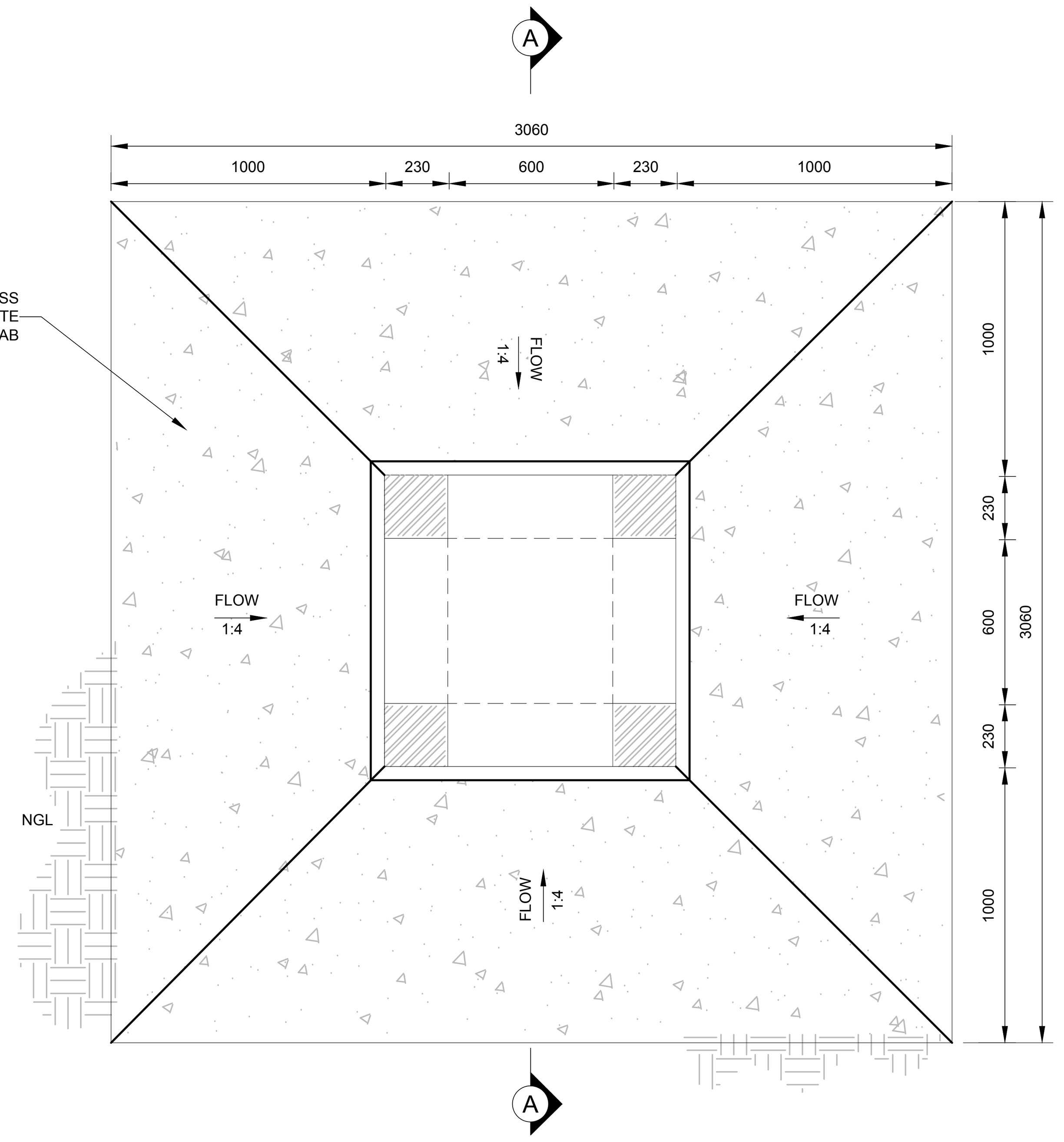
APPROVED BY — HEAD OF DEPARTMENT — DATE:

