# *PROPOSED DEVELOPMENT ON ERVEN 103 AND 104, WITTEDRIFT, PLETTENBERG BAY*

## *PRELIMINARY DESIGN REPORT FOR THE PROVISION OF CIVIL ENGINEERING INFRASTRUCTURE*


**REV 2 - APRIL 2020**

**VOLUME 1**

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# *Pre Amble*

**In Terms of Engineering Advice and Services Quality Management System (ISO) this report complies with:**

- QD20a
- QD20b
- QD20c

And was compiled in terms of:

- QP B201
- QP B405

Which included the following actions:

- ✓ Identify type of report to be prepared (letter, memo, TIA, report stage, technical specification, etc.)
- ✓ Select appropriate pro-forma or similar report previously prepared
- ✓ Prepare report in accordance with relevant guidelines and brief
- ✓ Word spell-check and grammar check report
- ✓ Compile drawings in accordance with QP B405
- ✓ Report reviewed by partner(s)
- ✓ Make necessary amendments
- ✓ Generate copies for submission and one spare copy for office library
- ✓ Author and Reviewer sign quality control fly-sheet

| **Comments** | **Name** | **Date** |
|---|---|---|
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*Chapter 1*
# *INTRODUCTION*

## 1.1 TERMS OF REFERENCE

Engineering Advice and Services (Pty) Ltd were appointed as Civil Engineers by Mr Lance del Monte, the Project Director of The Home Market, for the proposed development of a retirement village on erven 103 and 104 Wittedrift in Ward 1 of Bitou Municipal area. It is envisaged that the development will comprise of approximately 54 single residential units, a community centre and an assisted living area, as well as public areas for the community to use (refer to Locality Plan - **Dwg 1692-R-000**).

The proposed infrastructure will include a water reticulation network with metered erf connections, a waterborne sewerage system, a combination of an overland and piped stormwater system, as well as a road network.

This report serves as a base for the Client, the Bitou Municipality, and other professionals to use as input for the detail design process and to enable decision-making with regards to the infrastructural standards and layouts. The report will be supplemented by detailed designs once the servicing concepts contained in this preliminary design have been finalized and approved.

As per requirements of Bitou Municipality, the layout was submitted to GLS Consultants, who provided feedback in April 2020 as to suitable bulk water and sewer servicing and associated costs. Similarly, bulk stormwater infrastructure has been designed in accordance with the Stormwater Masterplan for Green Valley and Wittedrift compiled for the Bitou Municipality by Nadeson Consulting Services (Pty) Ltd [E008/OD/001 rev 0 dated April 2016]. Extracts from these reports have been incorporated in this report and are also included in the Annexures.

Although EAS has prepared this document and the associated drawings to a high standard and detail, it must be understood that, at this juncture, many assumptions have been made and engineering judgment calls where employed to arrive at the presented solutions. Each of the solutions require detail designs before finality on anything is reached.

## 1.2 PRIMARY OBJECTIVES

The primary objective of this preliminary study is to devise and propose ***a combination of appropriate engineering servicing systems*** which are fit for purpose for this particular development. More specifically, the preliminary design will concentrate on the following aspects :

- Highlighting aspects of the town planning layout that could increase servicing costs;
- Proposing appropriate civil engineering services design standards;
- Determining bulk service connection points;
- Undertaking preliminary designs and preparing drawings for the water distribution a waterborne sewerage system, as well as the roads and stormwater services ; and
- Undertaking a preliminary costing exercise of the civil engineering services, based on the information available at this juncture.

The township layout used for preliminary design purposes is as received from Erik Voigt Architects. A survey of the area has been conducted by Beacon Surveys and this will be used as the topographical base.

## *Chapter 2*
# *GEOGRAPHICAL INFORMATION*

## 2.1 LOCATION

The proposed development is located on erven 103 and 104, Wittedrift. The site is bounded by Main Road to the South, Kammassie Street to the West, Protea Street to the East and bisected by Rotterdam Street which runs East to West between erf 103 and erf 104. At this early stage of the report, it is important to mention that the intention is to close Rotterdam Street, which will be advertised with the proposed consolidation of erven 103 and 104.

## 2.2 TOPOGRAPHY, GEOLOGY AND VEGETATION

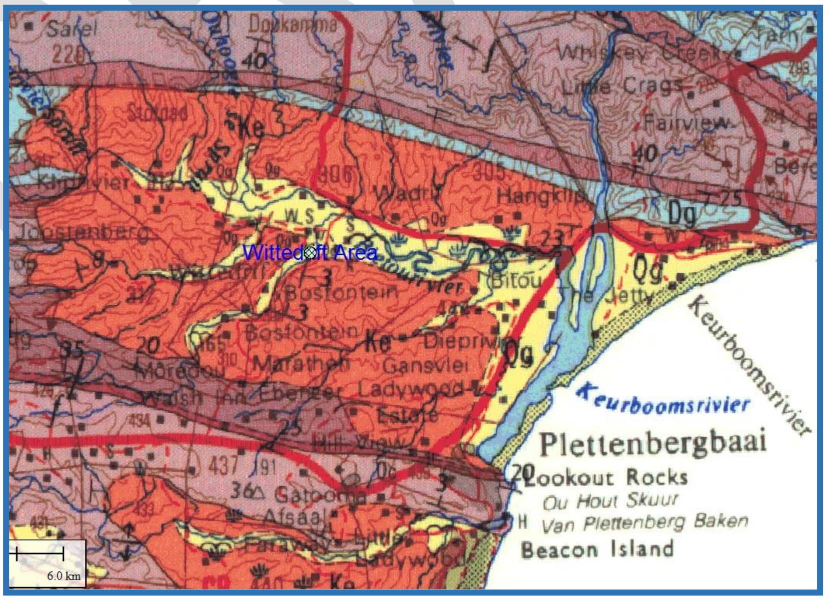
The topography of the two erven is distinctly different. Erf 104 slopes moderately from the South (Mainf Road) towards Rotterdam Road. Erf 103 again falls steeply towards the north, with the Bosfontein River at the northern boundary of this property. A floodline study has been carried out by Fraser Consulting Civil Engineers and the theoretical position of the 100 year floodline has been used as the limit of development on erf 103. There are no structures at present on either of the erven. There are however existing services crossing erf 104 that will need to be relocated.

A detailed geological investigation of the site has been conducted by Outeniqua Geotechnical Services, results of which were received on 10 February 2020. (Trial Holes positions and results are shown on **Dwgs 1692-R-002** to **1692-R-004**). The soil within Green Valley and Wittedrift is primarily cohesive clayey material of a dark red colour.

