FUNCTIONAL SERVICING
AND
STORMWATER MANAGEMENT REPORT

12705 Old Kennedy Rd
Caledon, Ontario

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1.0 Scope of work

Ram Engineering Inc. is authorized to undertake the preparation of a Functional Servicing and Stormwater Management Report for the proposed development of 12705 Old Kennedy Rd in Caledon. The property is proposed to be divided into three residential lots.

The site is located on the east side of Old Kennedy Road in a predominantly residential neighbourhood, north of the Highway 410 and east of Highway 10 in Caledon.

This Functional Servicing and Stormwater Management Report (FSR & SWM Report) showed that it is feasible to service the lands for the proposed residential use. This report details the implementation of the design criteria that takes into consideration the policies and guidelines of the Town of Caledon.

The property is being severed into three lots under separate ownership. Hence, the FSR & SWM Report s is done separately for each lot.

This FSR & SWM Report covers the grading, storm-water management, sanitary and water supply for the development of the site.

In this report, it is shown that the site can be serviced with the following:

- adequate stormwater management to meet the City standards and guidelines, discharging to existing outlets at the rear of the property at controlled levels and the front yard ditch.
- wastewater management (sanitary), discharging to an existing 250mm sanitary sewer on Old Kennedy Rd, south side; and
- water supply from a 200mm watermain on Old Kennedy Road.
- Water supply for Fire protection from a Hydrant on Old Kennedy Rd.

The location of the site is shown In Figure 1.

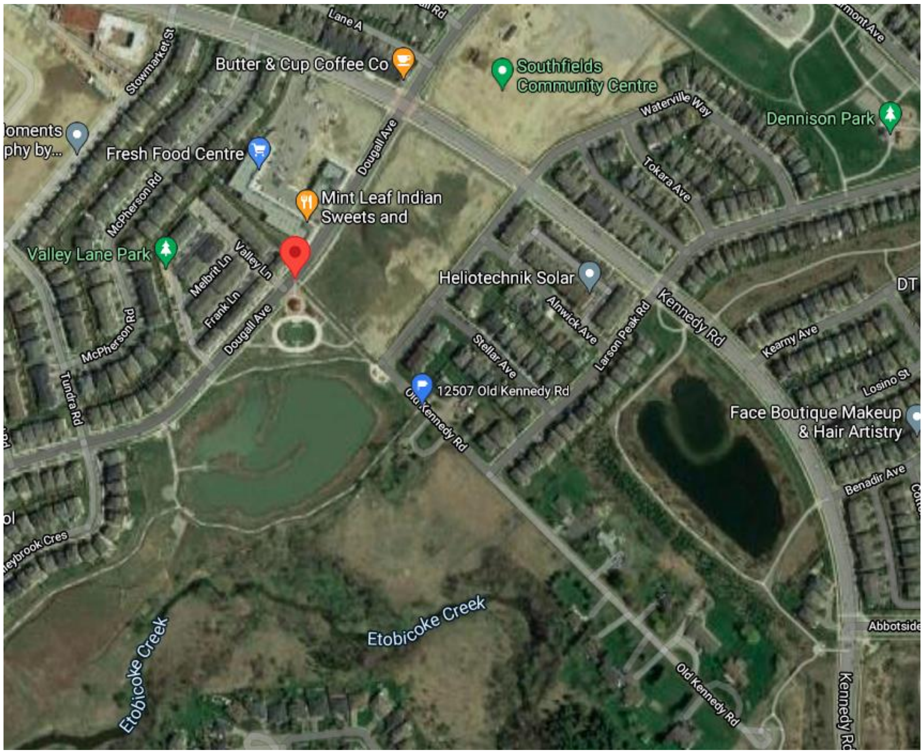


Figure 1: Location of Site

2.0 The Existing and Proposed Land use

The site has a development area of 0.20 ha. In the developed condition, all of the property is utilized. Currently a two storey house exist in the middle part of the property. There is a garage at the rear of the property.

The lands are relatively flat, generally sloping to Old Kennedy Rd. The rear part of the property drains to the back of the property.

The site bounded by townhouses along Waterville Way to the north, single family residences along Stellar Ave. to the east and single family residences to the south.

Etobicoke Creek is approximately 200m southwest of the property. Storm water quality and quantity ponds (E2 and E3) are servicing the surrounding lands.

There is no direct connection to stormwater sewers for eh site. The flows to the rear of the property goes to a sewer system that goes to Pond E3, while the front of the property goes vai a ditch to Pond E2.

The Ponds are part of the drainage plan for MOScorp VII Development Inc. Phase 1 prepared by DSEL (David Schaeffer Engineering Limited).

With regards to sanitary services, a connection is made to an existing manhole on Old Kennedy Road. Similarly, for water supply, a watermain on Old Kennedy Rd will provide individual loy connections to each lot.

2.1 Existing Land use

The site is historically used for residential purposes The existing grading of the land is shown in Figure 2 (topographic survey).