PROJECT : RICHARDSBAY IDZ ALTON NORTH PHASE 1

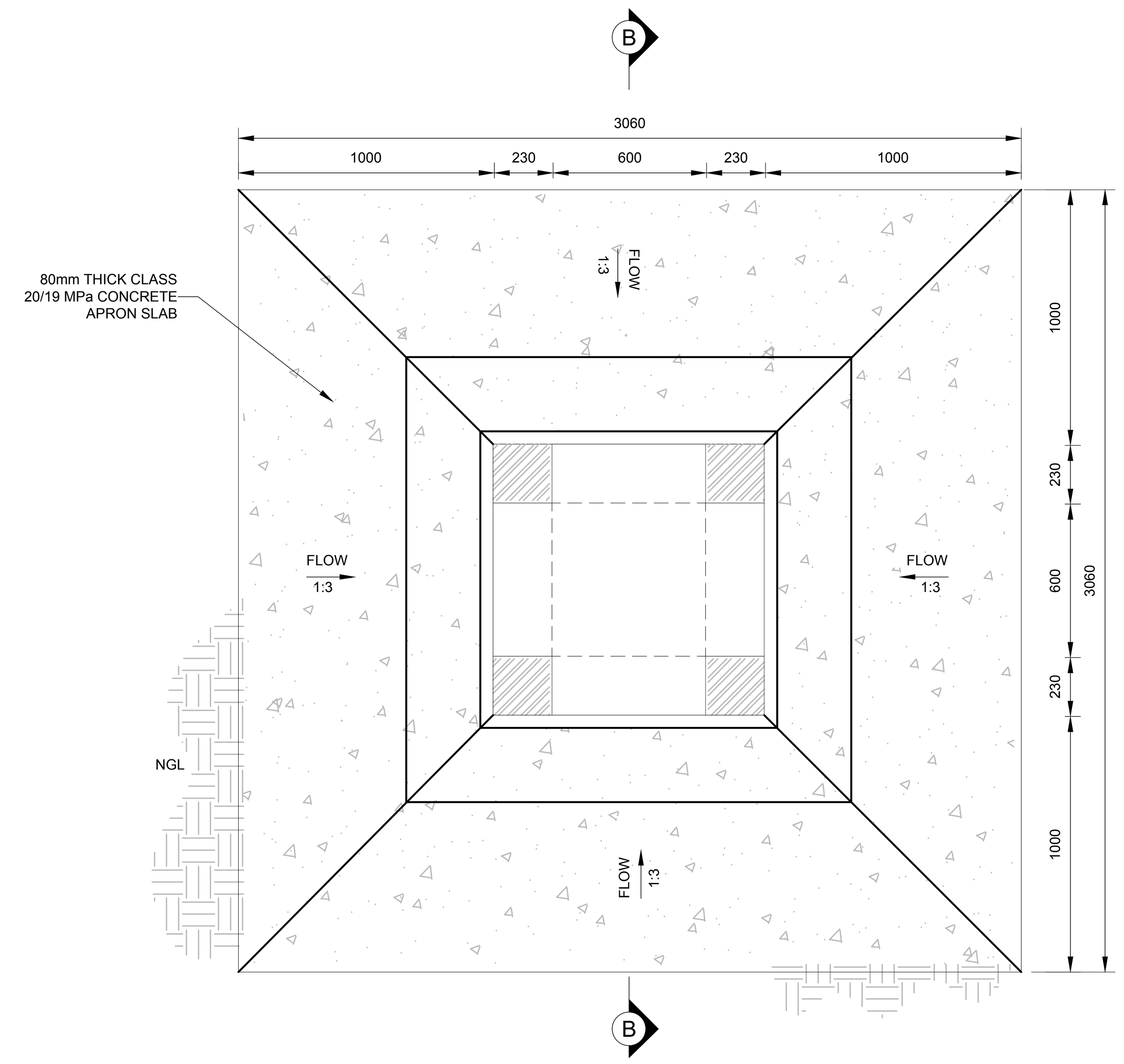
TITLE : STORMWATER - TYPICAL DETAILS

SCALE	FILE No	DISTRICT	TYPE OF DESIGN
1 : 10	N/2472/SW/3/002	RICHARDSBAY	DETAILS
	AGREEMENT No	ROAD No	PLAN No
			N/2472/SW/3/002