A vegetation study was not done but it is clear that the area is disturbed. Erf 104 is covered predominantly with grass and has a limited amount of trees on the south-western part of erf 104. Erf 103 has significant bush and trees which will generally remain undisturbed.

The geological map of Oudtshoorn (sheet no. 3322, scale 1:250 000) shows the site to be underlain by marine and estuarine terrace gravel and sand, partly calcareous material.

An extract of the local geology is shown below.




## 2.3 RAINFALL

Precipitation occurs mostly in the form of frontal systems. The Mean Annual Precipitation (MAP) value is about 650 mm, with rainfall occurring throughout the year. It receives the lowest rainfall (50mm) in June and the highest (72mm) in October. Weinert's climatic N number for the Plettenberg Bay area is approximately 2, implying that chemical weathering dominates over mechanical weathering.

***Chapter 3***

# *WATER SUPPLY & RETICULATION*

## 3.1 GENERAL

Potable water is essential for humans to live. Various options for supplying this resource are available and this includes rainwater tanks, collection from rivers, boreholes, etc. It has, however, become accepted practice in most first world countries to deliver treated water to a consumer via a water reticulation network, installed underground, to their home. A metered water supply will therefore be provided for each site. The following section deals with the proposed standards and includes present situation information as supplied by GLS Consulting.

## 3.2 PRESENT SITUATION [as per GLS Consulting Report to Bitou Municipality]

### 3.2.1 RETICULATION NETWORK

The existing 75 mm Ø pipe is currently supplied under gravity via the Wittedrift reservoir at a Top Water Level (TWL) of 70.0 m.a.s.l. The existing water reticulation system has sufficient capacity to accommodate the domestic flow of the proposed development in order to comply with the pressure criteria as set out in the master plan.

Upgrading of the existing reticulation network is however required to comply with the fire flow criteria. The capacity of the existing system to supply fire flow to the development at 10 m head is only 10,0 l/s.

### 3.2.2 RESERVOIR CAPACITY

The criteria for total reservoir volume used in the Bitou Municipality Water Master Plan is 48 hours of the AADD (of the reservoir supply zone). The existing reservoir volume available at the Wittedrift reservoir is 90 hours of the total AADD and is therefore sufficient to accommodate the proposed development.

## 3.3 DESIGN STANDARDS

The *"Guidelines for Human Settlement Planning and Design" (Red Book)* as compiled by the CSIR on behalf of the Department of National Housing in 2000 was used as a base for the preliminary design, and the water demand figures supplied by GLS Consulting were adapted as the estimated demand for the proposed development. The proposed development falls within future development area P062 of the Water Master Plan

A preliminary water reticulation layout is included as **Dwg 1692-R-100**, and has been designed based upon the following standards :

**(a) Water demand**

For residential sites the design will be based on *an average daily demand of **600 liters/erf/day** for 60 equivalent residential sites [as per GLS Consulting water demand figures], with an estimated 3 people per unit, giving a total AADD of 36 kl/day. For the requirements of fire demand, GLS provided a flow criteria (low risk) of 15 l/sat 10m.*

**(b) Residual Pressures**

The design is based on Table 9.17 of the 'Red Book" which calls for a minimum theoretical residual head of 24 meters under instantaneous peak demand and a maximum head of 90 meters under zero flow conditions for the residential units.

The network will have to be sized to accommodate residual pressures and it is suggested that a computer modeling programme like the Aquanet module of Civil Designer, be used to verify that the design at any point in the reticulation system under instantaneous peak flow conditions provides an acceptable solution. It is suggested to specify flexible water pipes and to limit velocities to 1,5 m/s in the pipes.

A further objective should be to, as a conservative benchmark, limit the maximum head in the system to 60 meters, as it puts less strain on the water distribution network.

EAS have run the Aquanet computer simulation to produce a base from which the detail designs can be undertaken (to be included later as **Dwg 1692-R-104,** which will indicate pipe and node ID's). Four scenarios will be tested, namely: maximum pressure and maximum velocities, both with and without fire flow. The results of these analyses will be included in **Annexure A as Tables 1 through 4.** Preliminary calculations show that a feasible and economic solution will be possible through detail design.

**(c) Isolating Valves**

The guidelines suggest that valves should be placed so that a maximum of 4 valves need to be closed to isolate a section of a suburb. We believe that the spacing is of no relevance as long as the service provider has the capacity to provide emergency repair services in an acceptable time frame. The size of the area to be isolated is determined purely by convenience and cognizance should be taken that cutting in extra valves in the future is possible.

It is proposed that valves on 75mm diameter pipe should be wedge gate type, Class 16, left hand closing with cap tops, manufactured in accordance with SABS 664-1989. If unavoidable on 50mm HDPe lines Cobra 1001/125 valves complete with "T" bar handles are proposed, but are not preferred.

**(d) Pipe Materials**

It is commonly accepted that the following materials are an appropriate product:

- For pipelines of 75mm diameter, we propose PVC-U SABS 966 class 9 (to suit the existing reticulation to serve the development.
- For pipelines of 50mm diameter we propose HDPe, ISO 4427, PE63, PN 10 to serve the development.

(Pressure reducing valves should be installed to limit all pressures to 60 m and below)

**(e) Erf Connections**

It is proposed that each site be provided with a metered water erf connection. Double erf connections should be used, where possible, to effect cost savings. It is important to realize that material prices vary and that the connections should be designed using current prices. Experience has taught us that by determining the prices of pipes and fittings at design stage and using appropriate combinations, a major cost saving can be effected without compromising quality. Details will be confirmed when the preliminary design is finalised, but at this juncture it is anticipated that HDPE pipes with compression fittings will be used. Typical details are included (See **Dwg 1692-R-102**).

**(f) Protective Cover to Pipes**

A significant cost which forms part of the project is the alternative chosen to protect water pipes, fittings and internal plumbing. Care should be taken in the design of this mechanism. For this project we propose the following:

- Communication pipes, in 300mm wide trenches, should be installed at a depth of between 450mm and 600mm,
- In road reserves, pipes should be installed at a depth which ranges between 740mm and 1000mm,
- In trafficked areas, pipes should be installed at a depth which ranges between 1000mm and 1250mm.

Various trench widths are also proposed with the selection of protective material differing for different pipe types and loading areas (see **Dwg 1692-R-101**).

**(g) Fire Hydrants**

**For individual Erven**
It is suggested, in line with “Red Book” standards, that hydrants be positioned on water lines with diameters not less than 75 mm and that they not be placed further than 180 m apart. The positioning should further be so determined that they are placed in vehicular thoroughfares and opposite erf boundaries (see **Dwg 1692-R-100**).