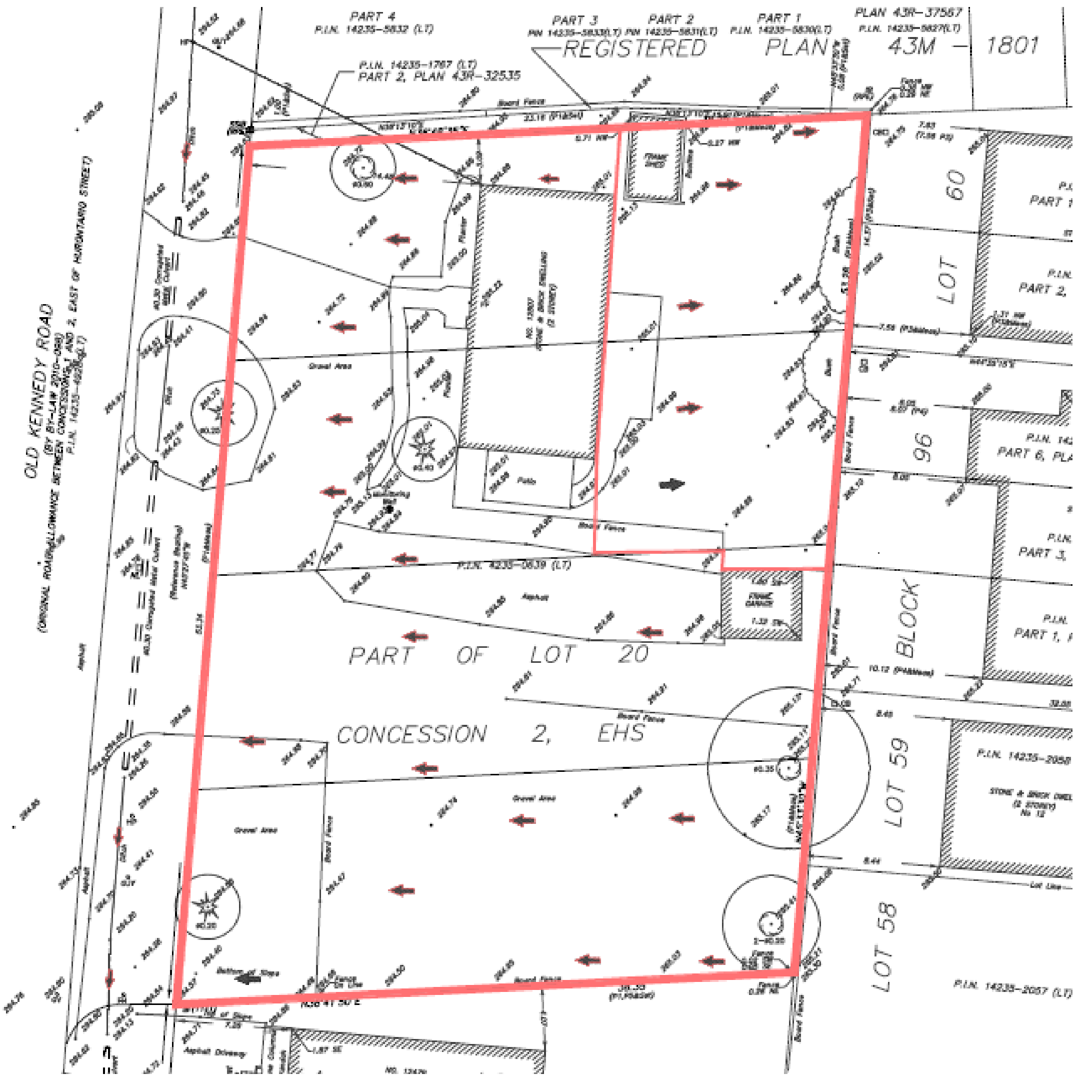


Figure 2: Existing Development and drainage

2.2 Proposed Land use

The proposed land-use is shown in Figure 3.

Figure 3: Proposed Development

The breakdown in proposed land-use is as follows:

Table 1: Landuse for proposed development

Land use	Area Ha)
Landscape	0.1018
Roof	0.0752
Pavers	0.0253
Runoff coefficient	0.2023

3.0 Stormwater Management for the development

All stormwater system designs for water quality and quantity controls will be in accordance with the MOE SWM Planning and Design Manual (2003) and Town of Caledon SWM Policies and Guidelines.

All stormwater calculations are based on the Town of Caledon rainfall data and the Rational method to calculate runoff. Where runoff volumes are required, the MTO IDF Look-up rainfall data is used as the town data does not give rainfall depths.

3.3 Rainfall Data

The Town of Caledon rainfall intensities are used in the calculations of runoff from the site. For The intensity is calculated with the following relationship:

$$i = \frac{A}{(t_c + B)^C}$$

Where i = rainfall intensity corresponding to the 100 year level, mm/hr
t_c = time of concentration, 10minutes

Table 2 below shows the coefficients for all storms.

Table 2: Rainfall data

Storm	A	B	C	Td	Intensity (mm/hr)
2	1070	7.85	0.7989	10.0	81.10
5	1593	11	0.8789	10.0	109.68
10	2221	12	0.9080	10.0	134.16
25	3158	15	0.9355	10.0	155.47
50	3886	16	0.9495	10.0	176.19
100	4688	17	0.9624	10.0	196.54

For rainfall depths, the MTO IDF data for Caledon are given in Table 3.

