CIVIL ENGINEERING REPORT

Proposed Development on
Portion 91 of Farm 304 Matjes Fontein
NOVEMBER 2024

PREPARED FOR:

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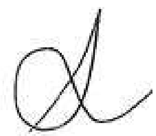
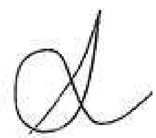
ISSUE & REVISION RECORD

QUALITY APPROVAL

	Capacity	Name	Signature	Date
By Author	Project Manager	Hugo Ras		2024-11-18
Approved by Design Centre Leader	Project Director	Hugo Ras		2024-11-22

REVISION RECORD

Revision Number	Objective	Change	Date
0	Issue to Client	None	2024-11-22

1. INTRODUCTION

ZS2 Consult was appointed by *Dr Nicky Frootko* to comment on the civil engineering aspects of the proposed development on **Portion 91 of Farm 304 Matjes Fontein**, Keurbooms Strand in the Western Cape. At the time when this report was compiled we did have any detailed civil engineering drawings or design specifications in our possession to review. We therefore based our report on our own high-level calculations and rational civil engineering assumptions and interpretations.

2. LOCATION

The property is located at Portion 91 of Farm 304 Matjes Fontein, Keurbooms Strand, at the following coordinates:

Latitude : 34° 0'21.77"S

Longitude : 23°26'12.52"E

Figure 2A: LOCATION OF PROPERTY

3. BULK WATER SUPPLY

3.1 CURRENT STATUS QUO

It is a known fact that the current Goose Valley / Matjiesfontein / Wittedrift bulk potable water supply system of the Bitou Municipality, that must provide potable water to the proposed development, is currently over its maximum capacity. This system can therefore not provide any potable water to the proposed development. The system must be augmented in the future but due to budget constraints this upgrade is most likely some time away.

3.2 POTABLE WATER DEMAND

A high-level estimate without detailed information or drawings of the proposed units are as follows:

- 60 UNITS of 1 or 2 Bedroom Units with a potable water demand of 500 litres / day as per municipal guidelines. That equates to 250 litres / day / person.

Table 1A: Potable water Demand Flows					
Description	**Nr Off Unit**	**Person per Unit**	**Persons**	**DEMAND per PERSON**	**DEMAND**
1/ 2 BEDROOM UNIT	60 units	2 pers	120 per	250 l/day	30000 l/day 30.0 kl/day

Average Water Demand	30000 l/day	1.1	**33000 l/day**	**0.3819 l/s**
Maximum Water Demand	33000 l/day	1.4	**46200 l/day**	**0.5347 l/s**
Peak Water Demand				**0.878 l/s**

Figure 1A: POTABLE WATER DEMAND: TABLE 1A

The proposed development requires the following potable water supply:

Average Water Demand: **33000 litres per day**
Maximum Water Demand: **46200 litres per day**
Peak Water Demand: **0.878 litres per second**

3.3 CURRENT POTABLE WATER DEMAND ON BULK LINE

The effect of the additional water demand of the proposed development is calculated as follows:

We counted the existing units in the Keurbooms Stand area, and we roughly estimate that there are currently approximately 450 units that are fed by the Keurbooms bulk water supply line.

Table 1B: Potable water Demand Flows											
Description	**Nr Off Unit**		**Person per Unit**		**Persons**		**DEMAND per PERSON**		**DEMAND**			
KEURBOOMS	430	units	2	pers	860	per	250	l/day	215000	l/day	215.0	kl/day

Average Water Demand	215000 l/day	1.1	**236500 l/day**	**2.7373**	**l/s**
Maximum Water Demand	236500 l/day	1.4	**331100 l/day**	**3.8322**	**l/s**
Peak Water Demand				**6.296**	**l/s**

Figure 1B: POTABLE WATER DEMAND: TABLE 1B

The current demand on the existing Keurbooms bulk water supply line is as follows (based on a high level rough estimate without detailed information):

Average Water Demand: **215000 litres per day**
Maximum Water Demand: **236500 litres per day**
Peak Water Demand: **6.296 litres per second**

NEW WATER DEMAND ON BULK KEURBOOMS BULK LINE				
Average Water Demand	0 l/day	1.1	**269500 l/day**	**3.1192 l/s**
Maximum Water Demand	269500 l/day	1.4	**377300 l/day**	**4.3669 l/s**
Peak Water Demand				**7.174 l/s**

The nett effect of the **additional demand** by the proposed development will be as follows:

269500 litres per day / 236500 litres per day = 14% INCREASE
269.5 Kilolitres per day / 236.5 Kilolitres per day = 14% INCREASE

The existing Keurbooms bulk water supply line is currently at full capacity and it is therefore clear that the bulk line will not be able to supply the proposed development with potable water.

