PROPOSED GREENGATE EXT 98

STORMWATER MANAGEMENT PLAN FOR THE PROPOSED TOWNSHIP OF GREENGATE EXT 98 SITUATED ON PORTION 260 (A PORTION OF PORTION 114) OF THE FARM RIETFONTEIN 189 IQ

30 July 2020

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TITLE : OUTLINE SCHEME REPORT	STORMWATER MANAGEMENT REPORT FOR THE PROPOSED TOWNSHIP OF GREENGATE EXT 98 SITUATED ON PORTION 260 (A PORTION OF PORTION 114) OF THE FARM RIETFONTEIN 189 IQ

ILIFA PROJECT NO :	DATE :	REPORT STATUS :
K19-040-02	29 July 2020	Revision 0

CARRIED OUT BY :	COMMISSIONED BY :
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SYNOPSIS :

STORMWATER MANAGEMENT PLAN FOR THE PROPOSED TOWNSHIP OF GREENGATE EXT 98 SITUATED ON PORTION 260 (A PORTION OF PORTION 114) OF THE FARM RIETFONTEIN 189 IQ

KEY WORDS :

Stormwater Management

QUALITY VERIFICATION

SACAS
ISO 9001:2015

This report has been prepared under the controls established by a certified ISO 9001:2008 quality management system developed by Ecosphere (Pty) Ltd.

Verification	Capacity	Name	Signature	Date
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No. of Copies Made:	Print Date:	No of Reprints:	Date:
1	29/07/2020	Nill	

Copies sent to:	Deliver Method:
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Robert Victor	Transmitted

CONTENTS

FIGURES

TABLES

ANNEXURES

1. INTRODUCTION

This stormwater management report addresses the accommodation and management of stormwater runoff for the proposed township of Greengate Ext 98 situated on Portion 260 (a portion of Portion 114) of ohe Farm Rietfontein 189 IQ. Please refer to the proposed township layout attached as Annexure A.

This report is submitted to the Gauteng Department of Agriculture and Rural Development (GDARD) as part of the requirements of issuing the record of decision (ROD) and the water use license (WUL).

2. BACKGROUND

2.1 PROPERTY DESCRIPTION

The proposed township is situated in the jurisdiction area of the MCLM. The site is situated adjacent to Beyers Naude Drive (Road K31) between Tuohyvale and Valley Road. The total area of the township is 8.8893 ha.

2.2 CURRENT LAND USE

The property is currently zoned “Agricultural”.

2.3 PROPOSED LAND USE

The anticipated land use is summarized in Table 1. The particulars of the proposed township are as follows:

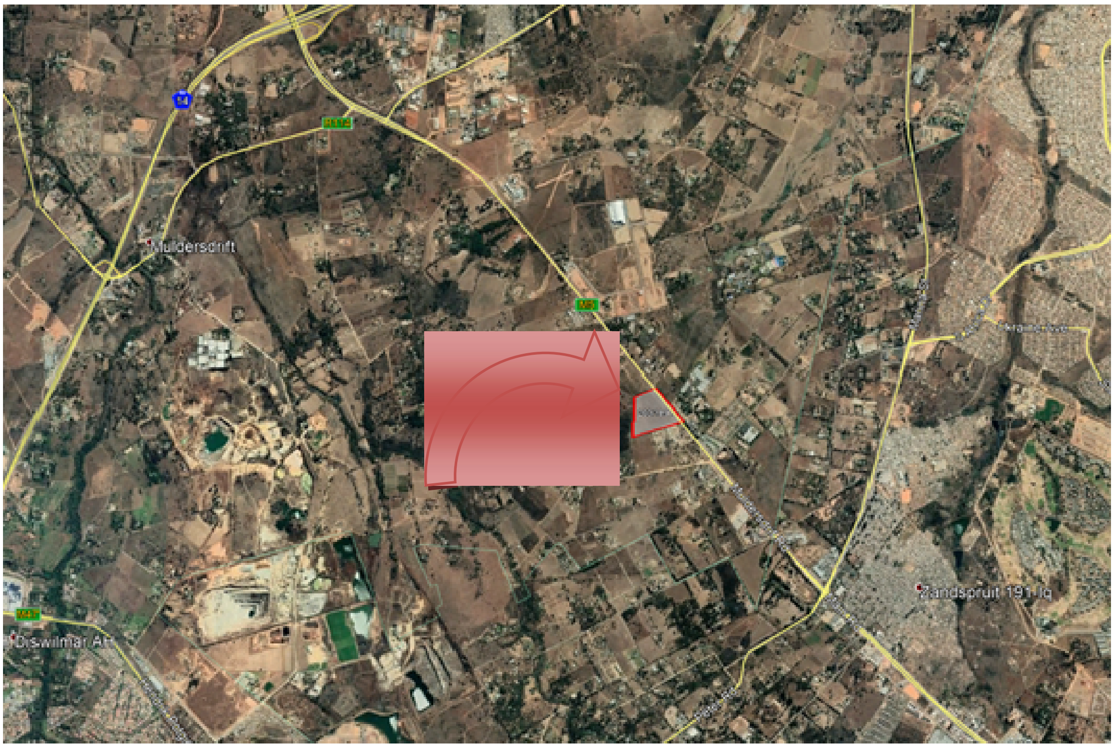
a) Erf 1 to 4 and 6 to 7 “Business 1”
   - Coverage 70%
   - FAR 0.8
   - Height 4 storeys

b) Erf 5 “Commercial”
   - Coverage 70%
   - FAR 0.8
   - Height 4 storeys

Table 1 : Proposed land use

DESCRIPTION	***ZONING***	***AREA (ha)***	***FAR***	***FLOOR AREA (m²)***
Erf 1 to 4 and 6 to 7	Business 1	5.1428	0.8	41,142.40
Erf 5	Commercial	1.1408	0.8	9,126.40
Roads		2.6057		
TOTAL		**8.8893**		**50,268.80**

Figure 1 : Locality Plan



3. EXISTING MUNICIPAL STORMWATER INFRASTRUCTURE

No existing municipal stormwater infrastructure is located in the vicinity of the proposed development.

4. NATURAL TOPOGRAPHY

The Wilgespruit river is located to the West of the proposed development in accordance with the 1:50 000 Topographical map (2627BB) as depicted on Figure 2.

The site is located fairly high upstream within the A21C quaternary catchment. According to the Water Resources of South Africa, 2012 study (WR2012), the Mean Annual Precipitation is 707 mm with a Mean Annual Runoff of 14.05 mm for the A21E catchment. The average volume runoff coefficient is therefore calculated at 0.020.

The topography of the area forms part of a rural area with a relatively steep slope downwards towards the Western boundary of the proposed development of approximately 7.0 %.