Table 3: Rainfall data

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	10.5	12.9	14.6	17.9	22.1	27.2	37.9	46.7	57.5
5-yr	13.8	17.0	19.2	23.6	29.1	35.9	49.9	61.5	75.7
10-yr	16.0	19.7	22.2	27.4	33.7	41.5	57.8	71.2	87.7
25-yr	18.7	23.1	26.1	32.1	39.6	48.8	67.9	83.7	103.1
50-yr	20.8	25.6	28.9	35.6	43.9	54.1	75.3	92.7	114.3
100-yr	22.8	28.1	31.8	39.1	48.2	59.4	82.7	101.8	125.5

3.1 Design Criteria

The Township requires stormwater management for developing the site such that there are minimal potential impacts on local flooding with the implementation of mitigation measures. The conversion of pervious land to impervious surfaces results in increased rate and volume of stormwater runoff, reductions in infiltration and groundwater recharge and reduction of evapotranspiration. Consequently, uncontrolled stormwater runoff can lead to increased flooding, degraded water quality, and hydrologic modifications. Hence, properly designed SWM measures are required to mitigate these impacts.

The Town of Caledon advocates a design that protect and improve surface water quality and the implementation of stormwater management facilities/ measures that are efficient and compatible to Low Impact Development stormwater management practices and principles.

The Town of Caledon requires that the flows that will be released to the ditch to the front of the property along Old Kennedy Road, and two catchbasins to the rear of the site. To achieve this flooding restriction, the following stormwater management criteria are applied to the site:

- 100 year Flood mitigation for development of the lands as runoff would increase as a result of the intensified urban re-development of the site;
- Release rate to the rear catchbasins be limited to the 10 year post development flows;
- Release rate to the ditch in front be also limited to the 10 year post development flows
- Erosion control during construction; and
- Water quality in accordance with the Ministry of Environment for Enhanced Level 1.

3.2 Allowable discharge from the site

The Town undertook a stormwater plan for the area, and came up with the following allowable discharge:

- The rear catchbasins are to receive flows for 0.050 hectare of the site at a runoff coefficient of 0.63 to the stormsewer system draining to Pond E3; and
- The remaining part of the site being part of an area of 0.28 hectare at a runoff coefficient of 0.50 will drain to a front yard ditch which ultimately discharges to Pond E2.

This is illustrated in Figure 4. The drainage plan is extracted from the drainage plan for MOScorp VII Development Inc. Phase 1, Drawing No. 12 prepared by DSEL (David Schaeffer Engineering Limited).

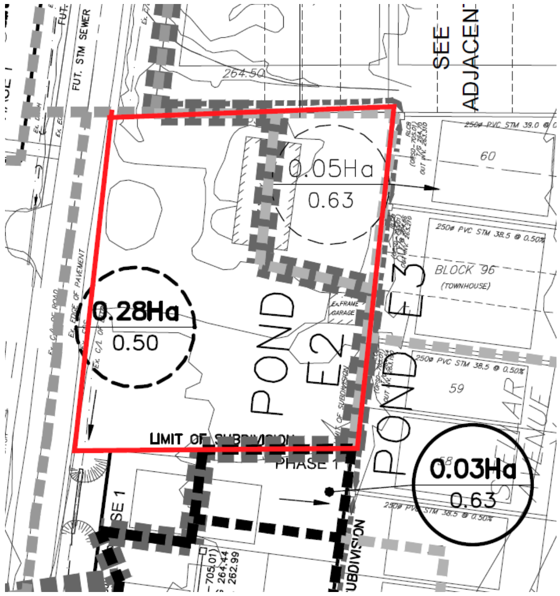


Figure 4: Proposed Development

<u>Permissible discharge to rear lot catchbasins</u>

Allowable drainage area	0.050	ha
10 year rainfall intensity	134.16	mm/hr
Runoff coefficient	0.63	
100 year Allowable Flow to Pond E3	0.012	m^3/s

<u>Permissible discharge to front ditch</u>

Area as part of 0.28 ha draining to Pond E2

Total drainage area (0.2025 - 0.050ha)	0.153	ha
Runoff coefficient (as per DSC plan)	0.50	
10 year rainfall intensity	134.16	mm/hr
10 year Allowable flow	0.0284	m^3/s

3.4 Proposed Drainage

The proposed drainage will mimic the existing drainage pattern, and take into consideration the external flows, as follows:

- The part of the site that will not be controlled due to the grading and natural slopes of the lands (a part of the front area);
- The area in the developed part of the site that will drain to the existing rear catchbasins;
- The area draining the front ditch
- All flows will be controlled to the allowable limit as detailed in Section 3.2.

The lands are to be severed into three lots, and each lot will have to manage their own stormwater management. Hence, mitigation measures are independent of each lot.

The proposed drainage areas which are considered in the stormwater options are shown in Figure 5.