Using Table 9.20, of the “Red Book”, as a base the design must ensure that any hydrant can deliver a minimum flow rate of 500 l/min with a residual pressure of 6m. Fire hydrants should be underground screw-down type with an overall maximum height of 320 mm and rising spindle. The hydrant must open by rotating the spindle in an anti-clockwise direction. The outlet must be of the London Round Thread type. The hydrant must further conform to SABS 1128: Part 1-1977. Hydrants will be positioned in consultation with the Bitou Municipality. Based on the layout, preliminary calculations show that one hydrant placed near the entrance will provide sufficient coverage for the development.

**(h) Location of Pipes**

Water pipelines should generally be positioned 2,5m away from erf boundaries unless otherwise specified. For this layout a deviation from the 2.5 m will in all likelihood be required. The exact positions will be finalised once road reserves have been defined and accepted (see **Dwg 1692-R-100**).

**(i) Corrosion Protection**

The proposed services should be protected in an appropriate fashion. It is proposed that any cast iron fittings be Fusion Bonded Epoxy (FBE) coated or cement mortar lined by an acceptable service provider. All bolts should be stainless steel and any non-plastic, metallic connections should be protected with denso tape. Galvanizing should not be used.

**(j) Connections of various Pipes**

Included as **Dwg 1692-R-102** are suggested connection methods at the various nodes as well as node details indicating change of direction and valve and hydrant points.

*Chapter 4*

# *SEWER NETWORK*

## 4.1 General

Various sanitation systems are available to the consumer, but it has been found that a waterborne system is the most appropriate for higher densities of development.

An existing 110mm Ø uPVC sewer extends down Protea Street and the most suitable sewer connection for the retirement village is into this pipeline near the intersection of Protea Streat and Rotterdam Road. At present, this sewer only carries effluent from two properties at this point (erven 90 and 91 Wittedrift). There is also an existing 110mm Ø uPVC sewer extending from erf 104 into the sewer in Protea Street, which carries no flow and will be abandoned.

Various options were considered to provide sewerage infrastructure to the proposed development. Based on ground based survey information, it was found that it is was not possible to provide a full gravity sewer system to serve the entire development based on existing ground levels.

This left two options, namely either a pumped system (which would have to be maintained by the residents) or shaping of the site to suit drainage of the properties. This second alternative was considered the most suitable option, primarily requiring shaping of a portion of erf 103 back towards the proposed roadway. The preliminary sewer design included in Volume 2 is based on a gravity sewer system to serve the development.

A layout indicating the proposed sewer network for the entire development is included as **Dwg 1692-R-200**, but will only be finalised during the detail design stage of the project. Typical sewer and erf connection details are included as **Dwg 1692-R-201**.

## 4.2 Present Situation [as per GLS Consulting Report to Bitou Municipality]

### 4.2.1 Drainage Area

The development falls within the existing Wittedrift pumping station (PS) drainage area. The proposed connection point for the development is into the existing 110 mm Ø sewer in Protea Street.

Sewage is currently pumped from the Wittedrift PS to the Aventura PS through a 125 mm Ø rising main. From the Aventura PS sewage is pumped through the 200 mm Ø rising main to the Ganse Valley Wastewater Treatment Plant (WWTP). The development is inside the sewer priority area.

### 4.2.2 Sewer flow

The sewer analysis for the master plan was done with a peak day dry weather flow (PDDWF) for the proposed development area on erven 103 & 104 (future development area P062 in the sewer master plan) of 25,2 kl/d.

There is sufficient capacity in the existing gravity sewer system between the proposed development and the existing Wittedrift PS to accommodate the proposed development.

## 4.3 Design Standards

The following documents were used as a base for the preliminary design:

- "Guidelines for Human Settlement Planning and Design" (Red Book) as compiled by the CSIR on behalf of the Department of National Housing in 2000;
- Sewer Design Requirements (for work submitted to the NMBM City Engineer for approval) Sewerage Division, Revision 3: June 1996; and the
- Standard Details July 2007 published by the NMBM.

**(a) Grades**

The purpose of installing sewer pipes at a grade is to ensure that the sewerage will be transported by gravity. It is recognized that a velocity of 0,7 m/s will produce sufficient energy to ensure that a cleansing velocity is achieved in the pipelines. It is thus suggested that pipes be designed and laid at grades to ensure this velocity is attained under PWWF. A minimum grade of 1:200 has been used in the preliminary design, which allows for gravity reticulation to the bulk sewerage connection point. Gradients can possibly be improved by lifting the south eastern portion of erf 104 Wittedrift.

Erf connections should be installed at a minimum grade of 1:60.

Indicative long sections of the sewer design are included as **Dwg's 1692-R-202 through 1692-R-203**.

**(b) Protective Cover**

As is the case for the water reticulation, a significant part of the project cost is determined by the alternative chosen to protect your pipes. Care should be taken in the design of this mechanism. For this project we propose the following:

- In mid blocks or road reserves, pipes should be installed at a minimum depth of 800mm,
- In trafficked areas, pipes should be installed at a minimum depth of 1000mm.
- Erf connections should be installed at a minimum depth of 450mm.

Trench widths are also proposed with the selection of protective material differing for different pipe types and loading areas (see **Dwg 1692-R-201**).

**(c) Pipe Diameters**

Pipe diameters have been sized to be the same (minimum) of 110mm to tie into the existing sewer reticulation. Flow calculations show that this diameter is adequate to cater for the flow from the development.

**(d) Manholes and Rodding Eyes**

Manholes should be spaced at maximum of 80 meters apart and at a change of direction and grade of the sewer pipe. Preference should be given to dolomitic pre-cast concrete (25 MPa/19) manholes. Standard details to consider are included on **Dwg 1692-R-201**.

Rodding eye manholes could be considered as options on the start of short lines as a cost saving initiative.

**(e) Pipe materials**

It is proposed that structured wall PVC u, Type 1, SANS 1601, 400 kPa pipes be specified.

**(f) Erf Connections**

Erf connections should be 110mm PVC u Type 1, SANS 1601, 400 kPa of the different types and configurations as indicated on the drawings. Typical details are included as **Dwg 1692-R-201**.

**(g) Location**

As a result of the town planning layout it is proposed that sewers for the individual erven mostly be installed in the road reserves at an average distances of 1.3m from erf boundaries (See **Dwg 1692-R-200**). Where this is not possible due to the building layouts, sewers are designed parallel to the proposed roadways. The exact positions of proposed sewers will be determined during detail design. Cognizance will need to be taken of a myriad of other services that need to be accommodated.

### h) Registered Servitudes

Once detail designs have been completed and accepted by the Local Authority the Land Surveyor will have to ensure that Servitudes are registered over public services as instructed by the Client and in terms of the Planning Approvals.