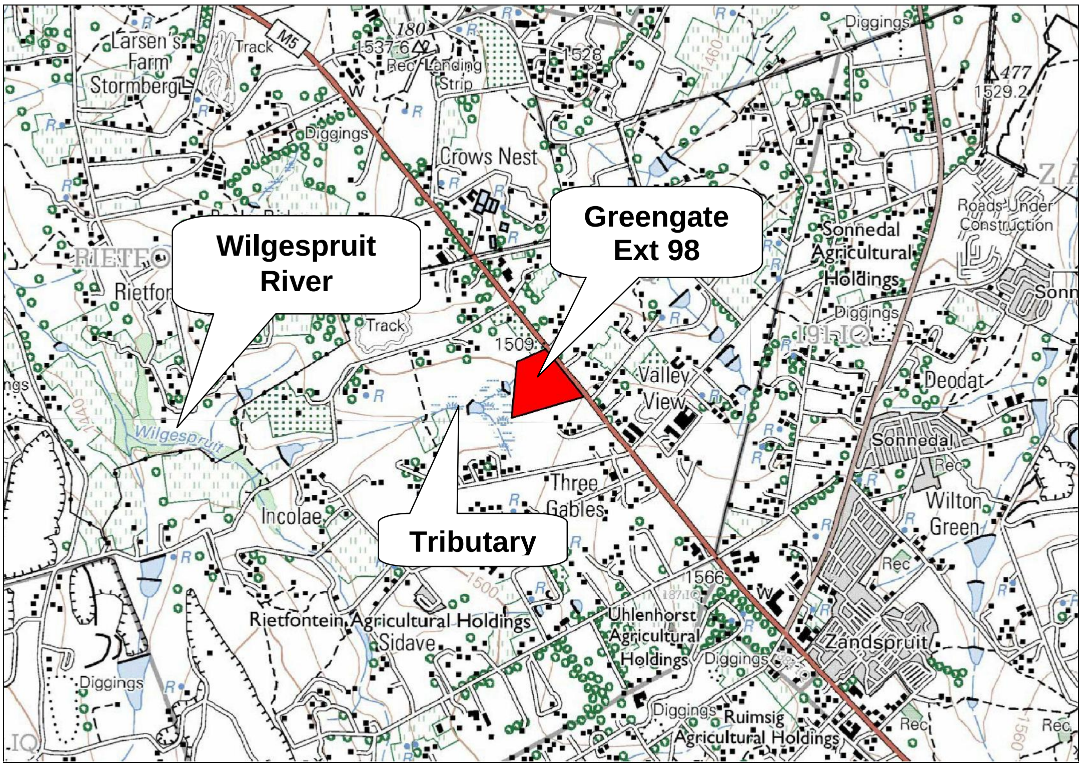


Figure 2: Extract from 1:50 000 topographical map (2627BB - 2007)

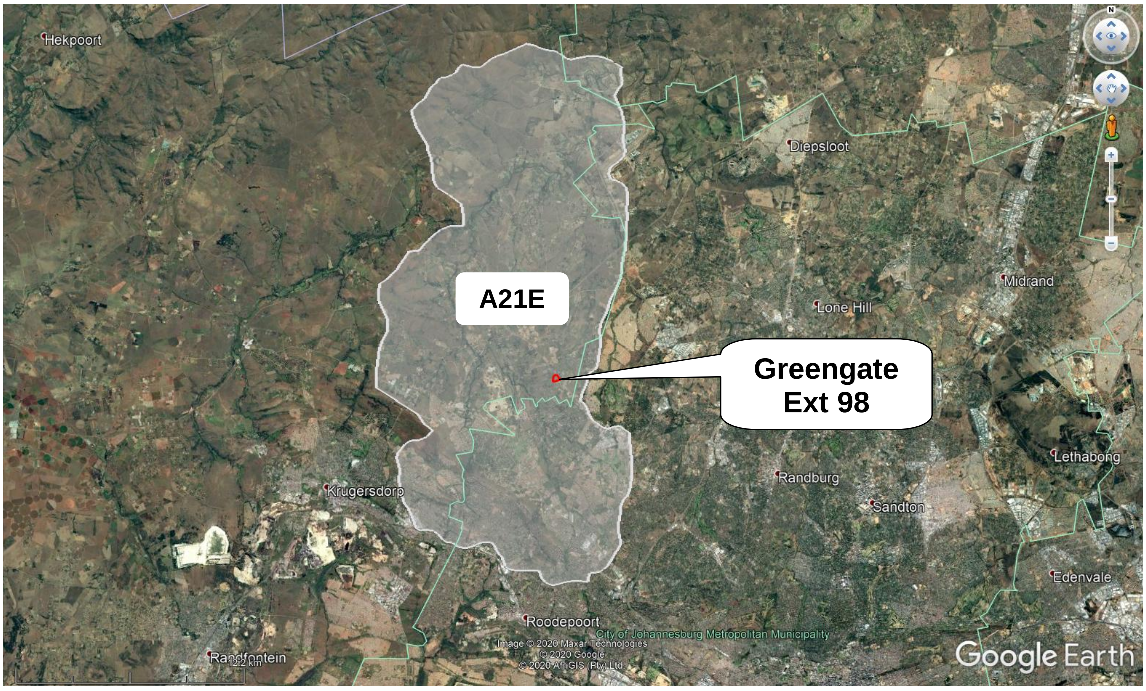


Figure 3: Quaternary Catchments A21E

5. HYDROLOGY

5.1 DESIGN RAINFALL

The design rainfall, summarized in Table 2, was compiled using the procedures to estimate design rainfall in South Africa developed by JC Smithers and RE Schulze. The design rainfall is also depicted graphically on Figure 4.

The Mean Annual Precipitation (MAP) was calculated at the site using this method as 671mm. The IDF coefficients were calculated, using linear regression, and summarised in Table 3.

Table 2: Point Rainfall (26° 3' S, 27° 53' E)

Storm Duration (min)	Return Period (Year)					
	1:2	1:5	1:10	1:20	1:50	1:100
5	10.0	13.6	16.2	18.8	22.5	25.5
10	15.0	20.3	24.2	28.1	33.6	38.1
15	18.8	25.5	30.4	35.3	42.2	47.8
30	23.9	32.4	38.6	44.8	53.6	60.7
45	27.4	37.3	44.3	51.5	61.6	69.8
60	30.3	41.1	48.9	56.9	68.0	77.0
90	34.8	47.3	56.3	65.4	78.2	88.6
120	38.2	51.8	61.7	71.7	85.7	97.1
240	45.3	61.6	73.2	85.2	101.8	115.3
360	50.1	68.1	81.0	94.2	112.6	127.5
480	53.9	73.1	87.0	101.2	120.9	136.9
600	56.9	77.3	91.9	106.9	127.8	144.7
720	59.6	80.8	96.2	111.9	133.7	151.4
960	64.0	86.8	103.3	120.1	143.6	162.6
1200	67.6	91.8	109.2	127.0	151.8	171.9
1440	70.0	95.0	113.0	131.4	157.1	177.9
1 Day	58.2	79.0	94.0	109.3	130.7	147.9

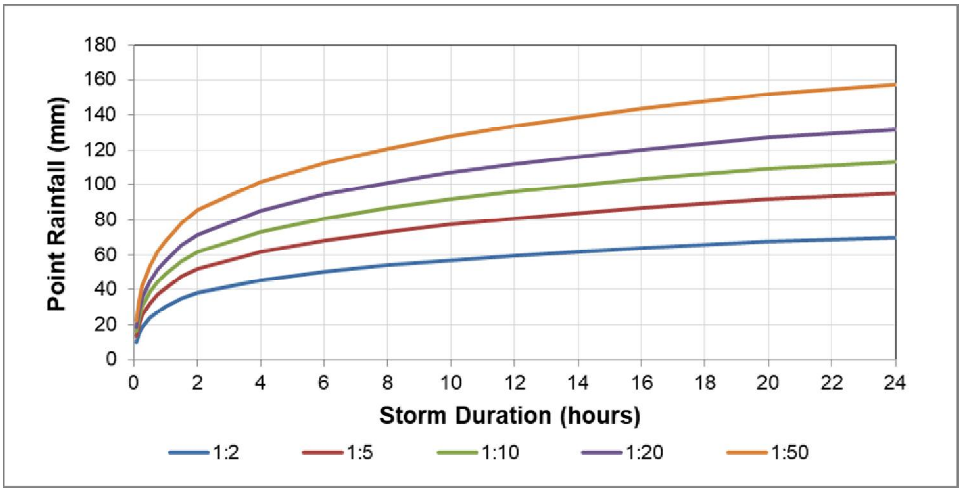


Figure 4: Depth-Duration-Frequency (DDF) curves

Table 3: IDF curve coefficients

Description	1in2	1in5	1in10	1in20	1in50	1in100
a	807	1095	1303	1515	1811	2051
b	6.26088					
c	0.77595					

5.2 SYNTHETIC DESIGN STORM

The Chicago design storm method was used to generate synthetic storms, using the IDF curve coefficients summarised in Table 3. Design storms with a total duration of 3 and 24 hours were generated respectively, using a storm advancement coefficient of 0.4. The mass curves of the synthetic storms are depicted on Figures 5 and 6.