3.4 RAINWATER HARVESTING

The developer states that rainwater harvesting on site will be utilised to accommodate the potable water demand of the proposed development. We calculated the possible amount of water that could be generated by rainwater harvesting. Our calculations are based on theoretical assumptions that only exists in a perfect scenario with no prolonged dry spells and with adequate storage space on site. We assume that no rainwater is wasted during a heavy rain down pour (high rainfall intensity) and that no water is wasted with storage tanks overflowing. Our calculations are as follows:

RAINWATER HARVESTING			
Annual Rainfall	**Period**	**Roof Area per Unit**	**Total Water Generated per Period**
710 mm 0.71 m	365 days	150 m2	106.5 m3 106500 litres 292 litres per day per unit

RAINWATER HARVESTING GENERATED versus PORTABLE WATER DEMAND				
60 units	292	litres per day per unit generated by rainwater harvesting	17507	litres per day total
Average Water Demand		**53%**	33000	litres per day total
Maximum Water Demand		**38%**	46200	litres per day total

SHORT FALL OF RAINWATER HARVESTING	
Average Water Demand	**47% Short fall**
Maximum Water Demand	**62% Short fall**

From the above calculations the following:

Water generated by harvesting: **17507 litres per day**

Potable water required: **33000 litres per day (Average demand)**

Potable water required: **46200 litres per day (Maximum demand)**

Our calculations above indicates that rainwater harvesting on site will be insufficient to accommodate the potable water demand of the proposed development.

3.5 BULK POTABLE WATER CONCLUSION

Firstly, the existing Keurbooms bulk water line do not have capacity to provide potable water to the proposed development.

Secondly, we are concern that the volume of possible generated rainwater harvested water and stored on site will not be adequate to provide the proposed development with sufficient potable water.

Overall, based on the options we are aware of, we are not convinced that there will be sufficient potable water supply to meet the demand required by the proposed development.

4. SEWAGE

4.1 CURRENT STATUS QUO

The area where the site of the proposed development is located has currently no formal municipal waterborne sewer reticulation system.

4.2 SEWAGE DEMAND

We do not have any detailed drawings of the units of the proposed development. We therefore assumed the following parameters for the calculation of the expected sewer load produced by the proposed development:

- 30 of 1 or 2 Bedroom Units with a sewer flow of 500 litres / day as per municipal guidelines
- 30 of 3 Bedroom Units with a sewer flow of 700 litres / day as per municipal guidelines

Refer to Table 2 below for estimates of sewage flows.

5.2.3 Where the proposed development has no specific development category description as listed in Table 2, estimation of sewage flows may be arrived at through interpolation between development categories.

Description	Sewage Flow
Residential Units	500 l/d for 1 or 2 bed unit with 200 l/d increments for each additional bedroom.
Offices	300 l/100 m^2 of gross floor area
Guest Houses	200 l/d per bed
Day Schools	40 l/d per pupil
Boarding Schools	120 l/d per pupil
Day Clinics and Police Stations	400 l/100 m^2 of gross floor area
Holiday Resorts	400 l/d per bed
Conference Centres with no beds	400 l/100 m^2 of gross floor area
Conference Centres with beds	400 l/d per bed
Restaurants	50 l/seat per day
Shopping Centres	300 l/100 m^2 of gross floor area
Communal Halls and Churches	50 l/seat per day
Service Stations	400 l/100 m^2 of gross floor area

Table 2: Guide for Estimation of Sewage Flows for Conservancy Tanks System

Figure 2A: SEWAGE DEMAND: TABLE 2

The Average Dry Weather Flow (ADWF) presented in Table 2A below was calculated using the Guidelines of Table 2 above. As per the Municipality Guidelines an allowance of at least 15% stormwater infiltration into the reticulation network was made over and above the estimated sewage flows based on the Municipality Guidelines.

Table 2A: Sewage Flows						
Description	**Nr Off UNITS**	**Demand (ADWF)**	**ADWF per UNIT**	**ADWF**	**Harmon Peak Factor**	**PWWF**
1/ 2 BEDROOM UNIT	30 units	3 l/m2	500 l/day	15000 l/day	3.8	57.0 kl/day
3 BEDROOM UNIT	30 units	3 l/m2	700 l/day	21000 l/day	3.8	79.8 kl/day
Total Demand				**36000 l/day**		**136.8 kl/day**
						1.58 l/s
Stormwater	15% of	36000		5400 l/day		
Total Demand				**41400 l/day**		

Figure 2B: SEWAGE DEMAND: CALCULATIONS

4.3 CONSERVANCY TANK OPTION

In the event that a conservancy tank option was to be considered for the proposed development, the size was calculated as follows:

The size of such a proposed conservancy tank to be regularly emptied as recommended is determined as per the municipal guidelines with reference to Table 3.