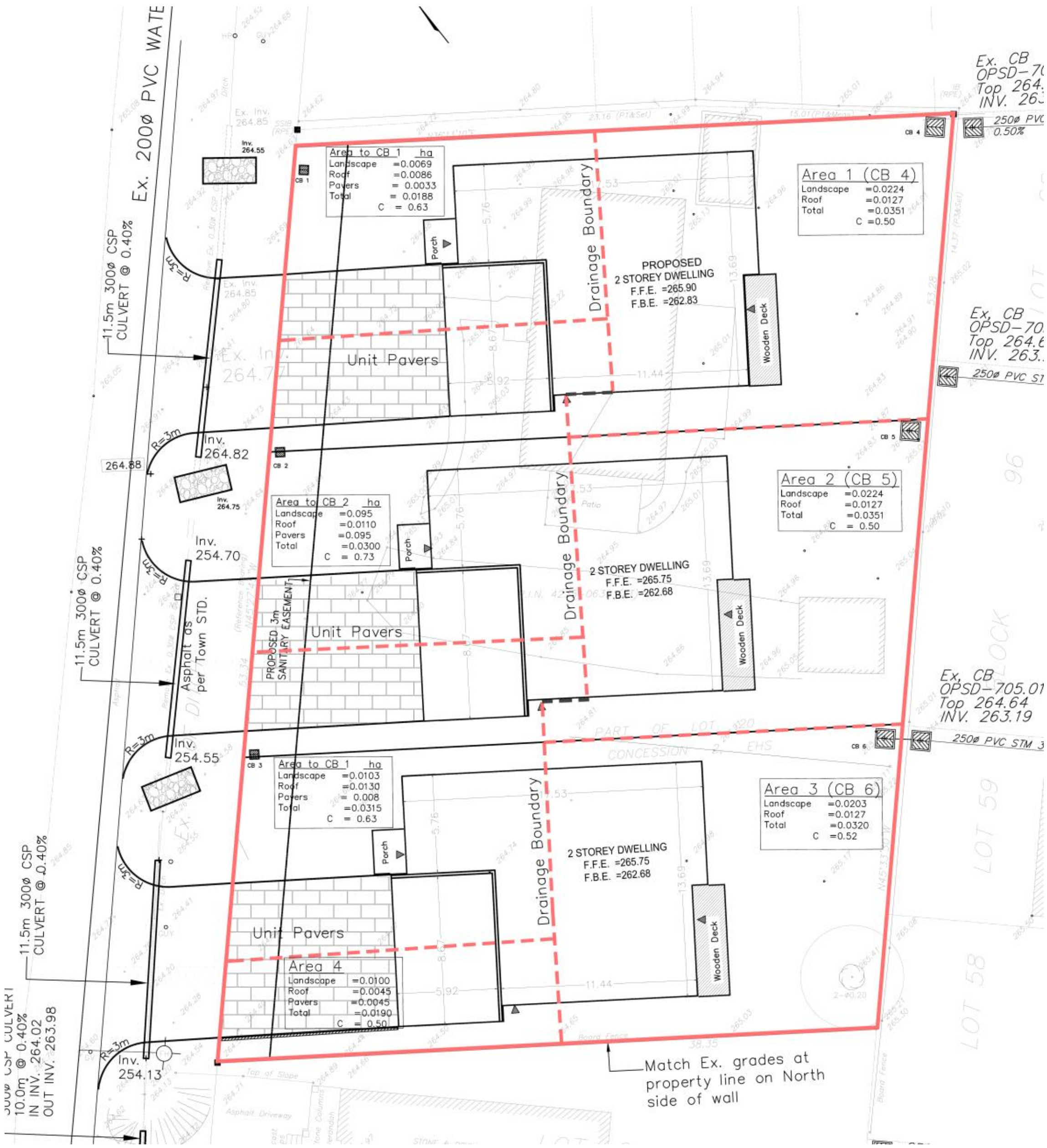


Figure 5: Proposed SWM plan

The full details of each drainage area are detailed in Table 4.

Table 4: Drainage areas for developed condition

Flow direction	Part	Landscape (ha)	Roof (ha)	Pavers (ha)	Total (ha)	C
	Controlled flows					
Flow to back	Area 1	0.0224	0.0127		0.0351	0.50
	Area 2	0.0224	0.0127		0.0351	0.50
	Area 3	0.0203	0.0127		0.0330	0.52
	Total	0.0651	0.0381		0.1032	0.51
Flow to Front	Area to CB 3	0.0103	0.0130	0.008	0.0313	0.63
	Area to CB 2	0.0095	0.0110	0.0095	0.0300	0.62
	Area to CB 1	0.0069	0.0086	0.0033	0.0188	0.63
	Total	0.0267	0.0326	0.0208	0.0801	0.63
	Uncontrolled flows					
	Area 4	0.0100	0.0045	0.0045	0.0190	0.50
	Total	0.1018	0.0752	0.0253	0.2023	0.55

3.4.1 Proposed Drainage to the front ditch

The drainage to the Old Kennedy Road ditch would be managed with infiltration beds. Area 4 will be uncontrolled.

Uncontrolled flows

The uncontrolled flow to the ditch is determined, and the reduced allowable runoff is calculated as detailed in Table 5.