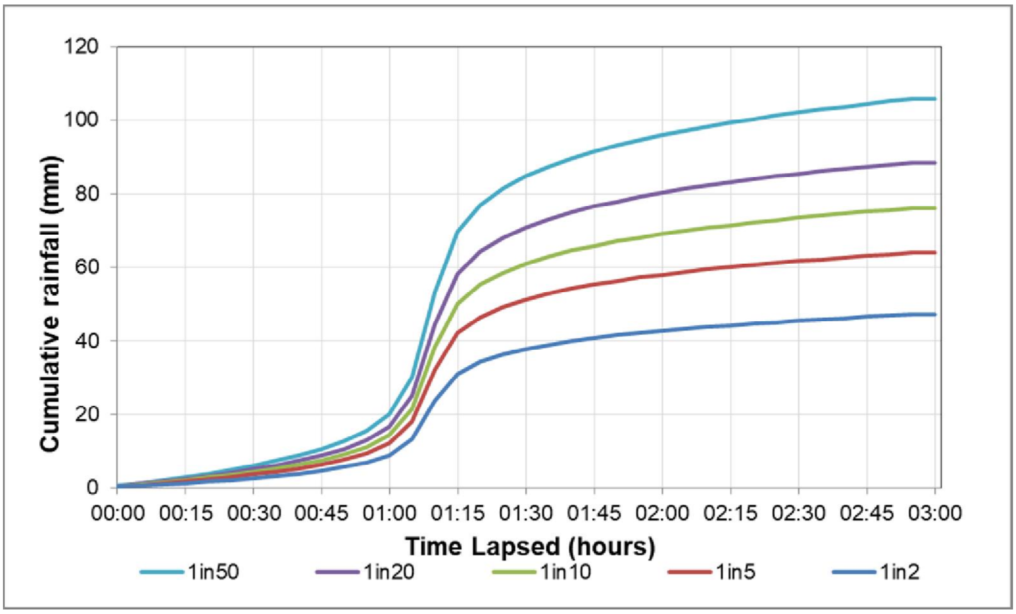


Figure 5: Synthetic storm mass curves for 3h storms

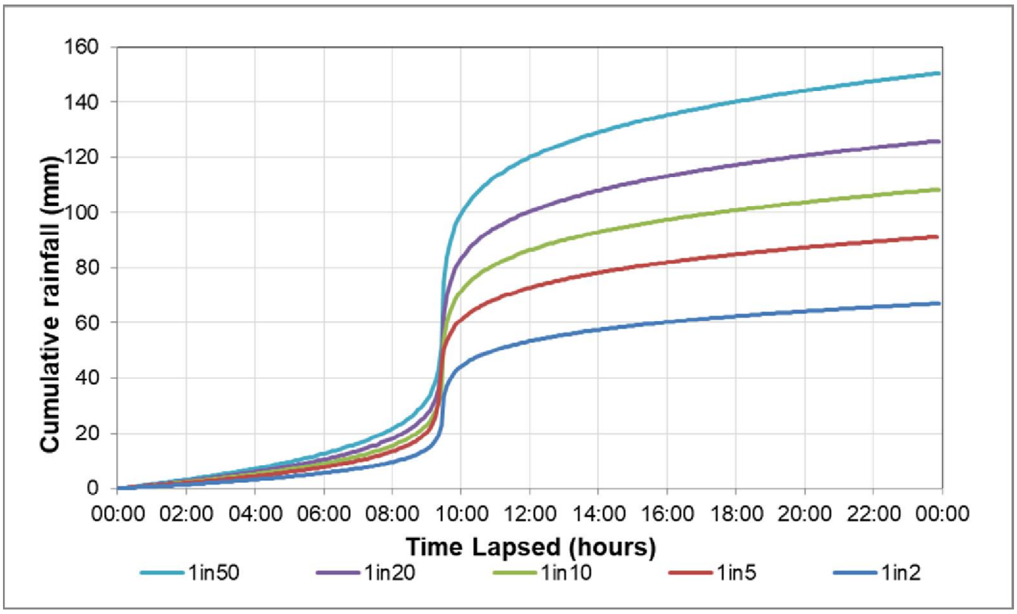


Figure 6: Synthetic storm mass curves for 24h storms

5.3 Soil characteristics

For the single event modelling, we used the Modified Green Ampt method for which the properties that needs to be considered are the hydraulic conductivity (κ), porosity (ϕ) and suction head (Ψ) of the soil.

A geotechnical study was done for the proposed development by Geotheta Consulting Engineers and Scientists, dated 19 December 2019, reference number 1911547/01. The test pits, 7 in total, were dug to the maximum reach of the TLB, ranging from 2.3m to 2.8m.

Only one test pit was excavated until refusal of the TLB on hardpan ferricrete at a depth of 1.7m blow natural ground. These holes were then profiled in accordance with the accepted practice, and samples taken for Laboratory analysis.

Sieve analyses were done on TP2 and TP7, at a depth of between 1.5m and 2.3m, and between 1.2m and 2.4m respectively. According to the actual sand/silt/clay compositions of TP2 and TP7, the soils can be classified, according to the soil texture triangle, as sandy-loam.

On the soil profiles, the upper layers are described mostly as silty-sand, but also as gravelly-silty-sand and silty-gravelly-sand. The soil profiles of all the test holes are attached as Annexure B.

According to the literature the infiltration parameters for sandy-loam soil are as follows:

- Hydraulic conductivity - 10.92 mm/h
- Suction head - 109.98 mm
- Initial deficit - 0.263 m^3/m^3.

The positions of the test pits are depicted on Figure 7, and the soil profiles of TP2 and TP7 are depicted on Figure 8.