5.3.1 As a minimum requirement, conservancy tanks shall be sized to be regularly emptied as recommended in Table 3. Where the proposed emptying frequency is less than the recommended retention period in Table 3, the owner of the premises shall comply with the provisions of Clauses 6.6.5 – 6.6.7 of these guidelines.

Description	Recommended Minimum Retention Period
Single Residential Units	14 days
Multiple Residential Units with less than 10 Units	14 days
Multiple Residential Units with more than 10 units	7 days
Offices	14 days
Guest Houses	14 days
Day Schools	14 days
Boarding Schools	14 days
Day Clinics and Police Stations	14 days
Holiday Resorts	7 days
Conference Centres with no beds	14 days
Conference Centres with beds	7 days
Restaurants	14 days
Shopping Centres	7 days
Communal Halls and Churches	14 days
Service Stations	7 days

Table 3: Minimum Design Sewage Retention Periods for Conservancy Tanks System

Figure 2C: SEWAGE DEMAND: RETENTION PERIODS

The recommended emptying frequency of the conservancy for a multiple residential unit development with more than 10 units is 7 days.

Conservancy Tank Size	**Sewage Load**	**Interval Requirement**	**Tank Size Requirement**
	41400 l/day	**7 days**	**289800 L**

However, the municipal guidelines specifies that an additional capacity of 72 hours (3 days) must be allowed for in the event of unforeseen events.

Conservancy Tank Size	**Sewage Load**	**Interval Requirement**	**Tank Size Requirement**
Size Required as per Table 3	**41400 l/day**	**7 days**	**289800 L**
72 hrs Emergency Storage	**41400 l/day**	**3 days**	**124200 L**
	Total	**10 days**	**414000 L**

The size of a conservancy tank required for the proposed development is thus 414 Kilolitres. This is an enormous amount of raw effluent to be emptied and cart away with trucks every 7 days. Even if we work on a minimum volume of effluent of 36000 l/day x 7 days = 252000 litres, it still appears to be impossible for municipal trucks or the trucks of a private service provider to cart away this large volume of effluent every 7 days.

We are thus of the view that a conservancy tank solution is not an option as a solution to the disposal of the generated sewage loads of the proposed development.

4.4 WASTEWATER TREATMENT FACILITY OPTION

We understand that the developer is proposing a wastewater treatment facility (package plants) that will be located on the site to treat the generated sewage flow. The treated “clean” water will then be utilized and disposed on the site by means of irrigation and other.

We do not have any drawings and design specifications of the proposed treatment plant and can therefore not comment. It is important to note that various required specifications must be adhered to by such a wastewater treatment facility (package plants) before it will be approved by the local municipality and other environmental entities.

These requirements for example include the following:

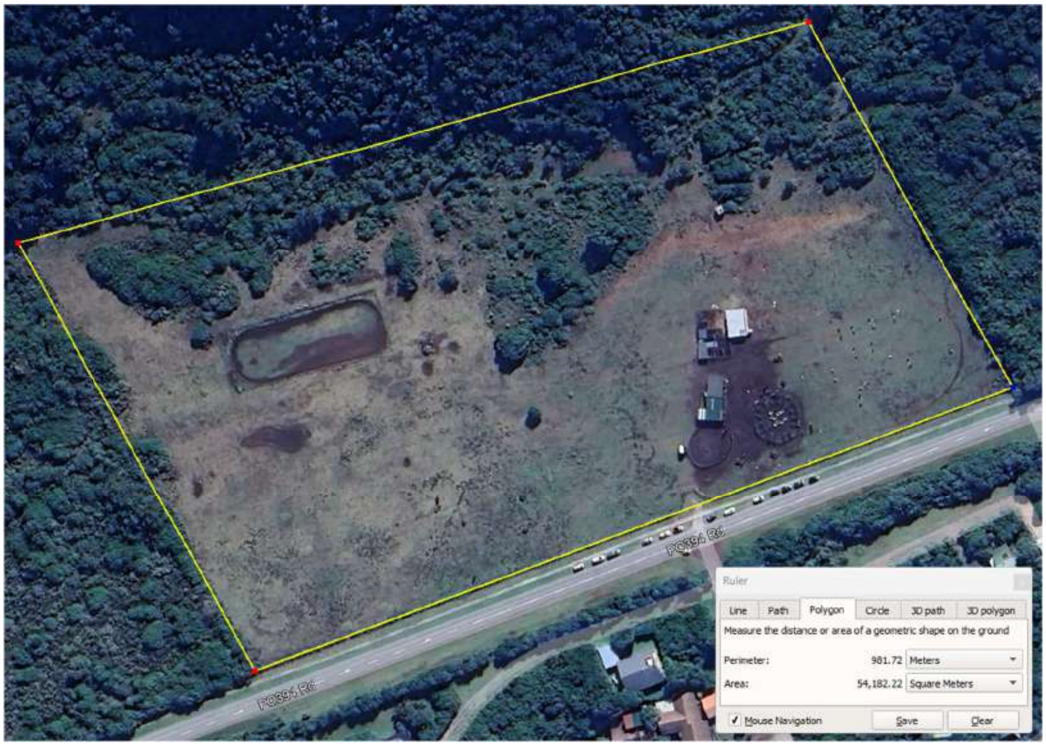
Process Design

- The Basis for Selection of a Design Flow Capacity (kl/d)
- Volumes of different Phases - anaerobic phase, biological reactor, clarifier,
- Process Configuration Drawings - anaerobic tanks, aerobic tank, clarifier tank, and a disinfection tank.
- Design Information of Reactor volumes design COD of maximum mg/l
- Disinfection Circulation (LPM), Buffer Feed Pump (LPM), and Discharge Pump flow rate (LPM).
- Phosphates Concentration in Feed Average Characteristics May estimate
- For normal municipal wastewater Total Phosphates are usually in the order of approximately 3% of COD,
- Buffer Tank: A buffer tank or septic tank is critical for the aboveground installation as
- Main Objectives of Aerobic Tank to Reduce Ammonia: the main two objectives for provision of aerobic zones in anaerobic/anoxic/aerobic activated sludge reactor systems in both for conversion of Ammonia into nitrates and conversion of carbonaceous matter (COD) into sludge mass. So aerobic zone is for both Ammonia and COD reduction; it is the same reason that the unaerated sludge mass fraction in these types of biological reactors is never allowed to be more than 60% of the total reactor sludge mass in the reactor.
- Clarification Tank Assists with Denitrification: The main purpose of the clarifier tank is to clarify, i.e., settle solids from the mixed liquor. Denitrification is achieved through recycling of a nitrate-rich mixed liquor from the aerobic zone. Recycling from the clarifier is mainly for recycling of sludge back to the beginning of the reactor, for an MLE Process.
- Mixed Liquor Recirculation for Denitrification
- Clarification Reduces Sludge Quantity from the System
- Disinfection Chlorine Contact Tank: Chlorine disinfection requires contact time to allow for killing of pathogens. Literature recommends that at least 30 minutes of contact time after chlorination should be allowed for effective disinfection. Ideally, chlorination should occur as the effluent enters the disinfection tank, not as it leaves the tank.
- Removal of Screenings and Sludge Dewatering: Removal of screenings and periodic removal of waste sludge are important elements of operation of a wastewater treatment system.

- Residual Chlorine (mg/l): The General Standard requires residual chlorine of 0.25 mg/l in final effluent.
- Effluent Discharge to Environment: Consideration of a Package Plant would be motivated within the municipality on the basis that effluent will be reused within the development. Thus, there should be no discharge to the environment. This is a fundamental requirement; otherwise, the current municipal Water Services Bylaw prohibits the department from approving package plants within a reticulated area.

Other Operational Related Comments

- Emergency Allowance for No Power Conditions: The design should allow for emergency conditions when there is no power supply. For the aboveground installation it would be ideal to include allowance of emergency storage in the Septic Tank or Buffer Tank. Alternatively, a standby generator should be included. The ideal situation would be to include both, as standby generators fail when they are not maintained properly.
- Bypass Piping and Valve System for Isolation of In-line Screen and Tanks: It would be ideal that bypass piping and valve system is allowed for isolation of in-line screen for maintenance purposes, or the need for isolation or removal of one of the tanks while keeping other tanks on duty.
- Detailed Operation and Maintenance Manual: A typical package plant should be delivered with a detailed Operation and Maintenance Manual that will include the process description, operational parameters (design sludge age, screenings removal, sludge removal and drying, disinfection, effluent re-use, sampling, testing requirements, etc.) as well as mechanical and electrical maintenance requirements. The manual should be sufficiently detailed to be handed over from one process controller to the next without the immediate need for supplier consultation as operational staff is changed.