Table 5: Drainage areas for developed condition

Uncontrolled area - 100 yr post development	0.0190	ha
Runoff coefficient	0.50	ha
100 year rainfall intensity	196.54	mm/hr
Uncontrolled Peak flow	0.0052	m^3/s
Adjusted allowable flow		
10 yr Allowable	0.0284	m^3/s
100 yr Uncontrolled	0.0052	m^3/s
Reduced allowable flow	0.0232	m^3/s

The flows to the three Catchbasins (CB 1, CB 2 & CB 3) are distributed as follows to optimize the storage needed to attenuate the post development flows to the desired levels, as shown below.

100 yr flow to CB 1	0.0055	m^3/s
100 yr flow to CB2	0.0087	m^3/s
100 yr flow to CB 3	0.0091	m^3/s
Total	0.0232	m^3/s

The storage for the three areas tributary to the catchbasins are determined as follows:

Area to CB 1	0.38	m^3
Area to CB 1	0.51	m^3
Area to CB 1	0.62	m^3
Total storage	1.51	m^3

The storage is provided in infiltration trenches. The process for sizing the trenches is detailed in Section 3.4.2.

The storage is calculated with the Modified Rational Method. This method is applied as explained in **<u>"Urban Stormwater Management Special Report No. 49", American Public Works Association, 1981 (pages 52 to 56).</u>** The calculations are appended to this report.

3.4.2 Design of infiltration trenches

MOECC and the Conservation Authorities recommends the use of infiltration facilities as an acceptable subsurface method to mitigate runoff from a site. On this site, this option is adopted for each of the three lots.

<u>Storage media and surrounding soils</u>

Infiltration trench/beds require an acceptable percolation rate of the underlying soil. Fisher Engineering in their Geotechnical Investigation dated November 23, 2020 investigated the suitability of the soil for that purpose.

The Geotechnical investigation found that the top layer of the soil (up to 450mm deep) is a dark silty sand with some gravel. Thereafter, there is a silty sand fill for an average depth of 500mm. Further down is native silty sand.

<u>Groundwater level</u>

Seepage/groundwater was assessed as part of the Geotechnical Investigation. The report states that "No measurable amount of water was found in Boreholes 2,5 and 6 on completion of the respective soil borings". In fact, groundwater in the other boreholes were far deeper than the proposed 1.2m depth of the infiltration bed. Hence, the groundwater would not affect the infiltration of the stormwater runoff.

Percolation rate

The Geotechnical report states as follows:

"*Results indicate that on-site native soils predominantly consist of sandy silt till soils of compact relatively density in the upper portion. Estimated permeability of the native compact sandy silt till soils is anticipated to be in the range of 10^{-5} cm/sec or less. Estimated percolation time is expected to be 30 min/cm*".

The percolation rate is calculated as 20mm/hr. This rate is used to determine the sizing and effectiveness if the proposed infiltration trenches.

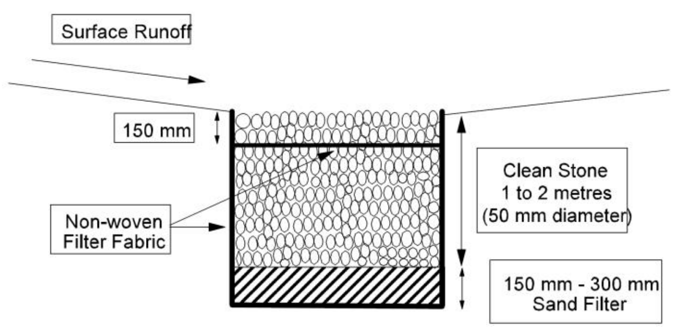
MOEE Recommended design

The design of infiltration trench is outlined in MOEE SWM Planning and Design Manual (2003), Equation 4.3 (MOECC).

$$A = \frac{1000V}{Pn\Delta t}$$

Where A = Bottom area of trench (m2)
V = runoff volume to be infiltrated
P = Percolation rate od surrounding native soils (mm/hr)
n = porosity of storage media (0.4 for clear stone)
Δt = retention time (24 to 48 hours)

The typical cross section is shown below.



3.5 Flood mitigation for flows to the rear catchbasins

The areas draining to the rear catchbasins are divided into three parts to reflect the ownership of each lot, as follows:

Area 1	0.0351	ha	$Q_{allowable}$	0.00399	m^3/s
Area 2	0.04	ha	$Q_{allowable}$	0.00399	m^3/s
Area 3	0.0330	ha	$Q_{allowable}$	0.00375	m^3/s
Total	**0.1032**	**ha**	$Q_{allowable}$	**0.01174**	$\mathbf{m^3/s}$

For each area, the runoff will be reduced to the allowable limits with an irrigation tank and an infiltration bed.

3.5.1 Stormwater management for Area 1

Area 1 will drain to the catchbasins at the rear of the property from where the flows are conveyed to Pond E3. The areas that form part of Area 3 are as follows:

- Landscape 0.0224 ha
- Roof 0.127 ha

Storage tank

Storage will be in an irrigation tank and an infiltration trench, with a total release of 0.00399 m^3/s. The tank will have a capacity of $1.5m^3$; hence, the discharge from the roof will be set to optimize the storage of $1.5m^3$.