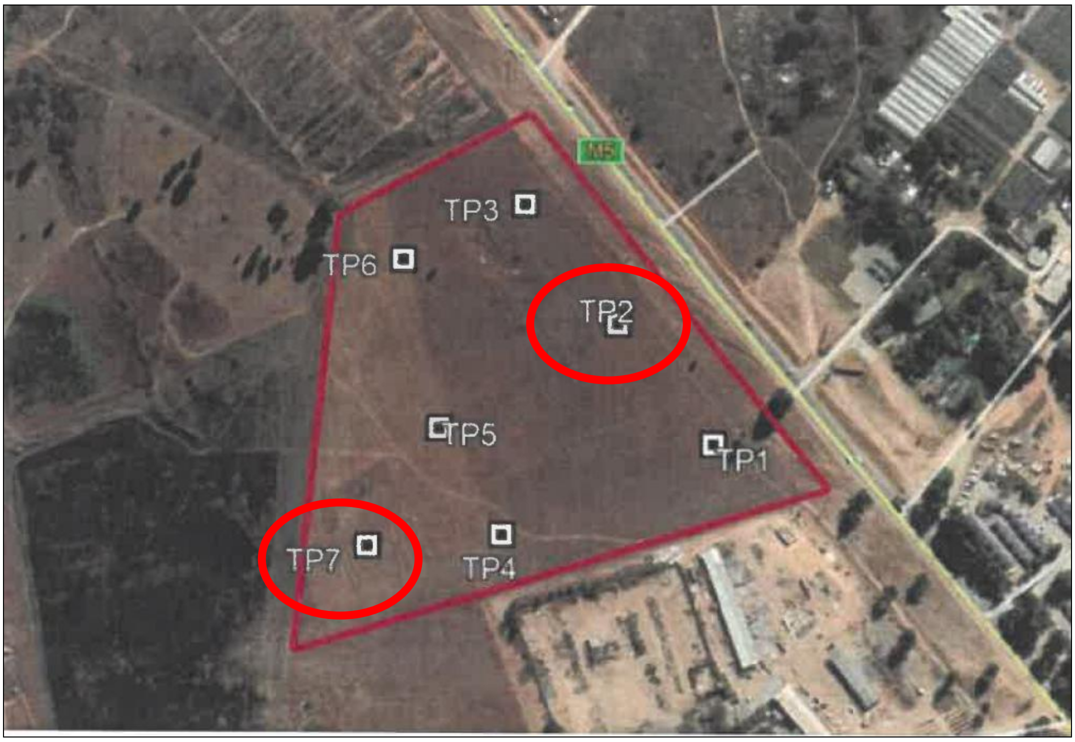


Figure 7: Test hole positions

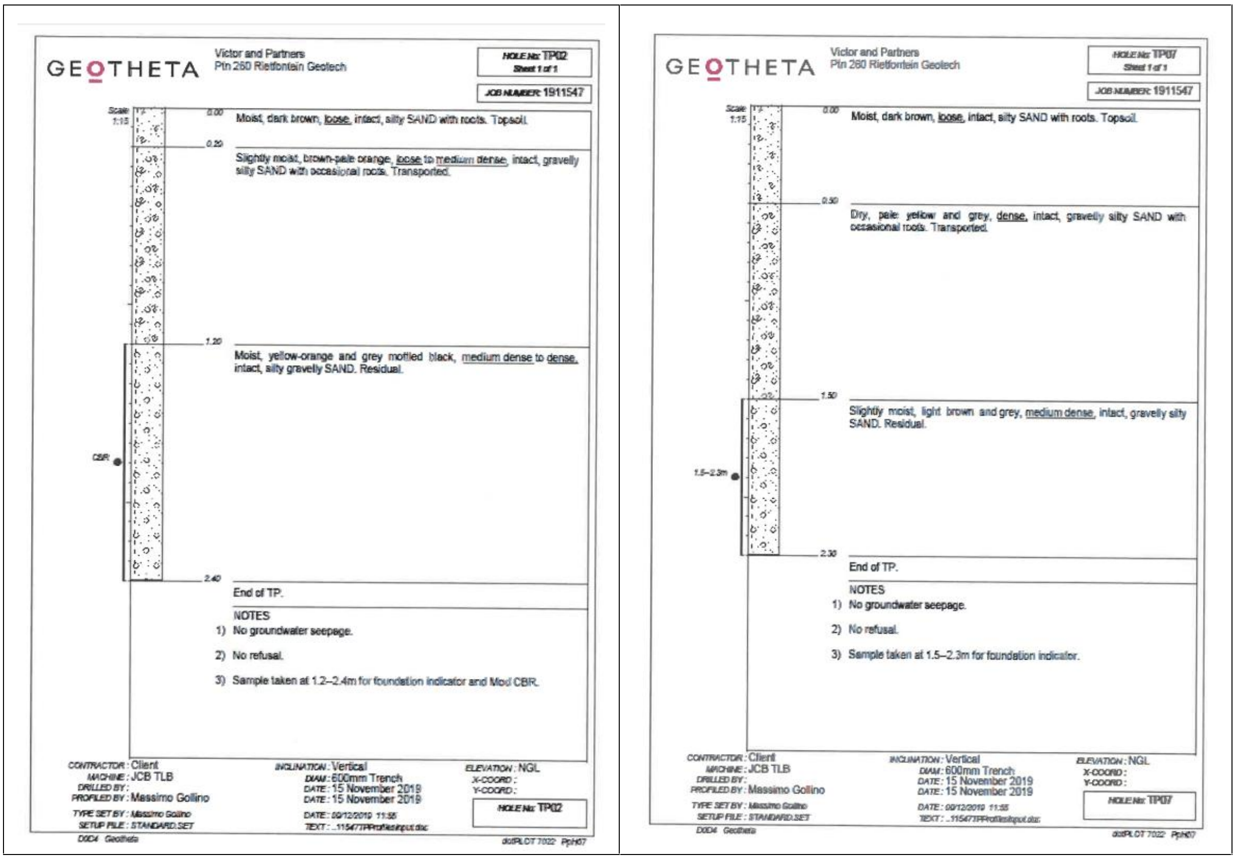


Figure 8: Soil profiles of TP2 and TP7

6. Pre Development Runoff

The pre development runoff was calculated using two deterministic methods, namely the Rational method and the SCS-SA method, as well as using the Stormwater Management Model (SWMM) to simulate a single synthetic storm.

6.1 Rational Method

A single catchment was assumed with an overland flow-path length limited to 150m, and the remainder of the drainage length assumed to be a natural drainage course.

The development is not subjected to stormwater runoff from higher lying areas. The runoff from the K31 can be diverted along the Northern boundary of the proposed development, towards the K56. The total area of the catchment areas is approximately 7.1 ha. The catchment area is depicted on Figure 9, and the characteristics are summarised in Table 4.

Table 4: Pre-Development catchment runoff

DESCRIPTION		***CHARACTERISTICS***					
Catchment area (km²)		0.071186					
Defined Watercourse (km)		0.165					
Overland flow (km)		0.150					
Roughness coefficient (r)		0.3 (Sparse grass over fairly rough surface)					
Height at 10% (m)		1502.0					
Height at 85% (m)		1517.0					
Ave. Slope (m/m)		0.069541					
Tc (hour) (minutes)		0.342 (21)					
Distribution	Rural	100%					
	Urban	0%					
	Lakes	0%					
Surface Slope	Flat Areas	100% (C=0.08)					
Permeability	Permeable	100% (C=0.08)					
Vegetation	Grassland	100% (C=0.21)					
Return period (year)		1in2	1in5	1in10	1in20	1in50	1in100
Rainfall (mm)		21.5	29.2	34.8	40.4	48.3	54.7
Average Intensity (mm/h)		63	85	102	118	141	160
Saturation (Flat and Permeable)		0.50	0.55	0.60	0.67	0.83	1.00
Final Runoff coefficient		0.185	0.204	0.222	0.248	0.307	0.370
Peak Runoff (m³/s)		**0.230**	**0.343**	**0.446**	**0.579**	**0.857**	**1.170**

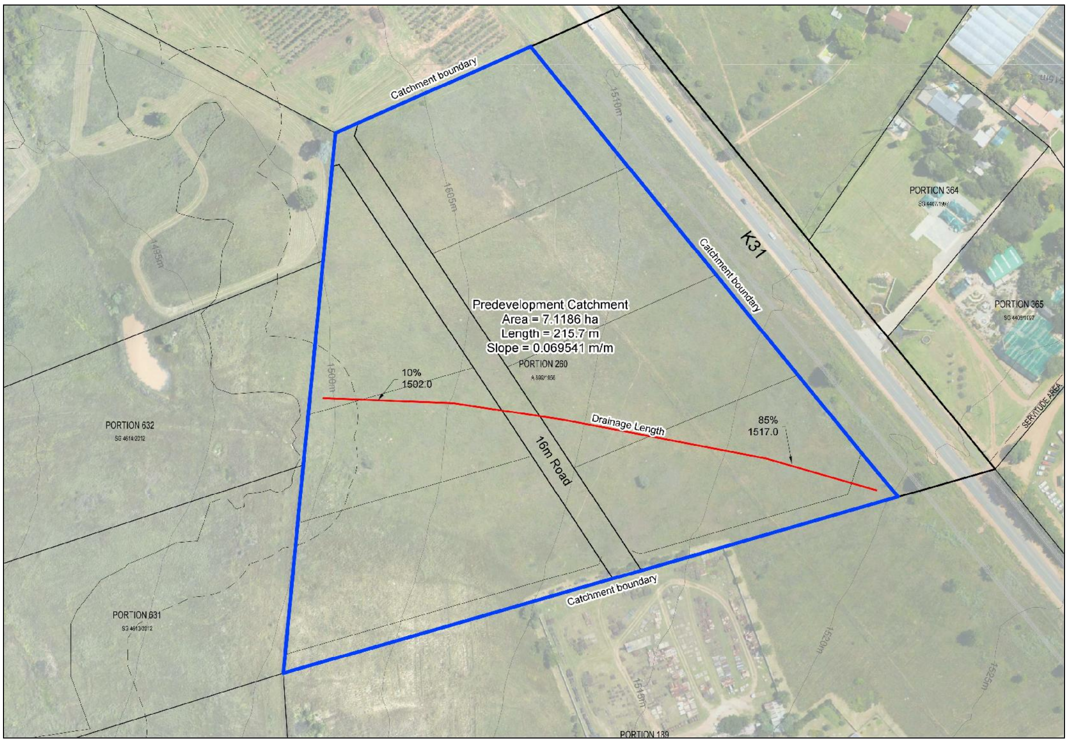


Figure 9: Pre-development catchment area

6.2 SCS-SA METHOD

The initial curve number for the pre-development condition is determined using the generalised SCS soil grouping classification for South Africa depicted in Figure 10. The proposed development is located between soil groups B and C, and therefore a soil group of B/C will be used.