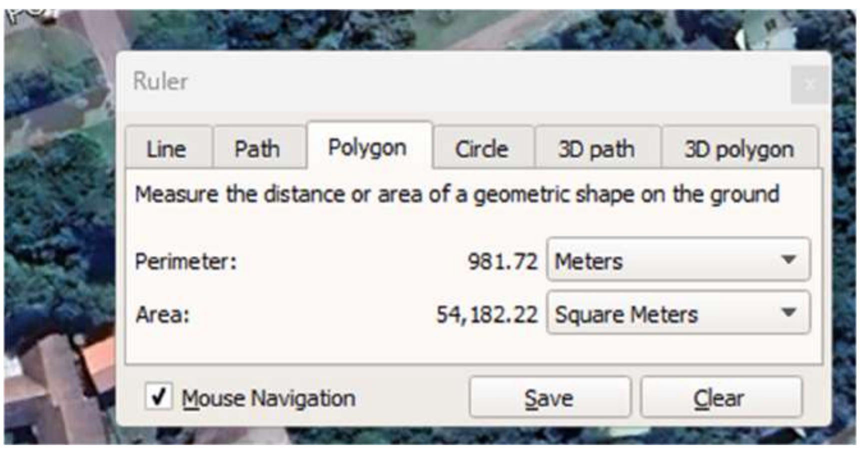


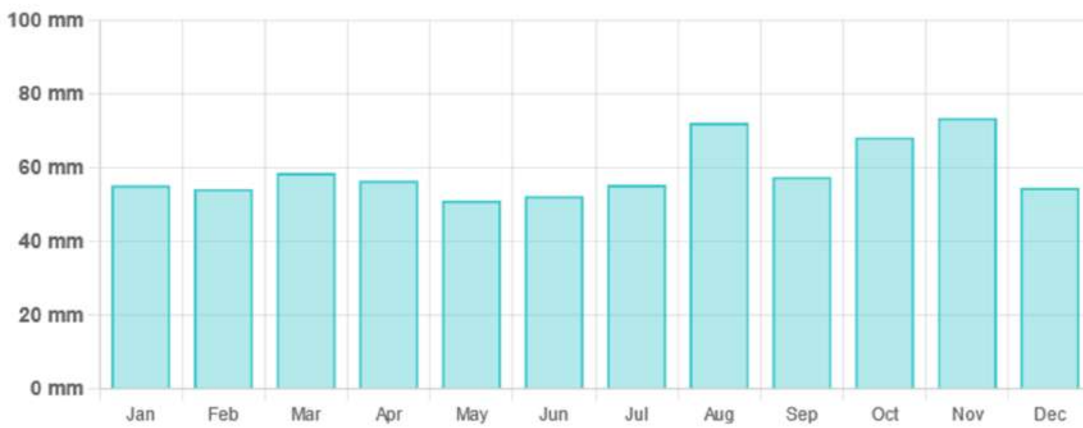
Figure 2D: SEWAGE DEMAND: LEVEL CLEARED AREA OF PROPSED DEVELOPMENT

The developer proposes to dispose of the treated wastewater on site be means of irrigation and other. The volume of treated wastewater from rough estimated calculations will be in the order of 36000 litres per day. Now to put this volume of water in perspective the following: The area of development is approximately 54182 m2 (level cleared area of site as per Figure 2D). Now the irrigation area is assumed to be 30% x 54182 m2 (70% are buildings and roads and retention ponds and other) equals to 16255 m2 (irrigation area).

Now 36000 litres per day divided per 16255 m2 equates to 2.215 litres per m2 or 0.002215 m3 per m2. This the equivalent of 2.215 mm of rain per day or 808 mm per year.

Keurboomstrand Precipitation: Average Monthly Rainfall and Snowfall

This graph shows the average amount of rainfall per month in Keurboomstrand (Western Cape). The numbers are calculated over a 30-year period to provide a reliable average.



- On average, November is the wettest month with 74 mm of precipitation.
- On average, May is the driest month with 51 mm of precipitation.
- The average amount of annual precipitation is 710 mm.

The treated water generated by the sewer treatment plant is more than double the average 710 mm rainfall for the Keurbooms Strand area, if it is compared to the estimated available irrigation area on the development area.

4.5 SEWAGE CONCLUSION

We are therefore concern that the volume of generated treated water is too excessive to be utilised on the site as per the intention of the developer.

5. STORMWATER

5.1 SITE TOPOGRAPHY

The stormwater management is problematic on the proposed site. The site has two high points, one very high on the northern boundary (labelled as "HP2") and another low high point at the southern boundary against the Keurbooms road (labelled as "HP1"). This means that storm water that is generated on the site and on the northern adjacent high lying area is land locked on the site with no natural drainage of the site possible.