# *Chapter 5*
# *ROADS*

## 5.1 PROPOSED ROAD HIERARCHY & LAYOUT

The road hierarchy depends on the function that the road needs to fulfill, whether it be mobility or accessibility or various combinations thereof. Generally, this is also dictated by the town planning layout and the number of erven which are served. The table below provides a broad indication of the hierarchical functionality of the road network which is found in the proposed development.

| Road Class | Maximum Dwelling Units Served | Preferred Maximum Length |
|---|---|---|
| Local Distributors - Class 4 | 400 - 1500 | n/a |
| Residential Access Collectors - Class 5a | 200 | 500 m |
| Residential Access Loops - Class 5b | 120 | 300 - 500 m |
| Access Cul de Sacs - Class 5c and 5d | < 60 | 150 m |

The proposed road layout is indicated on the road layout plan (see **Dwg 1692-R-300**).

Typical long sections for internal roads is included (see **Dwgs 1692-R-301** through **1692-R-304**) and incorporate the road reserve and road widths proposed in the following section of this report.

## 5.2 GEOMETRIC STANDARDS

The following tables extracted from the "Guidelines for the Provision of Engineering Services in Residential Townships" (Blue Book) published by the erstwhile Department of Community Development should serve as input for the Town Planners to make decisions around the engineering guidelines that exist.

Final geometric designs should be checked using the "Turn" module of Civil Designer to ensure unrestricted vehicle access in the development. Where vehicle access cannot be achieved, appropriate signage will need to incorporated to prevent vehicle access.

### (a) Road Reserve Widths

| Road Class | Preferred | Minimum |
|---|---|---|
| Local Distributors - Class 4 | 20 m | 16 m |
| Residential Access Collectors - Class 5a | 16 m | 12 m |
| Residential Access Loops - Class 5b | 12 m | 12 m |
| Access Cul-de-Sacs - Class 5c and 5d | - | 8 m |

***Note that the road reserves for this development do not comply with the guidelines, but the Client has indicated that these should not be changed.***

**(b) Roadway Widths**

| **Road Class** | **Proposed** |
|---|---|
| Minor Collectors - Class 4 | 8.0 m |
| Local Distributors - Class 4 | 7.4 m |
| Residential Access Collectors - Class 5a | 6.2 m |
| Residential Access Loops - Class 5b | 6.0 m |
| Access Cul-de-Sacs - Class 5c and 5d | 3.0 m |

***Note that the road widths for this development do not comply with the guidelines but the Client has indicated that these should not be changed.***

**(c) Road Crossfall**

It is proposed that a standard cross fall of 2.5% be adopted on all roads. Where necessary, this may be adjusted to provide a maximum 3% or a minimum of 2%.

**(d) Bell Mouth Radii at Intersections**

| **Road Class** | Proposed |
|---|---|
| Local Distributors - Class 4 | 12.0 m |
| Residential Access Collectors - Class 5a | 10.0 m |
| Residential Access Loops - Class 5b | 8.0 m |
| Access Cul-de-Sacs - Class 5c and 5d | 6.0 m |

***Bell Mouth radii for this development do not comply with the guidelines due to the reduced road widths.***

**(e) Horizontal Alignment**

| **Road Class** | **Desired Maximum Speed (km/h)** | **Minimum Centerline Radius** |
|---|---|---|
| Local Distributors - Class 4 | Max speed : 60<br>Design speed : 50 | 150m (4% super-elevation) |
| Residential Access Collectors - Class 5a | 40 | 15m |
| Residential Access Loops - Class 5b | 30 | 15m |
| Access Cul-de-Sacs - Class 5c and 5d | 15 - 20 | 10m |

**(f) Vertical Alignment**

| Road Class | Max Gradient | Min Gradient | Min K-value | Min Vertical Curve |
|---|---|---|---|---|
| Local Distributors - Class 4 | 7% | 0.4% | 6 | 30 m |
| Residential Access Collectors - Class 5a | 10% | 0.4% | 6 | 30 m |
| Residential Access Loops - Class 5b | 12% | 0.4% | 1 | 20 m |
| Access Cul-de-Sacs - Class 5c and 5d | 10% | 0.4% | 1 | 20 m |

In order to inform the final designs long sections of the ground lines in the center of the road reserves have been extracted. These should be studied carefully by the Town Planners so that they might adapt their layout to ensure that vertical alignments can be achieved in accordance with the guidelines. The drawings depicting the latter are included as **1692-R-301 through 1692-R-304**.

## 5.3 PAVEMENT DESIGN

The pavement design can only be undertaken once the results of the in-situ CBR tests have been received. The structural design process recommended by and as detailed in the “Guidelines for the provision of Engineering Services and Amenities in Residential Township Development” (Red Book) will be utilised, with the following variables being utilised:

**(a) Road Category**

Local Distributor Roads (Minor Bus Collector) Class 4 - Category UB
Local Access Roads Class 5a and 5b - Category UC
Local Access loops, etc Class 5c- 5f - Category UD

**(b) Structural Design Period**

Category UB - 15 years - 25 years analysis (probably 20 years)
Category UC - 20 years
Category UD - volumes are too low to apply structural design analysis

**(c) Design Traffic**

The cumulative equivalent traffic loading (E80's per lane) can then be estimated taking into consideration the probable split in mode and vehicle class and by applying the appropriate load equivalency factors. This exercise will have to be done as part of the detail design.

**(d) Availability of Materials**

From inspection it appears that no suitable road building material occurs on site. At this juncture the preliminary cost estimates rely on utilizing road building material from a commercial source.

**(e) Subgrade CBR’s**

The pavement structure will be determined by the CBR of the in-situ subgrade material at the relevant founding level. Depths below finished road level to be used for determining the design CBR of the subgrade are suggested as :

Category UB - 800 mm
Category UC - 600 mm
Category UD - 400 mm

The CBR groups as proposed in the Red Book will be utilised in determining the extent and nature of the selected layerworks, as per the catalogue design method (refer to the table below).

| Design CBR of Subgrade | Upper Selected Layer | Lower Selected Layer | In-situ Subgrade |
|---|---|---|---|
| < 3 | Special treatment required | | |
| 3 - 7 | 150 mm G7 | 150 mm G9 | Rip and compact to 150 mm |
| 7 - 15 | 150 mm G7 | - | Rip and compact to 150 mm |
| 15 - 25 | - | - | Rip and compact to 150 mm |
| > 25 | - | - | Use as subbase or base layer |

**(f) Structural Design**

The catalogue design method as detailed in the Red Book will be utilised for the structural pavement design. For the asphalt surfaced roads, it is anticipated that a granular base (G2) and subbase (G5) will probably be required. Selected layers will probably also be granular and will depend on the design CBR of the in-situ subgrade material as indicated in the above table.