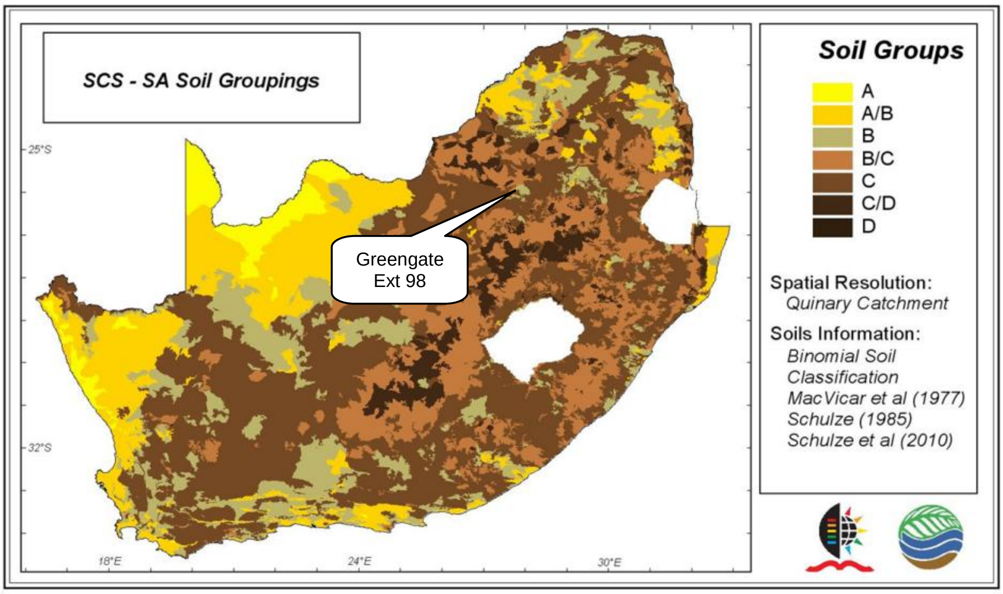


Figure 10: Generalized SCS soil grouping classification for South Africa

The initial curve number is also based on a land cover class, land treatment and stormflow potential, of which the following was selected:

- Land cover class - Veld (range) and pasture
- Land treatment - Veld/pasture in fair condition
- Stormflow potential - Moderate
- Initial curve number - 75

The adjustment of the initial curve number was done using the visual SCS-SA software, choosing the median condition method to account for typical antecedent moisture condition, with the following classes and intensity distribution type:

- Soil depth class - Deep soil
- Vegetation cover class - Intermediate
- Soil texture class - Sandy-loam
- Intensity distribution - Type 3
- Time of concentration - 0.342 hours (as calculated previously)

The peak discharge obtained from the Visual SCS-SA software, is summarised in Table 5.

Table 5: Peak discharge from the Visual SCS-SA software

Description	Return period (Years)					
	1:2	1:5	1:10	1:20	1:50	1:100
Design daily rainfall (mm)	58	79	94	109	131	148
Total runoff depth (mm)	14.7	27.0	36.9	47.6	64.3	77.8
Total stormflow volume (m^3x10^3)	1.0	1.9	2.6	3.4	4.6	5.5
Computed curve number	70.7	70.7	70.7	70.7	70.7	70.7
Peak discharge (m^3/s)	0.4	0.7	0.9	1.2	1.7	2.0

6.3 STORMWATER MANAGEMENT MODEL (SWMM)

The pre-development runoff using SWMM was calculated assuming a maximum overland-flow length of 150m. By the time runoff has travelled this distance it has consolidated into rivulets and therefore no longer behaves as overland-flow over a uniform plane. Based on this assumption, the catchment was divided into sub-areas with flow-path lengths of 150m or less. From Table 6, the weighted average slope is 6.99%, and dividing the total area by the 105m flow-path length, the width of catchment that was used for this simulation is 678m.

Using the synthetic storms generated using the CDS method, and the soil characteristics, the results of the simulations is summarised in Table 7.

Table 6: Flow lengths and slopes of undeveloped catchment

Sub-area	Flow-path length (m)	Associated area (ha)	Upstream elevation (m)	Downstream elevation (m)	Elevation difference (m)	Slope (%)
1	105	3.3371	1507.0	1499.0	8.0	7.62
2	105	2.8390	1514.0	1507.0	7.0	6.67
3	105	0.9434	1520.0	1514.0	6.0	5.71
Total	315	7.1195			21.0	
Average	105					6.67
Weighted Ave						6.99

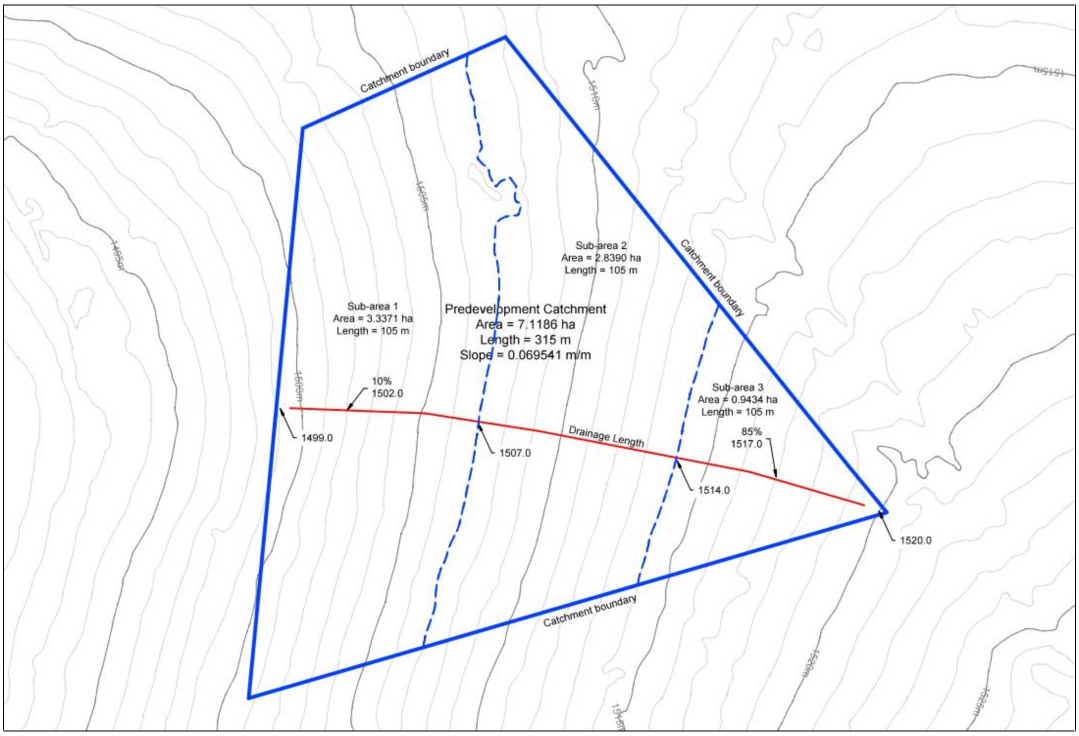


Figure 11: Sub-areas of undeveloped catchment

Two total durations for the synthetic storms were analysed, namely 3 and 24 hours respectively, in order to investigate if it has any significant effect on the peak discharge. It is therefore clear from the results in Table 7 that this had very little effect on the peak discharge.