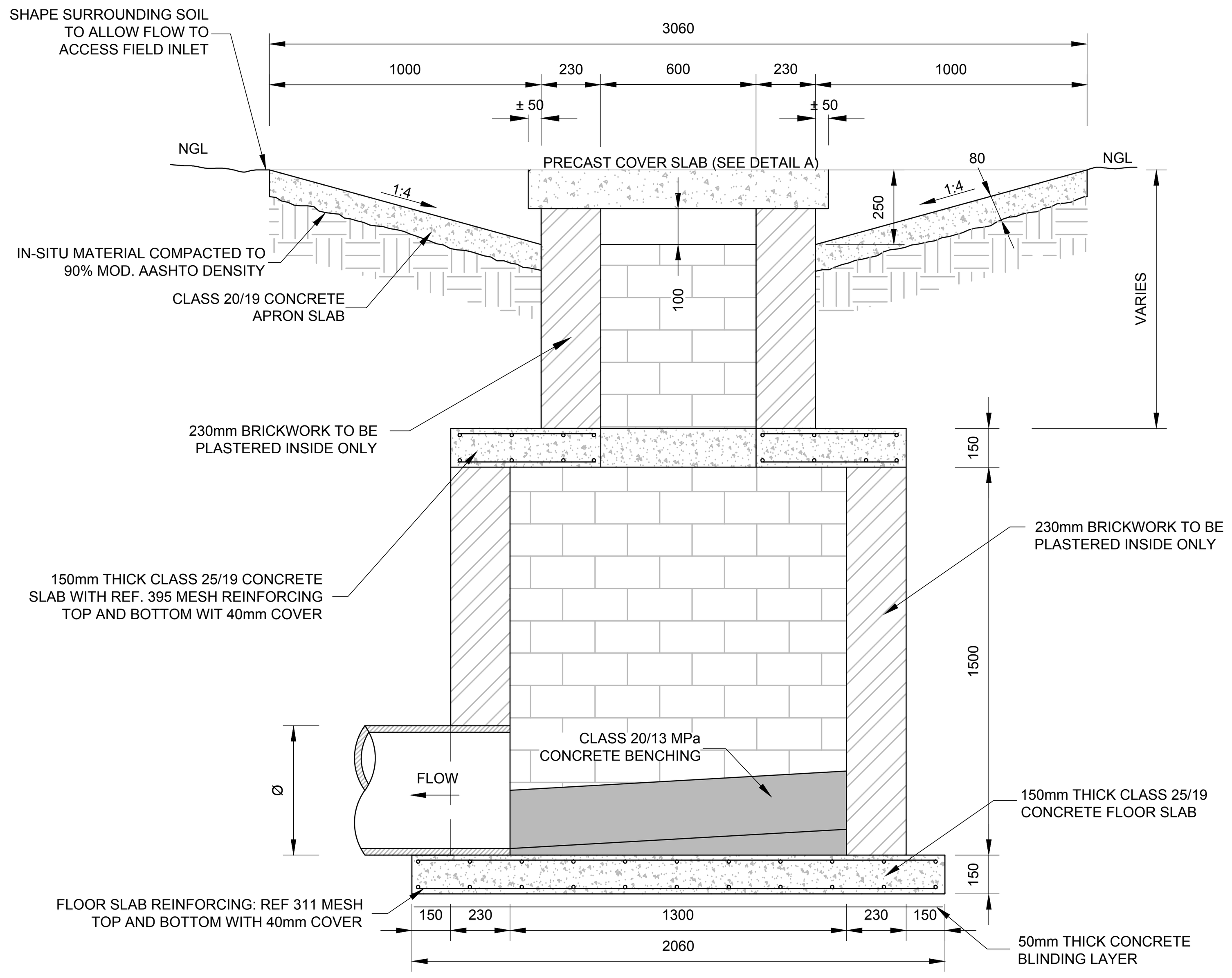
TEK.Nr.
DWG.No.
N/2472/SW/3/002



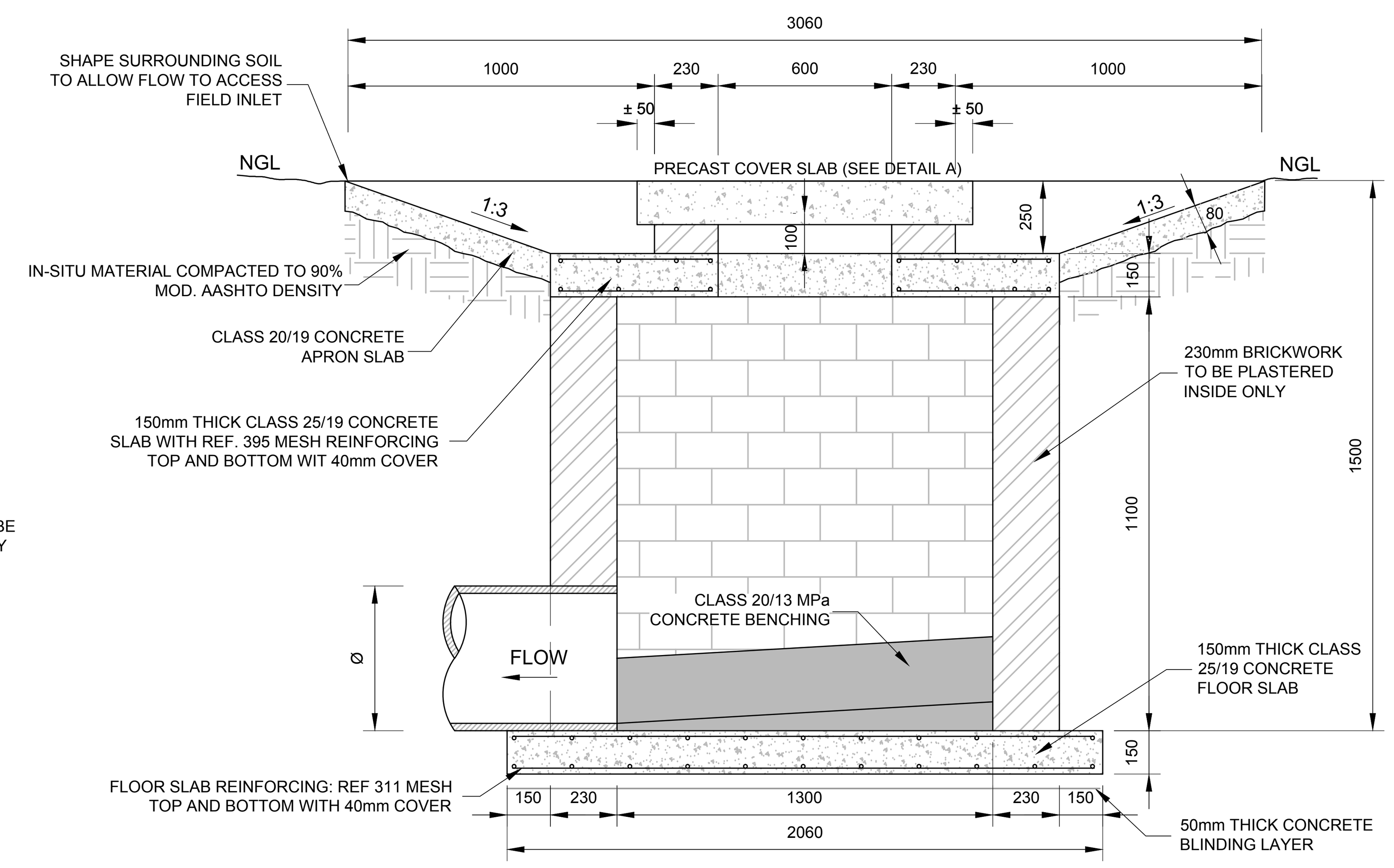
PLAN: FIELD INLET TYPE 1