The calculations show that $0.0035m^3/s$ will have to be release from the roof so that an optimum storage of $1.5m^3$ of storage is utilized. The details of the storage calculations are provided in the Appendix of this report.

Infiltration bed

The infiltration bed will accept the runoff from the roof ($0.0035m^3/s$) and the runoff from the landscape areas, and provide enough storage to release to the allowable limits for Arae 1.

The storage required in the infiltration bed is calculated with the Modified Rational Method, and is determined to be $2.18m^3$.

The calculations appended to this report shows that a 1.2m x 2.5m x 1.2m deep will be adequate for infiltrating the runoff the drainage area.

3.5.2 Stormwater management for Area 2

Area 2 will drain to the catchbasins at the rear of the property from where the flows are conveyed to Pond E3. The areas that form part of Area 3 are as follows:

- Landscape 0.0224 ha
- Roof 0.127 ha

The storage in the tank, the release rate from the tank, the storage in the infiltration bed and the release rate from the infiltration bed are the same as Area 1.

3.5.3 Stormwater management for Area 3

Area 3 will drain to the catchbasins at the rear of the property from where the flows are conveyed to Pond E3. The areas that form part of Area 3 are as follows:

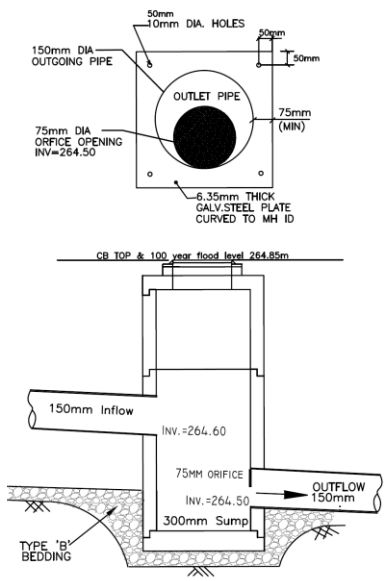
- Landscape 0.0203 ha
- Roof 0.127 ha

Storage in an irrigation tank will be the same as Area 1 and 2 as the runoff is expected to be the same from a similar sized drainage area.

The landscape area is marginally different. The storage required in the infiltration trench is determined as 1.86m^3. The storage provided is 2.18m^3.

3.6 Control of flows to Rear catchbasins

The post development runoff is attenuated to the allowable release rate with an orifice. Figure 5 shows the outflow configuration of the orifice.



CB 1, 2 & 3
Orifice Plate Details
(NTS)

Figure 6: Orifice plates

The typical orifice equation is used to size the lower orifice.

$$Q = CA\sqrt{2gh}$$

Where Q = discharge (m^3/s)
g = gravity (9.81 m^2/s)
A= area of orifice (m^2)
C= orifice coefficient (0.62 for a plate)
h= head (m)

For Area 1, 2 and 3, the head on the orifice is the same, as follow:

Top of CB	264.85	m
Invert of outflow	264.50	m
Orifice size	0.075	m
Head	0.31	m

Substituting in the equation, the size of the orifice is 58mm,

Q =	0.0040	m^3/s
C =	0.62	
h=	0.31	m

$Q = C\,A\,(2gh)^{0.5}$

A=	0.0026007	m^2
d=	58	mm

However, a 75mm orifice will be used as the Town requires a minimum of 75mm.

Similar calculations show that a 75mm orifice at the outflow pipe to Infiltration bed1, 2 and 3 would be required to control the outflow to the 10 year storm to the ditch on Old Kennedy Road.

3.7 Control of flows to front ditch from catchbasins

The process used in determining the size of the orifice is the same as those for the rear catchbasins. Likewise, a 75mm orifice late is used.

3.8 Post development discharge from the site

The post development discharge from the site is meeting the allowable discharges to the back and front of the property.

In the front ditch, the total discharge will be 0.0252 m^3/s, corresponding to the 10 year allowable at a runoff coefficient of 0.50. The total storage in three infiltration beds is 1.51 m^3/s.

In the rear of the property, the total discharge is 0.112m^3/s. corresponding to the 10 year at a runoff factor of 0.63 allowable runoff. Three storage tanks with 1.5m^3 each, and three infiltration beds with a 2.3m^3 storage are provided for each lot.

4.0 Water Quality

Quality control is required by the Town of Caledon and MOECC. The design is based on Guidelines for provincial water quality standards, “MOE Water Management Policies; Guidelines, Provincial Water Quality Objectives 2003” ("The Blue Book") and Provincial Policy Statement "Water Quality and Quantity"

The site has a small area earmarked as the driveway which will be paved with pervious stones. Hence, minor runoff is expected. Further, the runoff goes into a ditch which will provide water quality, as the sediments and some contaminants are expected to be deposited in the ditch. The flow will go to Pond E2 which is a quantity and quality facility.

In consideration of the above, no stormceptor is considered for the site.