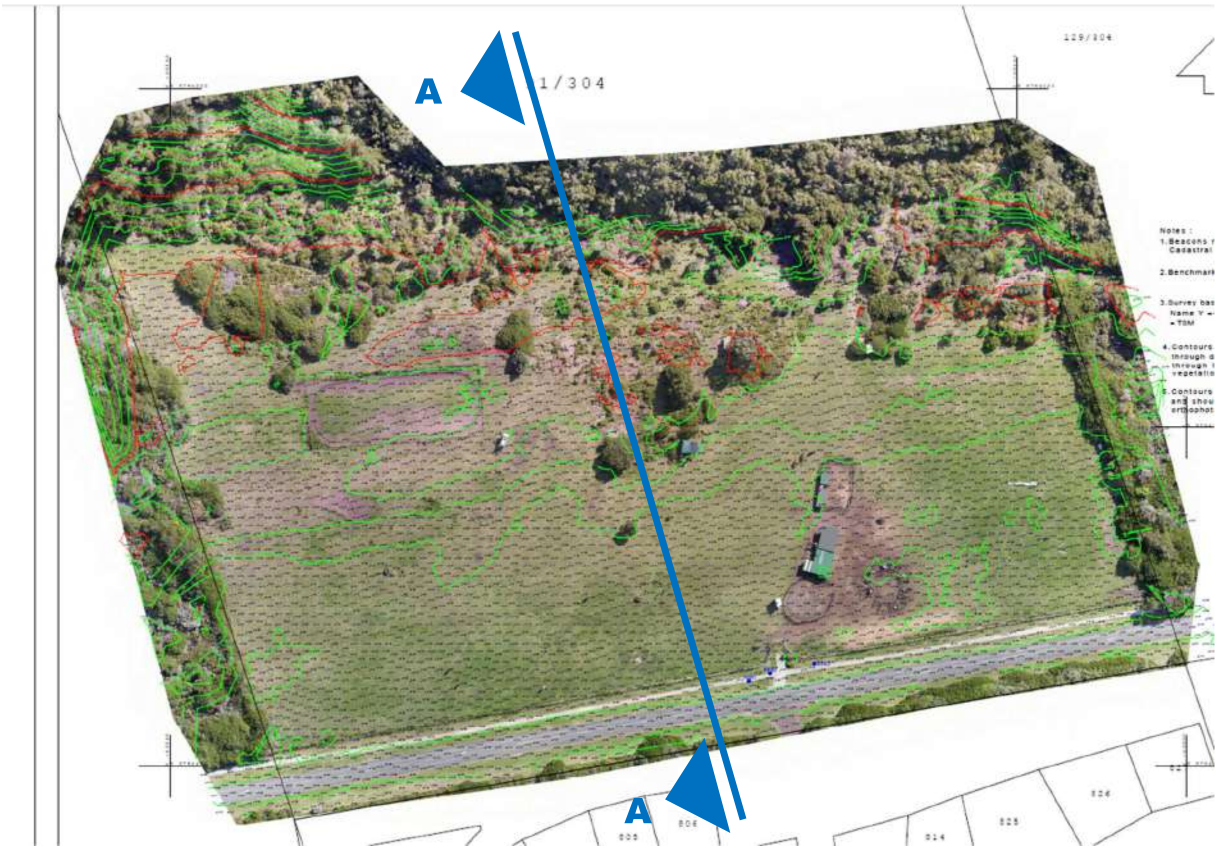


Figure 3A: STORMWATER: ARIAL VIEW OF SITE AND SECTION A-A

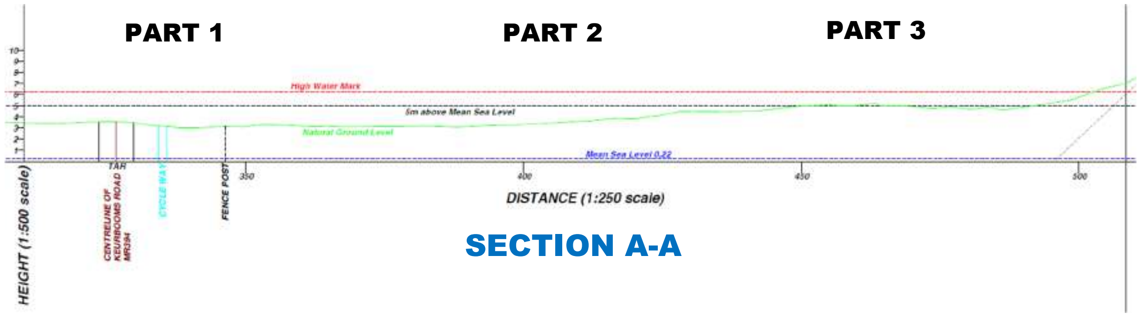


Figure 3B: STORMWATER: SECTION A-A

The land locked site with the trapped stormwater between the high points is illustrated below with the enlarged Section A-A part 1, part 2 and part 3.