SCALE 1:20



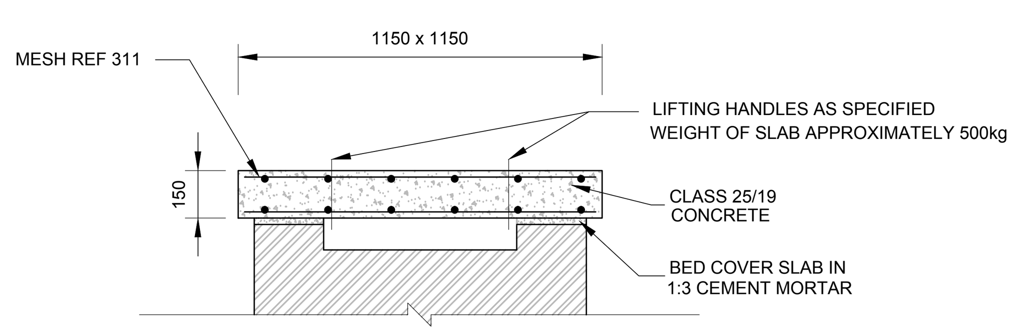
PLAN: FIELD INLET TYPE 2
SCALE 1:20



SECTION A-A:FIELD INLET TYPE 1
SCALE 1:20



SECTION B-B:FIELD INLET TYPE 2
SCALE 1:20



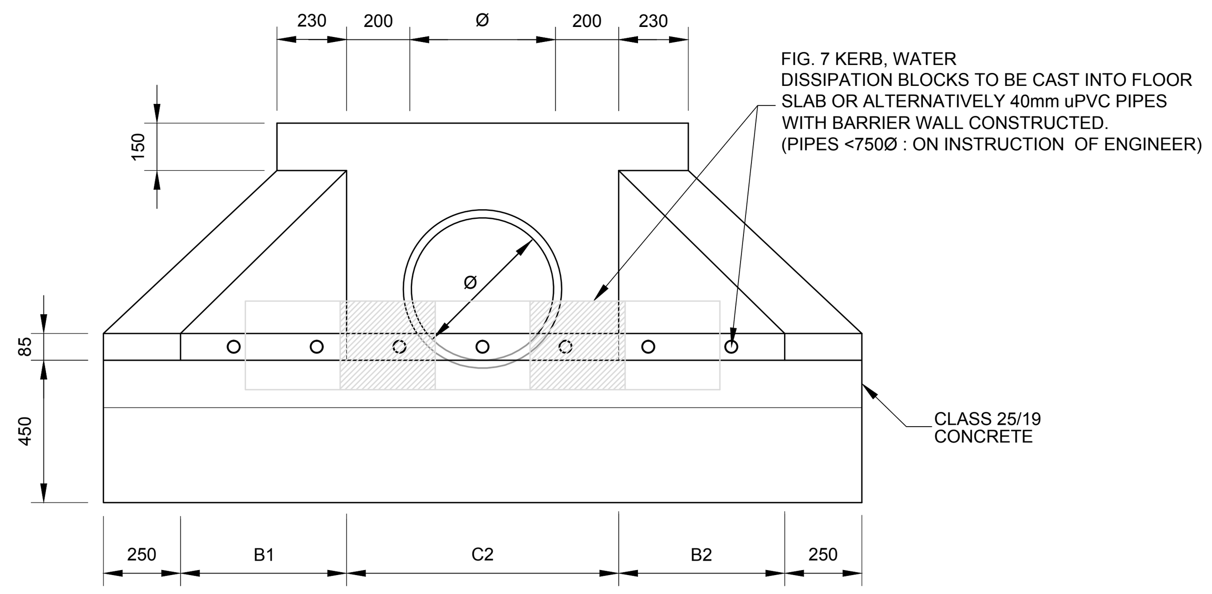
DETAIL A: OF PRECAST COVER SLAB
SCALE 1:20