5.0 Erosion and Sediment Control

During construction, there would be the potential for erosion to take place and the movement of sediment from the site to take place. To minimize this concern, the following measures should be implemented:

1. All sediment and siltation control measures shall be implemented by the contractor before the start of construction, where it is appropriate.
2. A silt fence as per the Town of Caledon standard should be installed around the perimeter of the site.
3. A mud mat should be placed at the entrance/ access point of the construction site. The contractor should follow the Town of Caledon standard for its design and implementation.
4. Area drains should be protected by sediment catchbasin barriers as per the City standards.
5. If the construction is delayed for more than 30 days, all bare areas should be seeded.
6. The municipal road should be kept clean of mud and dirt; it should be cleaned at least once a week, or as required by the Town of Caledon.

The construction engineer should inspect the erosion and sediment control devices frequently and after major storm events to make sure that they are functional. Repairs should be done as appropriate.

6.0 Sanitary servicing

Wastewater from the project area is accounted for in the “Sanitary Drainage Plan” prepared by David Schaeffer Engineering Ltd (Dwg. No. 9) for the MOSCORP V11 Development Inc., Phase 1. The plan shows that for the 0.20ha of the site, discharge is permitted for a population density of 39 persons per hectare, which is 8 persons. Figure 7 shows the drainage area data.

For the three houses, and an Ontario average family of 2.9 persons, the total population expected is 9 which is 1 more than allowed. This is a marginal increase in the flow.

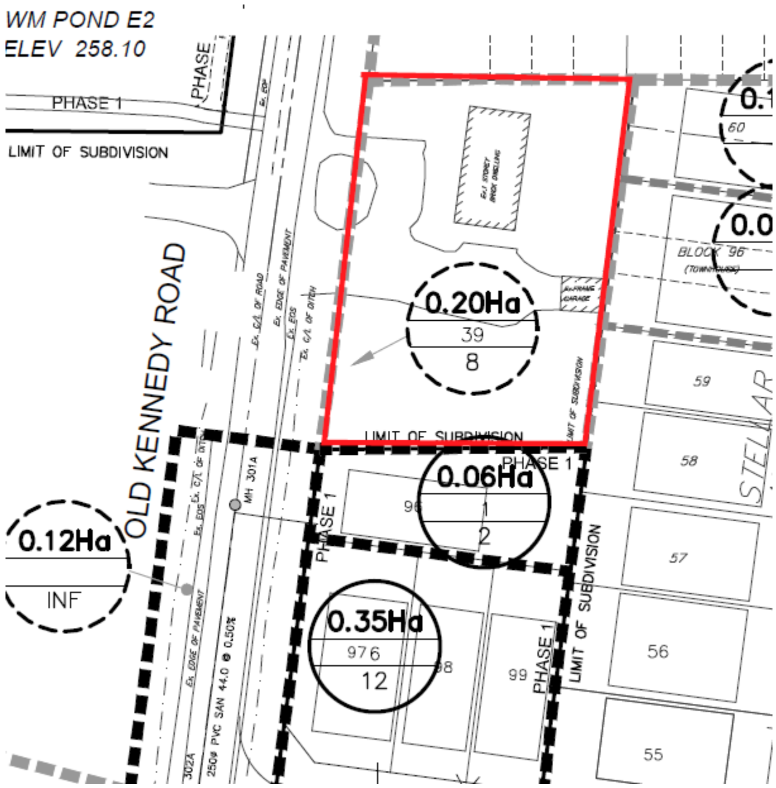


Figure 7: Sanitary Drainage Area Plan

The sanitary connection is made to existing Sanitary Manhole 3A to the site. A 3m easement is provided at the front of the property for a 200mm sanitary sewer to serve all there lots with 125mm sanitary connections as shown in Figure 8.