## 5.4 ROAD MARKINGS AND SIGNAGE

All road markings and signage are to be in accordance with the requirements of the South African Development Community Road Traffic Signs Manual (SADCRTSM). Drawing **1692-R-600** included in Volume 2 shows the proposed road markings and signs.

## 5.5 SIDEWALKS

Allowance has been made for 1m wide sidewalks along roads within the proposed development, the positions of which are shown on drawings **1692-R-300** and **1692-R-600**

## 5.6 PUBLIC TRANSPORT EMBAYMENTS

No public embayments are provided within the development.

# *Chapter 6*
# *STORMWATER*

A Stormwater Management Plan, prepared by Nadeson Consulting Engineers (Pty) Ltd and supplied by Bitou Municipality has been used as the basis for the required stormwater infrastructure to accommodate flows within the catchment leading to the proposed development. [referred to as Catchment Area 1 (GV1) in the Stormwater Management Plan]

## 6.1 GENERAL

As stated in the Stormwater Management Plan, “The remaining portion of Catchment Area 1 drains along Rotterdam Street where it crosses Rotterdam Street with a 600mm [450mm Ø] diameter pipe which outlets in to an open field and eventually down to Bosfontein River.

The Green Valley analysis carried out by Nadeson Consulting Engineers (Pty) Ltd has applied a 2-year intensity storm with a total rainfall depth of 88.4mm using the South African 24-hour, Type 2 storm distribution. The analysis for the 50-year intensity storm uses a total rainfall depth of 267.2mm using the South African 24-hour, Type 2 storm distribution.

### 6.1.1 MINOR STORM ASSESSMENT RESULTS

The following was deduced by Nadeson Consulting Engineers (Pty) Ltd:

- The proposed pipe sizes have adequate capacity to accommodate the 2-year storm runoff with over 30% spare capacity (allowed for flash floods).
- Peak flow velocities are above the prescribed minimum (0,5m/s to avoid siltation) are below the absolute maximum for a concrete pipe and channels.
- Surcharges found under the 2-year storm runoff are due to pipes performing under pressure and not gravity.

### 6.1.2 MAJOR STORM ASSESSMENT RESULTS

The following was deduced by Nadeson Consulting Engineers (Pty) Ltd:

- The drainage channels have adequate capacity to accommodate the 50-year storm runoff.
- The road crossings will allow the channel to overtop the road and flow back in to the downstream drainage channel.
- The overland flow paths within the road reserves are able to convey the runoff from the 50-year storm event.

## 6.2 HYDROLOGICAL CHARACTERISTICS WITHIN THE PROPOSED DEVELOPMENT

Preliminary designs based on using the proposed roadway cross sections, indicate that accommodating the internal stormwater will be possible.

For the purposes of this report EAS has used a conservative approach based on black top roads with kerbs. To this end, it is proposed that a dual stormwater management system be used namely:

- Control of both the minor (1:2 year) and major (1:50 year) storms in an overland system which uses the road reserve as the stormwater conduit for the 1:2 year storm and the 1:50 year storm,
- Controlling the excess (1:2 year) water that cannot be dealt with effectively by the overland system in a piped network. Due to the small area of development, the roads have been designed in such a way that stormwater can be controlled and collected at one point, from where it connects to the proposed outfall pipework. The design must make allowance for an overland stormwater escape route from this low point in the roadway.

By combining the above the design will ensure the most cost effective service both in terms of capital expenditure and maintenance cost. The preliminary stormwater layout is indicated on **Dwg 1692-R-400**.

Due to small area encompassing the proposed retirement village (2.1 hectares), there would be no need for on-site attenuation of stormwater. A 600mm diameter, 100D concrete stormwater pipe will convey stormwater generated on the site from the lowest point (near the community centre) to the main pipeline leading to the discharge point. An overland escape route will also be created to allow major storm flow to discharge into the valley below. The outlet will be suitably protected with gabions, reno mattresses or other suitable materials to prevent scour.

### 6.3 DESIGN STANDARDS

A layout plan indicating the stormwater catchment areas is included as **Dwg 1692-R-401.** It is clear from inspection of these plans and the calculations tabulated thereon that the effect of development produces a negligible increase in stormwater run-off which will be attenuated on site.

A floodline analysis was carried out by Fraser Consulting Civil Engineers, which conclude that the 100 year recurrence interval storm its at approximate level 8m MSL, which is at least 1,5m below the existing level of the lowest properties on erf 103 and 4,5m below the proposed levels of the lowest properties (after shaping). Their hydrological calculations show the 100 year RI peak flow rate to be $30m^3/s$.

It is proposed that the internal stormwater design should be based upon the following approach.

#### (a) Run-off Calculation

Designing an effective stormwater control mechanism is essential to ensure minimum maintenance whilst being certain that property and life will be under no threat in heavy rainfall storms. It is important to determine the risks and design the system accordingly.

We are of the opinion that the “Rational Method” will produce results that are accurate and trustworthy for a development of this size.

The following extract from the report by Fraser Consulting Civil Engineers provides key hydrological information:

| Description | Value |
|---|---|
| Applicable Raingauge | Gauge 014633W Plettenberg Bay |
| Mean Annual Precipitation (MAP) | 647mm |
| 100 year RI one day rainfall | 194 mm |
| Catchment Area | 4.2 $km^2$ |
| Time of Concentration | 1.25 hours |
| Rainfall Intensity | 74 mm/h |
| Rational Formula C value | 0.32 |
| 100 year RI peak flow rate | 30 $m^3/s$ |

In most instances it is anticipated that the time of concentration ($t_c$) for the minor catchments will be less than 15 minutes. It is generally accepted that intensities that fall in this category can produce misleading data.

We have thus decided that due to the fact that the average slopes on the sites, combined with the area of the developments and assuming an average water speed, in either the road or a stormwater pipe of approximately 1 m/s would generate a $t_c$ of less than 15 minutes, that a $t_c$ of 15 minutes would be used for the design of the smaller fragmented areas.

To determine the anticipated run-off the following formula will be applied:

$$Q=CIA/3,6$$

**(b) Pipe and Catchpit sizes and positions**

The design philosophy is to determine the amount of stormwater that can be anticipated for a storm of 1:2 years and try and accommodate this water primarily in the road prism using the following criteria as a basis for determining the positions of catchpits or outlets into the veld (see table below).

| ROAD CLASSIFICATION | MAXIMUM ENCROACHMENT |
|---|---|
| Residential and lower-order roads | No kerb overtopping<br>Flow may spread to crown/center of road |
| Residential access collectors | No kerb overtopping<br>Flow spread must leave at least one traffic lane free of water. |

# *Chapter 7*
# *BULK SERVICES*

The developer may be liable for the payment of a Development Contribution (as calculated by Bitou Municipality) for bulk infrastructure upgrades as per Council Policy.