FIELD INLET DETAILS

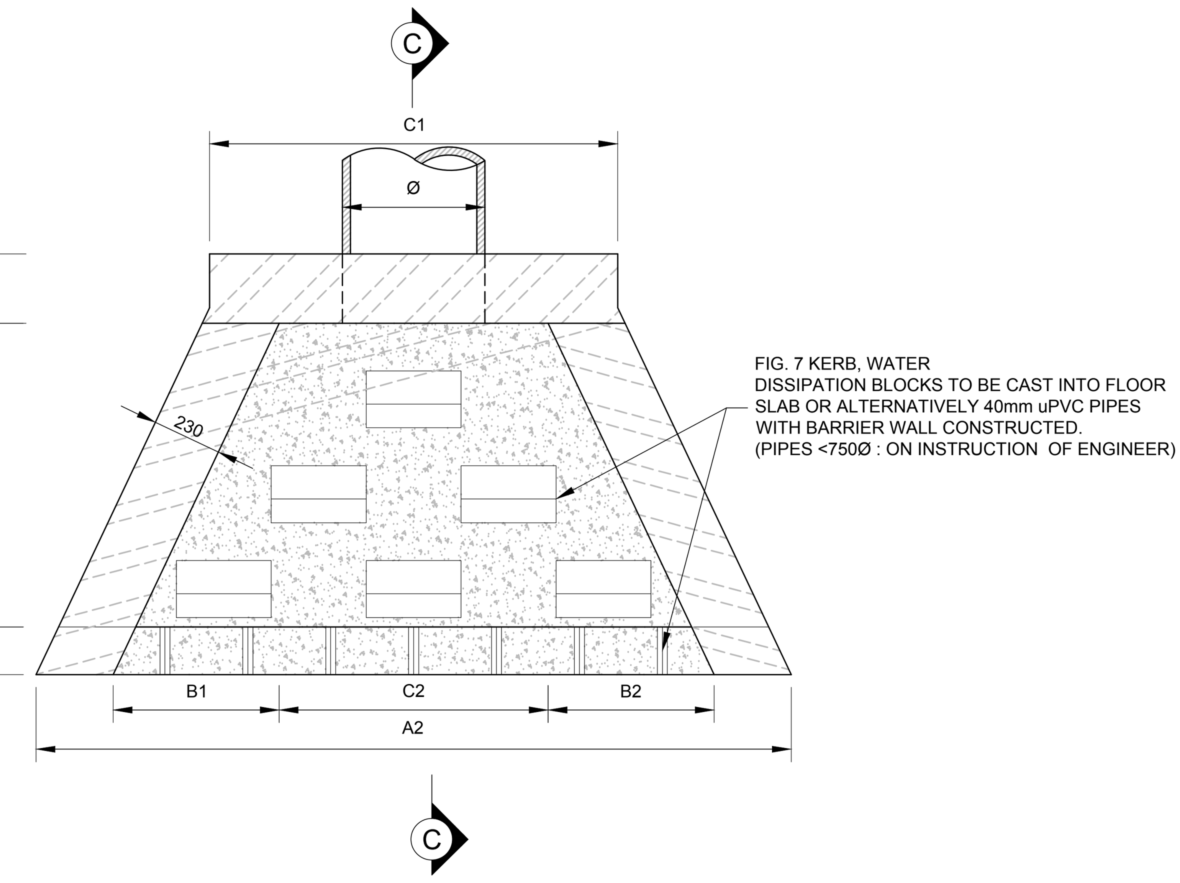
INLET OR OUTLET DIMENSION					
PIPE NOMINAL DIAMETER	450mmØ	600mmØ	900mmØ	1050mmØ	1200mmØ
A1	1000	1300	1900	2200	2500
A2	2505	3001	3994	4490	4987
B1	1021	1270	1766	2014	2262
B2	1021	1270	1766	2014	2262
C1	1350	1500	1800	1950	2100
C2	888	1038	1338	1488	1638