Table 7: Maximum undeveloped runoff using SWMM

Storm characteristics	Maximum Runoff (m^3/s)				
	1in2	**1in5**	**1in10**	**1in20**	**1in50**
3 hour	0.732	1.446	1.970	2.508	3.247
24 hour	0.749	1.623	2.136	2.683	3.468

6.4 Concluding Thoughts on the Pre-Development Peak Runoff

Different results for the pre-development peak discharge were obtained from three different approaches.

The Rational method uses fixed discharge coefficients for different mean annual rainfall based on surface slope, permeability and vegetation. Gross assumptions also need to be made in terms of initial saturation, over which the designer has very little control.

On the other hand, the SCS-SA method use the median soil moisture status, determined using the five largest independent daily rainfall totals in each year of record. It therefore gives a much better account of the soil moisture content just prior to the selected storm event.

The results from the SWMM simulation for the undeveloped catchment provide the highest peak discharges for the predevelopment condition, using the soil characteristics as obtained from the geotechnical investigation.

The design of certain stormwater infrastructure for the immediate condition will most likely be based on the results of the Visual SCS-SA method, but for the sizing of the future attenuation pond, the results from the SWMM simulation will be used as guide to ensure that the post development peak discharge does not exceed the pre-development peak discharge.

The results for both the pre and post development peak discharge are therefore obtained from the same method, which therefore provides a much fearer and realistic volume per area ratio for the attenuation pond.

7. PROPOSED STORMWATER

7.1 GENERAL STORMWATER DRAINAGE

The stormwater network is designed in order to safely channel the runoff from a 1:10 year storm event, to the nearby Wilgespruit river tributary located adjacent to the Western boundary. The internal roads are provided with kerb inlets at strategic positions to catch stormwater runoff from the development as indicated on Figure 9. The underground system will consist of “Interlocking Joint” concrete pipes with a minimum diameter of 450mm with various slopes, the maximum velocities in the network is 5.6 m/s, and the manholes are provided with a maximum spacing of less than 100m between manholes. Please refer to the proposed stormwater layout attached as Annexure C.

7.2 NETWORK ANALYSIS

The network was analysed using a Manning roughness coefficeint of 0.013 for the concrete pipes. The results of the analysis for the 1:10 year storm event is summarised in Table 8.

Table 8: Hydraulic analysis

Description	Diameter	Length (m)	Slope (%)	Flow (m^3/s)	Velocity (m/s)	Flow depth (m)
J1 to J2	450 mm	75.5	4.68	0.432	3.974	0.293
J2 to J3	600 mm	16.2	6.94	0.823	5.641	0.312
J3 to O4	600 mm	73.6	5.78	0.811	5.318	0.315
J5 to J6	525 mm	70.6	2.13	0.510	2.794	0.405
J6 to J7	675 mm	15.2	2.30	0.980	3.589	0.492
J7 to O8	675 mm	31.5	1.51	0.967	3.265	0.513

7.3 SUB-CATCHMENTS

The proposed development was divided into 10 sub-catchment areas depicted in Figure 12, with their characteristics summarised in Table 9. The soil infiltration parameters determined previously as well as generalised parameters used for all the sub-catchments are summarised as follows:

- Hydraulic conductivity 10.92 mm/h
- Suction head 109.98 mm
- Initial deficit 0.263 m^3/m^3
- Impervious manning roughness 0.020
- Impervious depression storage 0.000
- Pervious manning roughness 0.100
- Pervious depression storage 0.000
- Zero-Imperv Storage 0.000
- Routed runoff 100 %

Table 9: Post-development sub-catchment characteristics

Name	Outlet	Area (ha)	Width (m)	Slope (%)	Impervious Area %	Impervious Area (ha)
C1	J1	0.9290	63.2	5.43	80	0.7432
C2	J2	0.9230	63.4	5.98	80	0.7384
C3	J5	0.9550	66.2	5.87	80	0.7640
C4	J6	0.9950	72	5.08	80	0.7960
C5	O9	1.1320	76.4	7.21	80	0.9056
C6	O4	0.6780	72.1	7.16	80	0.5424
C7	O10	0.6710	99	5.91	80	0.5368
C8	J1	0.0790	19.4	5.17	80	0.0632
C9	J5	0.2330	16.3	3.97	80	0.1864
C10	J6	0.1300	18.1	1.09	80	0.1040
Total		6.7250			80	5.3800

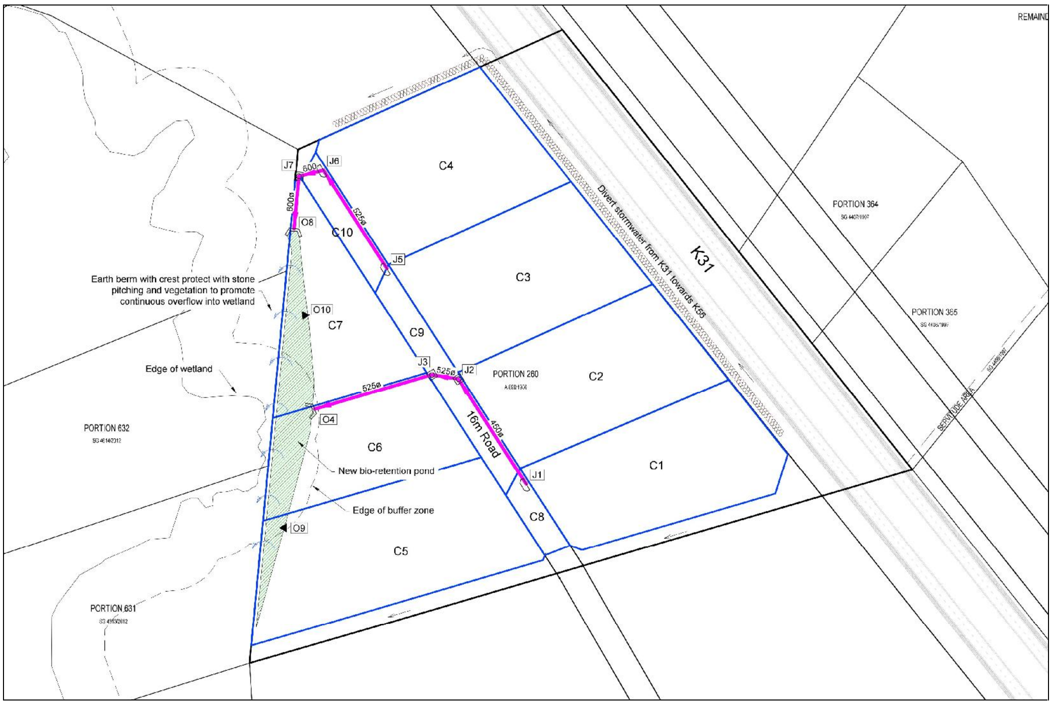


Figure 12: Post development sub-catchments

7.4 SUSTAINABLE URBAN DRAINAGE PRINCIPLES (SUDS)

The underground drainage system will be discharging into a new bio-retention swale/pond to be constructed along the western boundary of the proposed development, within the 32m buffer zone.

The swale/pond will increase the stormwater infiltration, reduce the peak discharge, and it will add a visually aesthetic component to the development. The pond's dimensions will be finalized during the detail design stage. However, preliminary indications are that a 1.2m deep pond will be sufficient.