## 7.1 BULK WATER SUPPLY [As per GLS Consulting Report to Bitou Municipality]

The existing water reticulation system has sufficient capacity to accommodate the domestic flow of the proposed development in order to comply with the pressure criteria as set out in the master plan. *Upgrading of the existing reticulation network and bulk supply system is required in order to comply with the fire flow criteria and bulk supply criteria as set out in the master plan*.

The Wittedrift reservoir is supplied with bulk water through a 90 mm diameter dedicated bulk pipeline from an offtake on the bulk pipeline between the Town reservoirs (situated at the Plettenberg Bay Water Treatment Plant) and the Matjiesfontein reservoir. The bulk pipeline to the Matjiesfontein reservoir also supplies bulk water to the Aventura reservoir.

This bulk system to the Wittedrift, Aventura and Matjiesfontein reservoirs does not have sufficient capacity and will have to be reinforced to accommodate the proposed development in the existing water system.

### 7.1.1 Implementation of the Master Plan – Water Network Upgrades

The following network upgrades are proposed in order to comply with the fire flow criteria as set out in the master plan:

<u>Network upgrade</u>
**Item 1** : 430 m x 160 mm Ø replace existing 75 mm Ø pipe = R 617 000 *
**Item 2** : 390 m x 110 mm Ø replace existing 75 mm Ø pipe = <u>R 412 000 *</u>
Total = R 1 029 000 *

(* Including P & G, Contingencies and Fees, but excluding VAT - Year 2019/20 Rand Value. This is a rough estimate, which does not include major unforeseen costs).

The fire flow will improve to 23,5 L/s@ 10 m head with the implementation of master plan item 1, to 14,1 L/s@ 10 m head with the implementation of master plan item 2 and to 38,4 L/s@ 10 m head if both items are implemented.

### 7.1.2 Implementation of the Master Plan – Bulk Water Supply Upgrades

The capacity of the existing 150 mm Ø bulk water supply pipe between the Town reservoirs and the draw off point to the Wittedrift reservoir can be augmented through the implementation of a cross-connection between the 150 mm Ø bulk pipeline and an existing 300 mm Ø network pipe (supplied with water from the Town reservoir A). The proposed cross-connection is shown on Figure 2 attached [**Refer Annexure C - GLS Consulting Report**].

<u>Bulk Water Supply Augmentation</u>

**Item 3**: 25mx160 mm Ø inter-connection between existing 150mm & 300mm Ø pipes = R 80 000 *
(* Including P & G, Contingencies and Fees, but excluding VAT - Year 2019/20 Rand Value. This is a rough estimate, which does not include major unforeseen costs).

Note: It is possible that the proposed connection is already implemented within the existing water system. *This should however be verified by the Bitou Municipality*.

The existing 75mm Ø bulk pipeline between Kammassie Street and Protea Street crosses through the proposed development and will need to be relocated to within the proposed road reserve and form part of the internal reticulation. The cost of isolating valves and re-routing of the pipeline will be to the Developer's account. *GLS Consulting have recommended that a new 75mm Ø pipe be installed in Kammassie Street to improve the existing network west of the proposed development.*

## 7.2 BULK SEWERAGE [As per GLS Consulting Report to Bitou Municipality]

### 7.2.1 Sewer Reticulation

There is sufficient capacity in the existing sewer reticulation system to accommodate the proposed development.

### 7.2.2 Pumping stations & rising mains

Wittedrift PS

The existing Wittedrift PS has a capacity of 8.0 l/s with accompanying 125 mm Ø rising main. This PS and rising main have sufficient capacity to accommodate the proposed development. *The capacity of the Wittedrift PS of 8.0 L/sand the diameter of the accompanying rising main of 125 mm should be verified by the Bitou Municipality*.

Aventura PS

The existing Aventura PS has a capacity of 32 l/s with accompanying 200 mm Ø rising main.
The existing instantaneous peak flow at the Aventura PS is as follows:

| | |
|---|---|
| Flow from upstream Wittedrift PS = | 8,0 l/s |
| Flow from upstream Twin Rivers PS2 = | 5,0 l/s |
| Flow from upstream Aventura resort PS = | 5,0 l/s |
| Flow from upstream Matjiesfontein PS = | 30,0 l/s |
| Total = | 48,0 l/s |

The combined instantaneous peak flow that can arrive at the Aventura PS (when the upstream Wittedrift PS, Twin Rivers PS2, Aventura resort PS and Matjiesfontein PS are pumping simultaneously) is more than the capacity of the Aventura PS (48,0 L/sversus capacity of 32,0 l/s) and overflowing of the Aventura PS will therefore occur if the size of the existing sump of the Aventura PS is insufficient to balance out the peak flows from the respective pumping stations.

GLS Consulting have however received information that the Aventura PS has recently been upgraded to a pumping capacity of 78 L/s, but no upgrading have been performed to the existing Aventura PS rising main.

A pumping capacity of 78 l/s at the Aventura PS will be sufficient to accommodate the proposed development at Wittedrift in the existing sewer system, but the flow velocity through the 200 mm diameter rising main will then be 2,5 m/s. It is not recommended to pump at flow velocities of more than 1,8 m/s over long distances due to high pumping costs.

The pumping capacity of 78 l/s should be verified and it is proposed that flow readings are taken at the Aventura PS to verify the duty point of the PS. The ultimate planned capacity of the Aventura PS is 135 l/s. We therefore recommend that the existing 200 mm Ø Aventura PS rising main is upgraded to a 355 mm Ø rising main (master plan item BPS34.2) to accommodate the ultimate planned Aventura PS capacity.

Note: *The capacity of the Aventura PS of 78 l/s and the diameter of the accompanying rising mains of 200 mm should be verified by the Bitou Municipality.*

### 7.2.3 Implementation of the Master Plan – Sewer Network Upgrades

*The following master plan item will be required to reinforce the existing system in order to accommodate the proposed development.*

Network upgrade (Minimum requirement)

BPS34.2 : 5 400 m x 355 mm Ø Upgrade existing Aventura PS rising main = R 21 270 000 *

(* Including P & G, Contingencies and Fees, but excluding VAT - Year 2019/20 Rand Value. This is a rough estimate, which does not include major unforeseen costs).