NOTE: THE DIMENSIONS FOR THE ABOVE NEEDS TO BE CONFIRMED PRIOR TO CONSTRUCTION OF ANY OUTLET OR INLET STRUCTURES

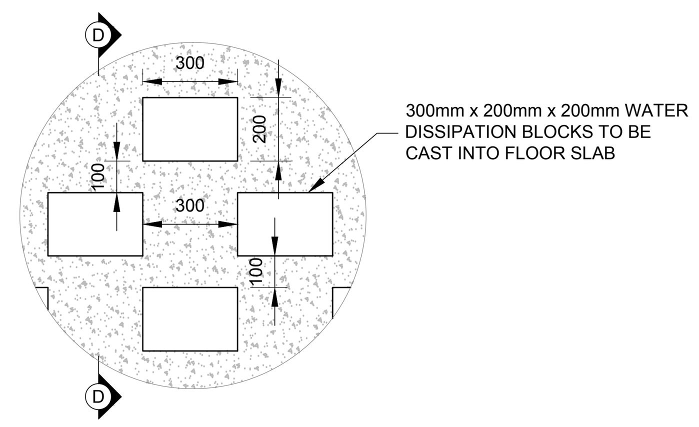
ALL DIMENSIONS IN MILLIMETERS UNLESS OTHERWISE STATED



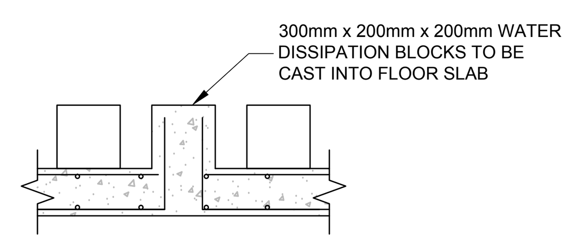
ELEVATION
SCALE 1 : 20



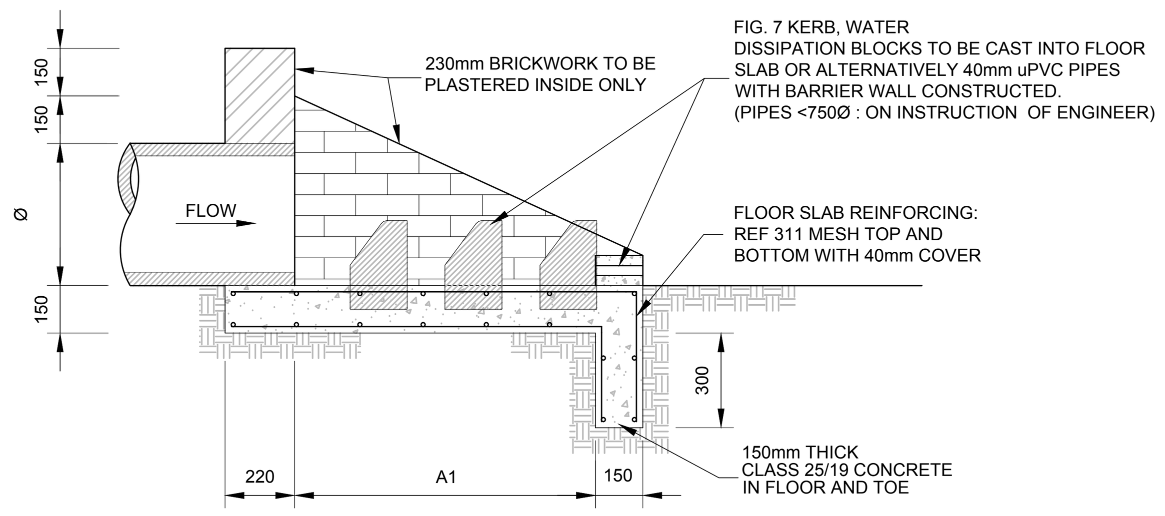
PLAN: ALTERNATIVE 1
SCALE 1 : 20



PLAN: ALTERNATIVE 2
SCALE 1 : 20



SECTION D-D: OUTLET STRUCTURE ALTERNATIVE 2
SCALE 1 : 20



SECTION C-C: IN- OR OUTLET STRUCTURE ALTERNATIVE 1
SCALE 1 : 20

FIELD OUTLET DETAILS (OUTFALL B)

(REF FOLDER: X:\02 PROJECTS\2000\2400 - 2499\N2472 - RICHARDSBAY IDZ ALTON NORTH PHASE 1F\1 DESIGN PHASE\F DRAWINGS\1 BASIC PLANNING\C STORMWATER\N2472SW3003.DWG)

NOTES:

1. DISSIPATION BLOCKS OR BARRIER WITH 40mm uPVC PIPES TO BE CONSTRUCTED ON OUTLET STRUCTURES FOR PIPE DIAMETER > 750mmØ AS INSTRUCTED BY ENGINEER ON SITE
2. ALL BRICKWORK TO BE PLASTERED ON THE INSIDE
3. BRICKFORCE TO BE PLACED IN EVERY THIRD COURSE
4. ALL CONCRETE BENCHING TO BE CLASS 20/13
5. BENCHING TO BE A MINIMUM THICKNESS OF 50mm
6. MINIMUM CONCRETE COVER TO REINFORCING TO BE 40mm