The pond will consist of various weir overflows, particularly sized to distribute the discharge along the total length of the western boundary, but will also be adequately sized to accommodate the peak runoff from a 1:25 year storm event.

Energy dissipaters will be constructed at the outlets into the swale/pond in order to minimize any potential scouring.

7.5 POST DEVELOPMENT PEAK RUNOFF

The peak discharge for each individual sub-catchment is summarized in Table 10, and the total peak discharge into the swale/pond is summarized in Table 11. The total indicated in Table 11 is however exclusive of routing through the network and is therefore slightly higher than the total of Table 10.

The hydrographs at the four outlets are depicted on Figure 13.

Table 10: Post development catchment runoff

Name	Runoff (m^3/s)				
	1in2	**1in5**	**1in10**	**1in20**	**1in50**
C1	0.226	0.330	0.402	0.476	0.579
C2	0.228	0.331	0.403	0.477	0.579
C3	0.235	0.343	0.417	0.493	0.599
C4	0.244	0.356	0.433	0.512	0.622
C5	0.283	0.411	0.500	0.590	0.716
C6	0.181	0.260	0.313	0.367	0.442
C7	0.183	0.261	0.314	0.367	0.442
C8	0.025	0.034	0.040	0.047	0.056
C9	0.060	0.087	0.106	0.126	0.153
C10	0.034	0.049	0.060	0.071	0.086
Total	1.699	2.462	2.988	3.525	4.275

Table 11: Maximum discharge into the bio-retention swale / pond

Name	Maximum discharge (m^3/s)				
	1in2	**1in5**	**1in10**	**1in20**	**1in50**
O4	0.635	0.923	1.120	1.323	1.603
O8	0.544	0.794	0.967	1.129	1.439
O9	0.280	0.407	0.494	0.584	0.708
O10	0.181	0.258	0.310	0.364	0.437
Total	1.641	2.382	2.892	3.399	4.187

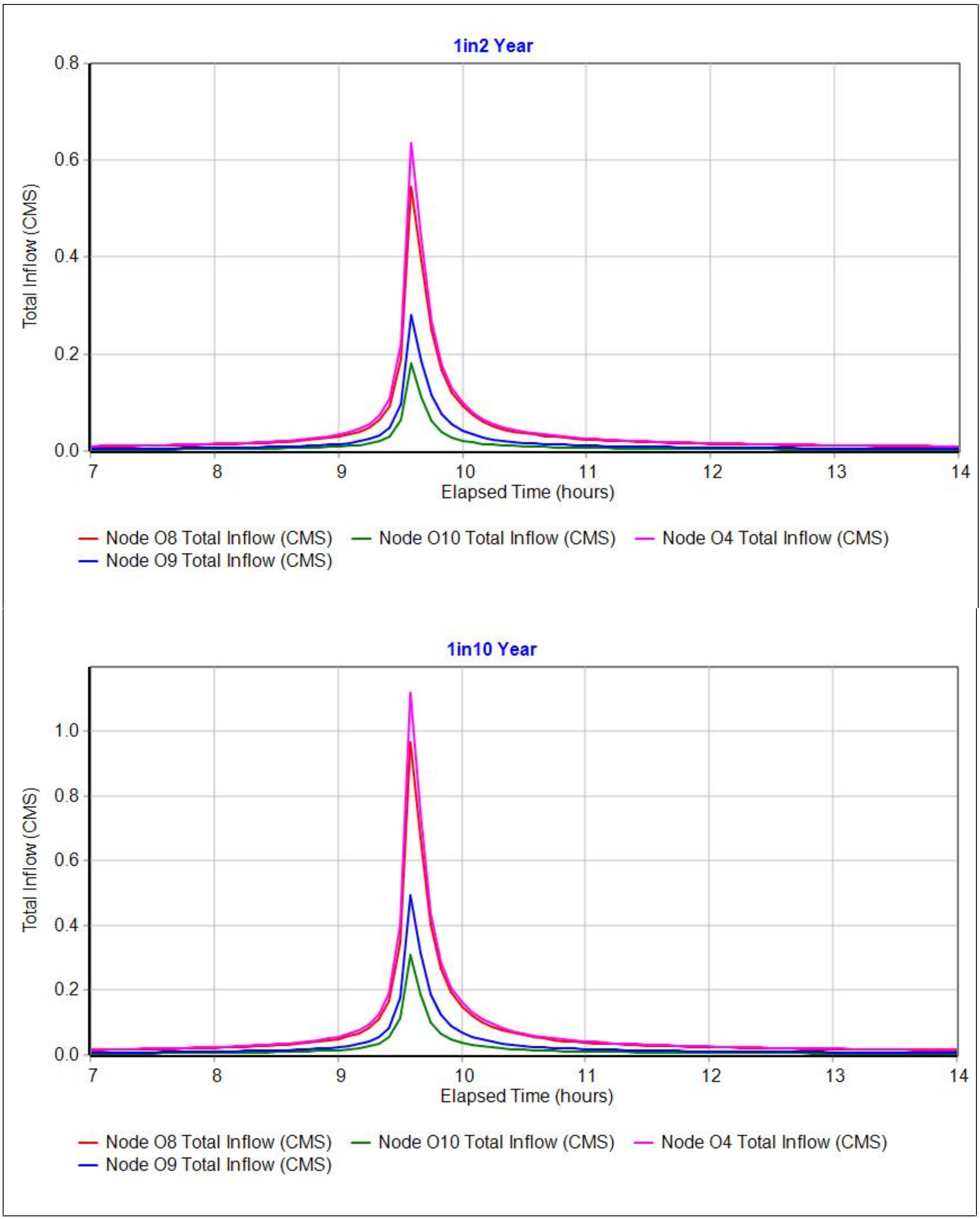


Figure 13: Hydrographs at outfalls

7.6 ATTENUATION

A routing analysis was performed using the procedure described in the Drainage Manual, where the continuity equation î∆t + Ô∆t = ∆S and the auxiliary function N = S/ ∆t + O/2, in order to calculate the required storage. The calculations were based on the summation of the four inflow hydrographs depicted on Figure 13. The characteristics of the proposed attenuation pond, which were used for this analysis, are as follows:

- Rectangular orifice:
  - Size of opening - 600 x 200 mm
  - Offset above pond invert - 300 mm
- Broad crested weir:
  - Total length - 10 m
  - Offset above pond invert - 700 mm
- Pond:
  - Depth - 1.2 m
  - Maximum Volume (1:10y) - 1764 m³
  - Volume / area ratio - 248 m³/ha

Please also refer to the typical outlet structure attached as Annexure E. The inflow and outflow hydrographs for the proposed attenuation pond for the 1:10 year storm event is depicted on Figure 14.