### 7.3 Road Access / Traffic Impact Assessment

Entrance to and exit from the retirement village will be onto Protea Street, which carries minimal traffic, as it ends in a cul-de-sac serving 10 erven. This equates currently to 40 trips per day. A retirement village typically generates 3.4 trips per unit during the course of a day. Given 54 units, this relates to 184 vehicle trips split 50% in and 50% out. This additional traffic can easily be accommodated on a road of this nature. During the worst peak hour, the development will generate approximately 16 trips compared to the 10 trips generated by the existing residential units along Protea Street. <u>Bitou Municipality requested that Engineering Advice and Services apply to the District Roads Engineer (Oudtshoorn) for approval of the access onto Main Street, which is a District Road. All efforts to contact the DRE have been without success.</u>

### 7.4 Bulk Stormwater [As per Reports by Fraser Consulting & Nadeson Consulting]

Apart from the floodline study carried out by Fraser Consulting Engineers, an analysis has been carried out of the subcatchments draining (from the south) towards the proposed retirement village. There are two subcatchments which drain towards the area as shown on Dwg 1692-R-401.

The Bitou Municipality provided the Green Valley and Wittedrift Stormwater Master Plan prepared by Nadeson Consulting Services (Pty) Ltd which recommends the following bulk stormwater upgrades in the vicinity of the proposed development.

- The existing 450mm diameter pipe from Hoof Street up to the outfall into the Bosfontein River is to be upgraded to a 900mm diameter pipe along Protea Street [GV 1.1]. The last section, which will accommodate flow from the development will be increased to a 1050mm Ø concrete pipe [GV1.1]. This falls within subcatchment C7 of the Nadeson Consulting Engineers report as depicted on Fig 4.1 included in the Annexures.
- A 2m wide lined channel (1m base width, 300mm depth) is proposed from the existing 450mm Ø outfall on the corner of Hoof Street and Kammassie Street down the length of Kammassie Street <u>to accommodate future stormwater network upgrades</u>. Until such time as these upgrades are implemented, *it is proposed that a grass lined channel be installed along Kammassie Street and flow entering this channel be allowed to discharge to the Bosfontein River beyond Kammassie Street*.

Cost estimate of bulk stormwater upgrades as extracted from Nadeson Consulting Services (Pty) Ltd Report

**Cost Estimate for Stormwater Upgrades**

| | | | Length (m) | Actual Length | Ø / Width | Depth (mm) | Rate/m | Cost as per Masterplan | Cost as per design | Cost incl Contingencies (26.5%) |
|---|---|---|---|---|---|---|---|---|---|---|
| Item | GV1.1 | 100D Concrete Pipe | 400 | 125 | 900 | | R 3 100.00 | R 1 240 000.00 | R 387 500.00 | R 1 568 600.000 |
| | | 100D Concrete Pipe | 60 | 45 | 1050 | | R 3 900.00 | R 234 000.00 | R 175 500.00 | R 296 010.000 |
| | | | | | | | | | | |
| | GV1.2 | Concrete lined channel | 250 | 160 | 1000 | 300 | 800 | R 200 000.00 | R 128 000.00 | R 253 000.000 |
| | | | | | | | | **Cost as per design** | **R 691 000.00** | **R 874 115.000** |

## *Chapter 8*
# *ANCILLARY SERVICES*

### 8.1 REFUSE COLLECTION

It is assumed that refuse will be collected by the municipality from the refuse collection point in Protea Street. The standards used for the design of the road network will not be able to accommodate refuse vehicles within the development.

### 8.2 EMERGENCY VEHICLES

The town planning layout does not make provision for road reserves which are wide enough to accommodate the Bitou Municipality's Emergency Services vehicles.

### 8.3 POST AND TELECOMMUNICATIONS

This service falls outside of the scope of this report. It is however suggested that the matter be looked at to ensure that services do not clash in future. Should telecommunication ducts have to be installed it is also cheaper to undertake this work under the Civils Contract.

### 8.4 STREET LIGHTING AND ELECTRICAL INSTALLATION

This service falls outside of the scope of this report and is being dealt with by Mr John Gant of Nako Triokon, who is dealing with the Electrical Planner, Mr Michael Kwampa in this regard. It is apparent there are issues with existing LV and MV cabling crossing the site. Allowance has been made for a servitude to accommodate relocation, should this be required. Should electricity ducts have to be installed it would be advisable to undertake this work under the Civils Contract.

# Annexure A
# Water Design Information

# Erf Connection Results

| Id | Category | Main dia | Branch dia | Discharge) | Residual head | Pressure |
|---|---|---|---|---|---|---|
| 23-1 | Retirement Unit | 25 | 20 | 0.083 | 61.569 | 6.038 |
| 23-2 | Retirement Unit | 20 | 20 | 0.083 | 61.013 | 5.983 |
| 23-3 | Retirement Unit | 20 | 20 | 0.083 | 56.799 | 5.57 |
| 23-4 | Retirement Unit | 25 | 20 | 0.083 | 56.843 | 5.574 |
| 32-1 | Retirement Unit | 20 | 20 | 0.083 | 50.576 | 4.96 |
| 32-2 | Retirement Unit | 25 | 20 | 0.083 | 50.828 | 4.985 |
| 32-3 | Retirement Unit | 25 | 20 | 0.042 | 50.828 | 4.984 |
| 32-4 | Retirement Unit | 20 | 20 | 0.083 | 50.482 | 4.951 |
| 34-1 | Retirement Unit | 25 | 20 | 0.083 | 56.206 | 5.512 |
| 34-2 | Retirement Unit | 20 | 20 | 0.083 | 50.400 | 4.943 |
| 34-3 | Retirement Unit | 20 | 20 | 0.083 | 50.327 | 4.935 |
| 34-4 | Retirement Unit | 25 | 20 | 0.083 | 63.791 | 6.256 |
| 34-5 | Retirement Unit | 20 | 20 | 0.042 | 63.527 | 6.23 |
| 34-6 | Retirement Unit | 25 | 20 | 0.083 | 63.793 | 6.256 |
| 34-7 | Retirement Unit | 25 | 20 | 0.083 | 50.467 | 4.949 |
| 34-8 | Retirement Unit | 20 | 20 | 0.083 | 63.507 | 6.228 |
| 34-9 | Retirement Unit | 25 | 20 | 0.083 | 63.794 | 6.256 |
| 35-1 | Retirement Unit | 25 | 20 | 0.083 | 62.738 | 6.152 |
| 35-5 | Retirement Unit | 25 | 20 | 0.083 | 59.223 | 5.808 |
| 35-3 | Retirement Unit | 25 | 20 | 0.083 | 58.946 | 5.781 |
| 35-4 | Retirement Unit | 25 | 20 | 0.083 | 57.799 | 5.668 |
| 35-5 | Retirement Unit | 20 | 20 | 0.083 | 57.025 | 5.592 |
| 35-6 | Retirement Unit | 25 | 20 | 0.083 | 55.193 | 5.413 |
| 35-7 | Retirement Unit | 20 | 20 | 0.083 | 54.237 | 5.319 |
| -35-8 | Retirement Unit | 25 | 20 | 0.083 | 52.626 | 5.161 |
| 37-1 | Community Centre | 25 | 20 | 0.208 | 50.961 | 4.998 |
| 37-2 | Retirement Unit | 25 | 20 | 0.042 | 51.132 | 5.014 |
| 37-3 | Retirement Unit | 20 | 20 | 0.083 | 50.766 | 4.978 |
| 37-4 | Retirement Unit | 25 | 20 | 0.083 | 50.949 | 4.996 |
| 39-1 | Assisted Living | 25 | 20 | 0.208 | 51.238 | 5.025 |