7. ALL CONCRETE FLOOR AND COVER SLABS TO BE CLASS 25/19 MPa CONCRETE
8. ALL EXPOSED CONCRETE EDGES TO BE ROUNDED WITH A NOSING TOOL
9. STORMWATER STRUCTURE IN ACCORDANCE WITH SABS 1200LE
10. STORMWATER INSIDE INLET TO BE CHANNELED BY CONCRETE BENCHING

FOR INFORMATION

Rev. Her.	Description Beskrywing	Date Datum	By Deur

AMENDMENTS/WYSIGING

SERVICE / DIENS

STORMWATER

DESIGNED/ONTWERP	A.J. NEL
DRAWN/GETEKEN	JM. VENTER
CAD FILE NAME/CAD LÊER NAAM	N2472SW3003.dwg
CHECKED/NAGESIEN	A.J. NEL
APPROVED/GOEDGEKEUR	J.L. VENTER

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CLIENT / KLIËNT

PROJECT / PROJEK

RICHARDSBAY IDZ ALTON NORTH PHASE 1

TITLE / TITEL

STORMWATER - DETAILS: INLET & OUTLET STRUCTURES

DATE/DATUM: MAY 2013

SCALE/SKAAL: AS SHOWN

SCALE REF. / SKAAL VERWYS.
4.0 2.0 0 2.0 4.0
3.0 1.0 1.0 3.0
meter

CONSULTANT DWG. NO. / KONSULTANT TEK. NR.
N2472SW3003

CLIENT DWG. NO. / KLIENT TEK. NO.

ANNEXURE E: ROADS INFRASTRUCTURE LAYOUT AND DETAILS

ANNEXURE F: RAILWAY INFRASTRUCTURE LAYOUT AND DETAILS

ANNEXURE G: GEOTECHNICAL INVESTIGATION APPENDIX

Appendix A

Appendix B

Appendix C