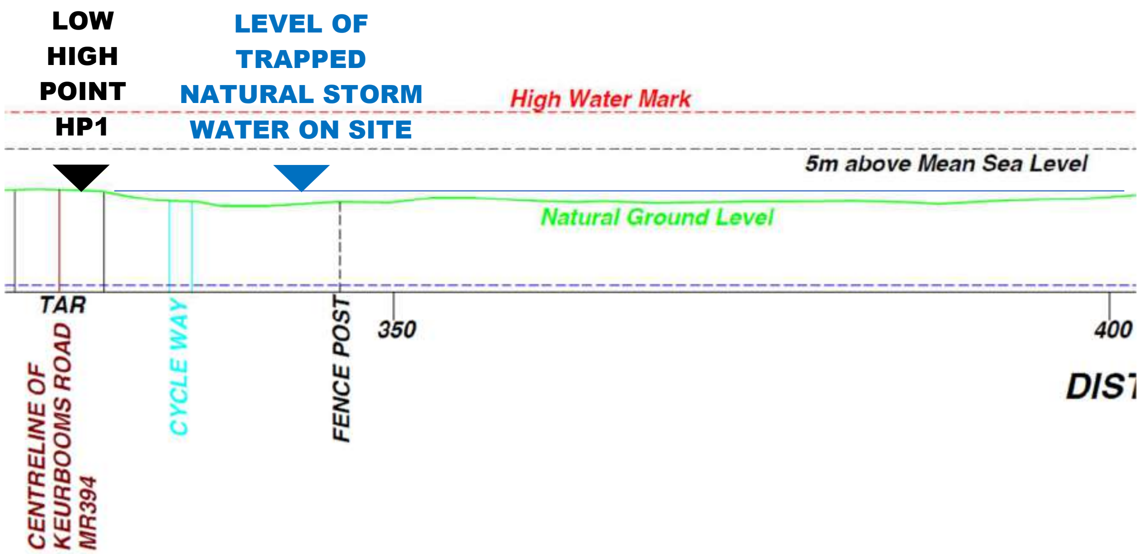


Figure 3C: STORMWATER: PART 1 OF SECTION A-A

Figure 3D: STORMWATER: PART 2 OF SECTION A-A

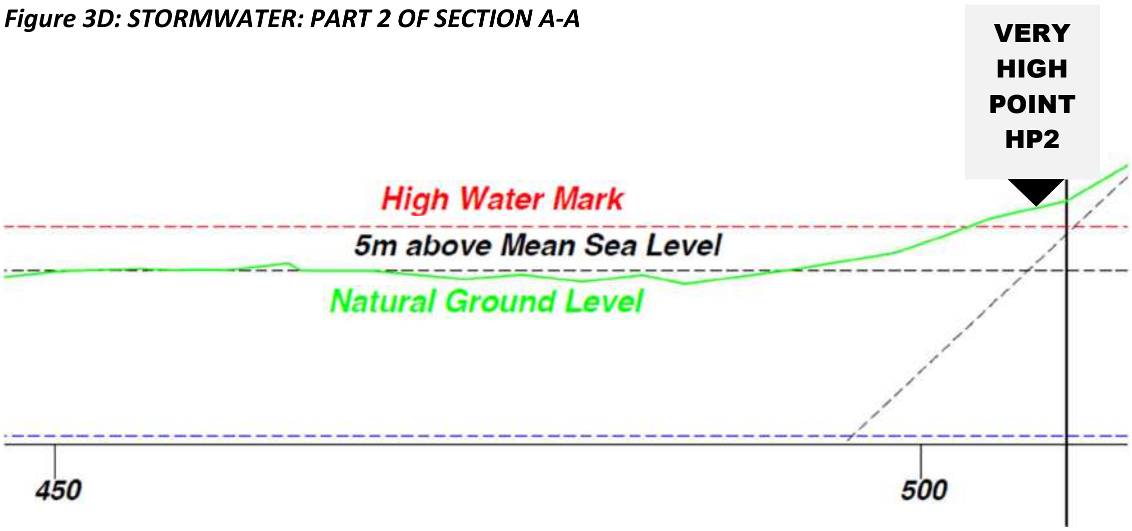


Figure 3E: STORMWATER: PART 3 OF SECTION A-A

The land locked site with the trapped stormwater between the high points is illustrated below with the enlarged Section A-A part 1, part 2 and part 3.