# Aquanet Analysis Results

| Instantaneous Peak Node Results - No Fire | | | | | | |
|---|---|---|---|---|---|---|
| Id | Name | Elevation | Head | Residual Head | Pressure | Discharge |
| 26 | FH | 12.69 | 70.24 | 57.555 | 5.644 | 0.112 |
| 18 | Node 1 | 13.04 | 69.71 | 56.672 | 5.558 | 0.599 |
| 2 | Node 2 | 10.05 | 69.71 | 59.66 | 5.851 | 0 |
| 21 | Node 3 | 13.56 | 69.72 | 56.154 | 5.507 | 0.833 |
| 20 | Node 4 | 12.77 | 69.83 | 57.052 | 5.595 | 0.699 |
| 23 | Node 5 | 12.87 | 69.73 | 56.863 | 5.576 | 0.382 |
| 1 | Node 7 | 11.46 | 70.4 | 58.94 | 5.78 | -2.625 |

| Instantaneous Peak Pipe Results - No Fire | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Id | From | To | Length | Diameter | Flow | Velocity | Loss | Gradient |
| 23 | 23 | 21 | 75.92 | 75 | 0.297 | 0.08 | 0.01 | 0.2 |
| 32 | 23 | 18 | 55.45 | 75 | 0.446 | 0.12 | 0.02 | 0.4 |
| 33 | 2 | 18 | 107.865 | 75 | 0 | 0 | 0 | 0 |
| 34 | 21 | 18 | 125.546 | 75 | 0.153 | 0.04 | 0.01 | 0.06 |
| 35 | 20 | 21 | 127.288 | 75 | 0.688 | 0.19 | 0.11 | 0.86 |
| 37 | 20 | 23 | 45.433 | 75 | 1.126 | 0.31 | 0.09 | 2.08 |
| 38 | 1 | 26 | 16.041 | 75 | 2.625 | 0.71 | 0.16 | 9.89 |
| 39 | 26 | 20 | 45.43 | 75 | 2.513 | 0.68 | 0.41 | 9.12 |

| Instantaneous Peak Node Results - Simulated Fire Flow of 15 l/s | | | | | | |
|---|---|---|---|---|---|---|
| Id | Name | Elevation | Head | Residual Head | Pressure | Discharge |
| 26 | FH | 12.69 | 64.23 | 51.545 | 5.055 | 15.112 |
| 18 | Node 1 | 13.04 | 63.7 | 50.661 | 4.968 | 0.599 |
| 2 | Node 2 | 10.05 | 63.7 | 53.65 | 5.261 | 0 |
| 21 | Node 3 | 13.56 | 63.71 | 50.144 | 4.917 | 0.833 |
| 20 | Node 4 | 12.77 | 63.82 | 51.042 | 5.006 | 0.699 |
| 23 | Node 5 | 12.87 | 63.72 | 50.853 | 4.987 | 0.382 |
| 1 | Node 7 | 11.46 | 70.4 | 58.94 | 5.78 | -17.625 |

| Instantaneous Peak Pipe Results - Simulated Fire Flow of 15 l/s | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Id | From | To | Length | Diameter | Flow | Velocity | Loss | Gradient |
| 23 | 23 | 21 | 75.92 | 75 | 0.297 | 0.08 | 0.01 | 0.2 |
| 32 | 23 | 18 | 55.45 | 75 | 0.446 | 0.12 | 0.02 | 0.4 |
| 33 | 2 | 18 | 107.865 | 75 | 0 | 0 | 0 | 0 |
| 34 | 21 | 18 | 125.546 | 75 | 0.153 | 0.04 | 0.01 | 0.06 |
| 35 | 20 | 21 | 127.288 | 75 | 0.688 | 0.19 | 0.11 | 0.86 |
| 37 | 20 | 23 | 45.433 | 75 | 1.126 | 0.31 | 0.09 | 2.08 |
| 38 | 1 | 26 | 16.041 | 75 | 17.625 | 4.8 | 6.17 | 384.58 |
| 39 | 26 | 20 | 45.43 | 75 | 2.513 | 0.68 | 0.41 | 9.12 |

# Annexure B
# Sewerage Flow Calculations

**SEWER FLOWS CALCULATIONS**

1. Middle income (Residential II): (i) 54 units
(ii) 3 people per erf
(iii) 750 l/e/d
(iv) Ave erf area is 300m²

2. Mixed Use (Community Centre): (i) 10 p/unit
(ii) 500 l/p/d

3. Mixed Use (Assisted Living): (i) 10 p/unit
(ii) 500 l/p/d

4. Design procedures is in accordance to the RED BOOK and HARMON
5. Infiltration = 15% for RED BOOK and 100% for HARMON

**RED BOOK**

Population = (54 X 3) + (10 + 10) = 182

Peak factor = 2.5

AADD $= \frac{(54 \times 750) + (1 \times 5000) + (1 \times 5000)}{24 \times 60 \times 60}$

= 0.584 l/s

PDWF = 2.5 X 0.584

= 1.461 l/s

PWWF = 1.461 X 1.15

= 1.680 l/s

**HARMON**

Peak factor $= 1 + \frac{(\quad 14 \quad)}{4 + p \wedge 0.5}$

= 4.163

AADD $= \frac{(54 \times 750) + (1 \times 5000) + (1 \times 5000)}{24 \times 60 \times 60}$

= 0.584 l/s

PDWF = 4.163 X 0.642

= 2.431 l/s

PWWF = 2.431 X 2

= 4.862 l/s

# Annexure C
# Flood Study Report

# Annexure D
# Geotechnical Results

# Annexure E
# Preliminary Project Costing