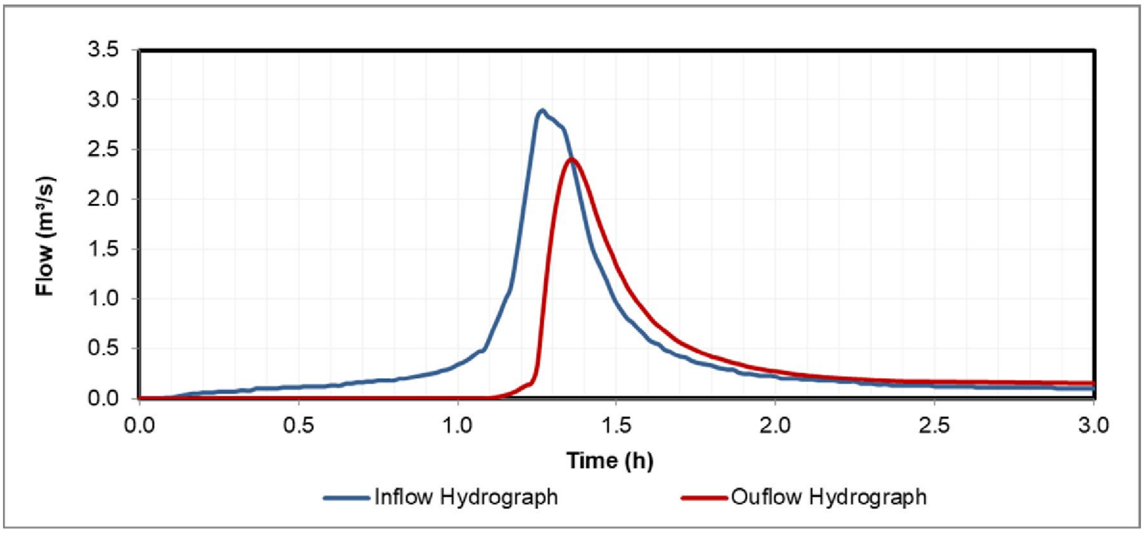


Figure 14: Inflow and outflow hydrographs (1:10 year)

7.7 Runoff volumes

The runoff volumes for the total catchment area of 7.1 ha, for the pre and post-development conditions were calculated from a continuous simulation model.

In the absence of rainfall data closer to the site, the rainfall data for the Johannesburg Botanical Gardens (BOT) were used for this simulation. BOT is located approximately 17 km south of the proposed development. The annual rainfall for BOT is summarized in Table 12.

The pre-development runoff volume was calculated using a single catchment area with the infiltration characteristics as calculated previously, whereas the post-development runoff volume was calculated using the proposed stormwater network comprising of 10 sub-catchment areas as defined previously.

The average total pre-development runoff is therefore 1,630 m³ per annum, whereas the post-development is 32,830 m³ without a bio-retention cell, and by introducing a bio-retention cell, the post-development runoff volume is 8,407 m³. This is equivalent to volume runoff coefficients of 0.033, 0.780 and 0.180 respectively. The pre-development runoff is slightly higher than the average of 0.020 for the entire A21E quaternary catchment, which could be attributed to the fairly high slope of the natural ground level. The post development runoff is considerable higher than the pre development condition due to an increase of 80 % of impermeable surfaces, which result in the drastic increase of 1914% in runoff volume, but this can be reduce to 416% by the introduction of a bio-retention cell. This needs to be re-evaluated once the site development plan is compiled.

This dramatic increase in runoff volume is attributed to the nullification of the infiltration capacity due to impermeable surfaces. With permeable surfaces the infiltration capacity exceeds the low intensity rainfall of the majority of the rainfall, but the infiltration capacity becomes effectively zero when impermeable surfaces are introduced.

It is therefore important to introduce a bio-retention swale / pond in order to reduce the volume of surface runoff and to replenish the ground water levels instead, and to reduce the peak discharge at the same time. A similar simulation was therefore conducted using a bio-retention cell with the following properties:

- Area of bio-retention cell - 2000 m²
- Berm height - 200 mm

- Soil layer:
  - Thickness - 300 mm
  - Porosity - 0.5
  - Field Capacity - 0.1
  - Wilting point - 0.05
  - Conductivity - 30
  - Conductivity slope - 10
  - Suction head - 60 mm
- Storage layer:
  - Thickness - 300 mm
  - Void ratio - 0.3
  - Seepage rate - 10
- Drain ignored

The volume runoff results of the pre-development as well as the post-development conditions, including both scenarios, with- and without a bio-retention cell is also summarized in Table 12.

Table 12: Pre and post-development runoff volumes

Year	Annual Rainfall (mm)	Pre-development		Post-development			
				Without Pond		With Pond	
		Vol (m^3)	C	Vol (m^3)	C	Vol (m^3)	C
Jul '95 to Jul '96	961	2,900	0.04	53,490	0.78	16,751	0.24
Jul '96 to Jul '97	1027	2,190	0.03	56,973	0.78	17,777	0.24
Jul '97 to Jul '98	494	1,780	0.05	27,544	0.78	7,455	0.21
Jul '98 to Jul '99	607	540	0.01	33,551	0.78	4,641	0.11
Jul '99 to Jul '00	549	3,820	0.10	30,913	0.79	13,973	0.36
Jul '00 to Jul '01	359	300	0.01	19,798	0.77	2,999	0.12
Jul '01 to Jul '02	195	70	0.01	10,716	0.77	1,262	0.09
Jul '02 to Jul '03	432	230	0.01	23,857	0.78	3,590	0.12
Jul '03 to Jul '04	468	410	0.01	25,743	0.77	4,379	0.13
Jul '04 to Jul '05	522	2,140	0.06	29,198	0.79	8,122	0.22
Jul '05 to Jul '06	525	110	0.00	28,896	0.77	2,380	0.06
Jul '06 to Jul '07	376	510	0.02	20,817	0.78	3,390	0.13
Jul '07 to Jul '08	795	1,950	0.03	44,102	0.78	12,008	0.21
Jul '08 to Jul '09	589	500	0.01	32,384	0.77	7,760	0.19
Jul '09 to Jul '10	932	7,640	0.12	53,044	0.80	18,855	0.28
Jul '10 to Jul '11	876	4,390	0.07	49,136	0.79	17,418	0.28
Jul '11 to Jul '12	287	260	0.01	15,884	0.78	3,117	0.15
Jul '12 to Jul '13	418	130	0.00	23,064	0.78	2,004	0.07
Jul '13 to Jul '14	457	850	0.03	25,351	0.78	4,145	0.13
Jul '14 to Jul '15	769	1,720	0.03	42,897	0.78	11,874	0.22
Jul '15 to Jul '16	756	1,790	0.03	42,068	0.78	12,645	0.23
Average	590	1,630	0.03	32,830	0.78	8,407	0.18

8. FLOOD LINES

The proposed development is certified to be not affected by the 1:100-year flood lines as per the provision of Section 144 of the National Water Act, 1998 (Act 36 of 198). A copy of the floodline certificate is attached as Annexure D.

9. CONCLUSION

From the report it is evident that the stormwater runoff from the proposed development will be safely channeled from the higher lying areas to lower lying areas. New stormwater kerb inlets will be constructed at strategic positions along the internal roads. The runoff will be conveyed in an underground stormwater drainage system consisting of interlocking joint concrete pipe and discharging into a new bio-retention pond / swale located adjacent to the Western boundary of the proposed development. The underground network will be discharging into the bio-retention pond / swale at two different outfall points located within the buffer zone. The stormwater discharge from the pond into the wetland will be from a number of weirs distributed along the length of the Western boundary.

The peak discharges for the 1:10 year pre- and post-development conditions are 1.970 and 2.890 m^3/s respectively. The mean annual runoff volumes from a continuous simulation model were found to be 1,630 and 32,830 m^3 for the pre- and post-development conditions respectively. Introducing a bio-retention pond / swale will reduce the post development runoff volume to 8,407 m^3.

The proposed bio-retention pond / swale was also analysed as an attenuation pond. The required storage is 248 m^3/ha in order to ensure the post development runoff for the 1:10 year runoff does not exceed the pre development runoff.

From the hydraulic analysis, it is also evident that the proposed pipe network will have sufficient capacity to accommodate the peak discharge from a 1:10 year storm event. Adequate energy dissipation must also be provided at the outlets to prevent any possible scouring.

10. References

1. Red Book, The guidelines for human settlement planning and design volume.*CSIR, 2000.*

2. Drainage Manual. *: The South African National Roads Agency Limited, 2006.*

3. Water Sensitive Urban Design (WSUD) for South Africa: Framework and guidelines. *: Water Research Commission, 2014.*

ANNEXURE A

(Township layout)

ANNEXURE B

(Soil profiles)

ANNEXURE C

(Proposed stormwater layout)

ANNEXURE D

(Floodline certificate)

ANNEXURE E

(Typical outlet structure)