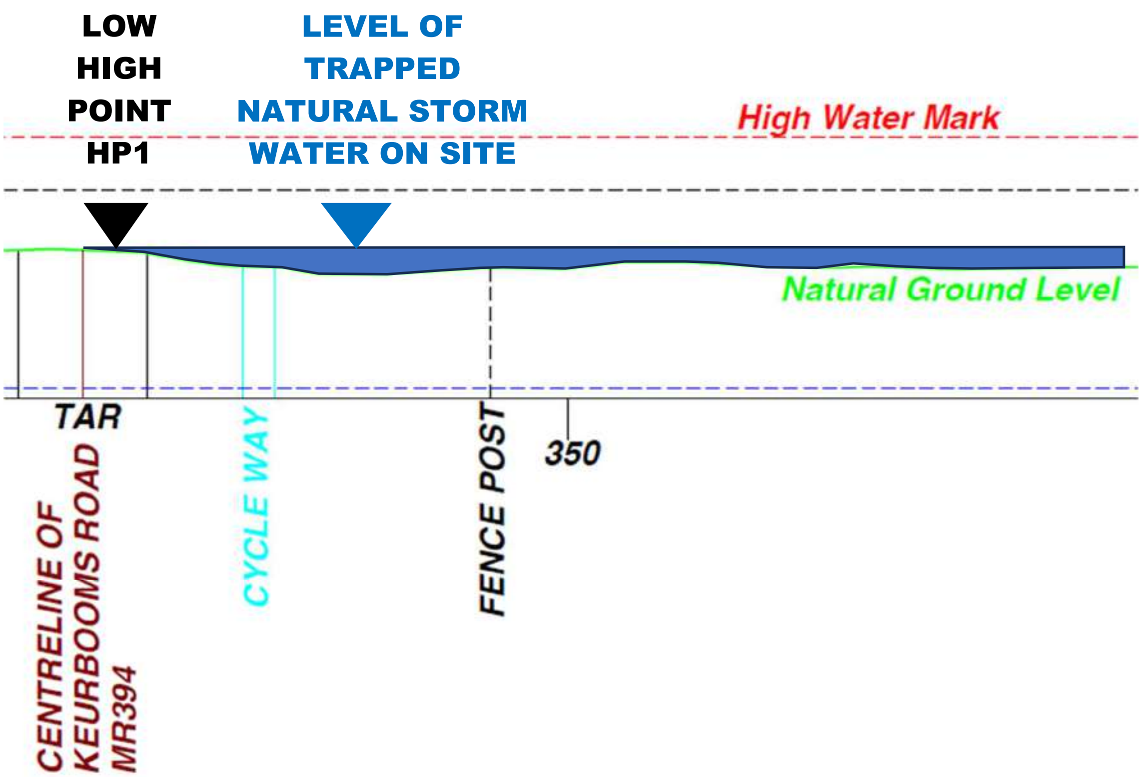


Figure 3F: STORMWATER: PART 1 OF SECTION A-A ENLARGED

5.2 FORMAL STORMWATER INFRASTRUCTURE

There is currently no existing municipal formal infrastructure around the site. Formal stormwater infrastructure would include a pipe reticulation system and channels. Therefore, it is currently not possible to discharge the stormwater that will be generate on the site by the proposed development into a nearby stormwater system. It is understood that the developer proposes that three retention ponds will be constructed on the site to accommodate all the site generated stormwater.

A retention pond is based on the principal that large volumes of stormwater, generated by a rain event, is collected and stored at the time of a rain event in these proposed retention ponds. The temporary stored water in these retention ponds is then slowly released (soak away) into the underlying soil over time. This is a common practice that is utilised these days by property owners to retain and accommodate their generated stormwater on their properties in the case where informal stormwater is not present or where the existing formal infrastructure capacity is not adequate to accommodate the additional flow from a new proposed development.

However, in this case the existing water table is very high due to the low ground levels and nearby estuary. Refer to next paragraph 5.3. We have not seen to date any drawings indicating the proposed location of these ponds on the site, but these ponds will obviously be located at the low points on the site so that stormwater will gravity feed to these ponds.

Unfortunately, the lower the invert level of the ponds, the closer the bottom of the pond will be to the existing high water table level, and it might even be below the existing water table level. This high water table is very problematic for the draining process of the proposed retention pond as the high water table will prevent these ponds from draining and thus defy the objective of the design principal of these ponds. These retention ponds will thus be ineffective.

In our view, because of the reason provided above, the proposed stormwater design of the proposed development is flawed.

5.3 HIGH WATER TABLE

The images below show the current existing level of the water table in the area. These measurements indicate that the top of the water level of the existing water table is between 1.5m and 1.8m below natural ground level. This water level could also be expected on the site of the proposed development (close proximity to site). This could even be closer to the natural ground level at the low points on the site of the proposed development.

The high water table on the site of the proposed development will have an impact on the following:

- Effectiveness of the proposed retention ponds
- Design of the foundation system of the top structures (residential units) on the site
- Design of possible swimming pools at the residential units

Ground-water measurements on Portion 14/91 directly opposite the proposed development site, were taken at low-tide during a dry rainfall period and measured between 1.5m and 1.8m below ground level.

Ideally ground-water levels should be measured over a period of a year, under all weather and tide conditions, because the ground-water and the sea are connected at sites such as these in the Coastal Zone, causing levels fluctuate significantly .

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Figure 3G: STORMWATER: HIGH WATER TABLE

5.4 STORMWATER CONCLUSION

In our view, because of the reasons provided above, the proposed stormwater design of the proposed development is flawed.

6. EXECUTIVE SUMMARY

6.1 BULK POTABLE WATER SUPPLY

We are not convinced that the potable water supply to the proposed development is adequately addressed.

6.2 SEWAGE

We are not convinced that the disposal of the anticipated sewage generated by the proposed development is adequately addressed.

6.3 STORMWATER

We are not convinced that the disposal of the anticipated stormwater generated by the proposed development is adequately addressed.

6.4 FLOODING

The possible flooding of the low-lying site is a major concern. It must be understood that that the homeowners will have a problem with homeowner insurance as insurance companies will identify the site as a high risk prone to flooding and could most likely declare the top structures (residential units) on the site as uninsurable.