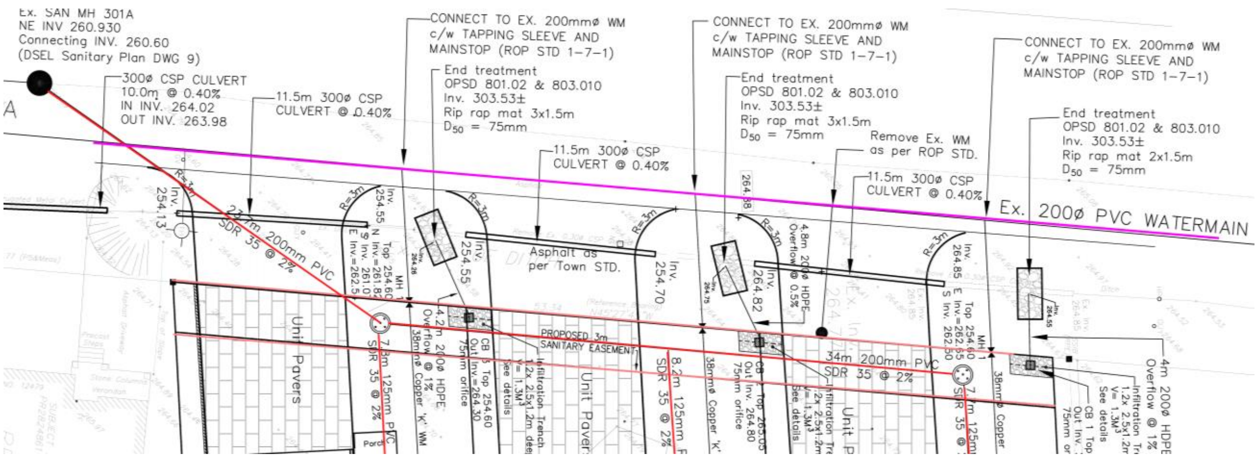


Figure 8: Sanitary connection to the lots

7.0 Water supply

The domestic water for the three lots will be obtained from a 200mm watermain (by others) on Old Kenedy Road. There should be a confirmation that this 200mm watermain is installed. If its not so, then the owner should open a discussion with the city/Region to have this watermain installed.

There are three 38mm connection to each of the house within the development.

With regards to fire demand, water supply will be accessed from an existing hydrant as per the David Schaeffer Engineering Ltd (Dwg. No. 9) for the MOSCORP V11 Development Inc., Phase 1 (Region of Peel No. 52503-D). The location of the hydrant is shown in Figure 9.

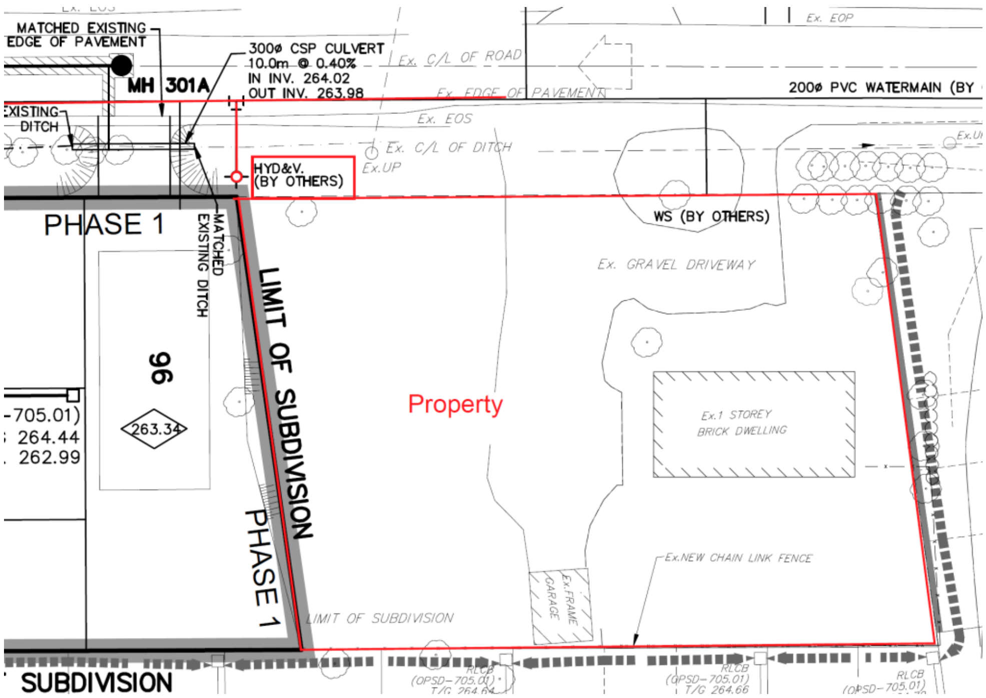


Figure 9: Sanitary connection to the lots

8.0 Summary

The Functional Servicing and Stormwater Management Report for 12705 residential development has addressed the key requirements as stated in the Design Criteria. These are:

Stormwater management

- Adequate storage on site is provided for flows to the rear of the property going to Pond E3 and to the front going to Pond E2
- 75mm orifice outlet controls are proposed at each catchbasins to achieve the 10 year control for the flow from the development;
- Infiltration beds are used to mitigate the increased runoff for each lot, at the front and back of the houses.
- Three tanks are proposed to use the back portion of the roof to irrigate the landscape.
- No water quality measures are proposed as the driveways are will be constructed with pervious stones and the areas a minor.

Sanitary servicing

Sanitary services are provided a 200mm sewer in a proposed 3m easement. This pipe s connected to an existing sanitary manhole on Old Kennedy Road.

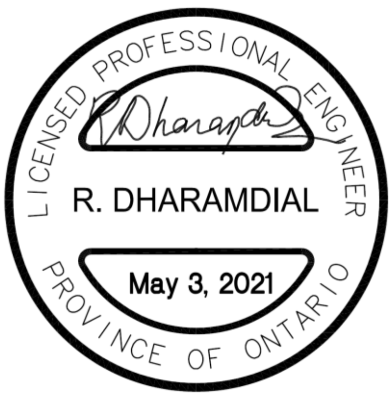
Water supply

Domestic water supply is obtained from a 200mm watermain. However, it should be verified this this watermain is installed.

Water for fire fighting is obtainable from a hydrant near to the south property line.

This report shows that all services can be provided to the three lots so that they can be developed into three houses.

Respectfully submitted,



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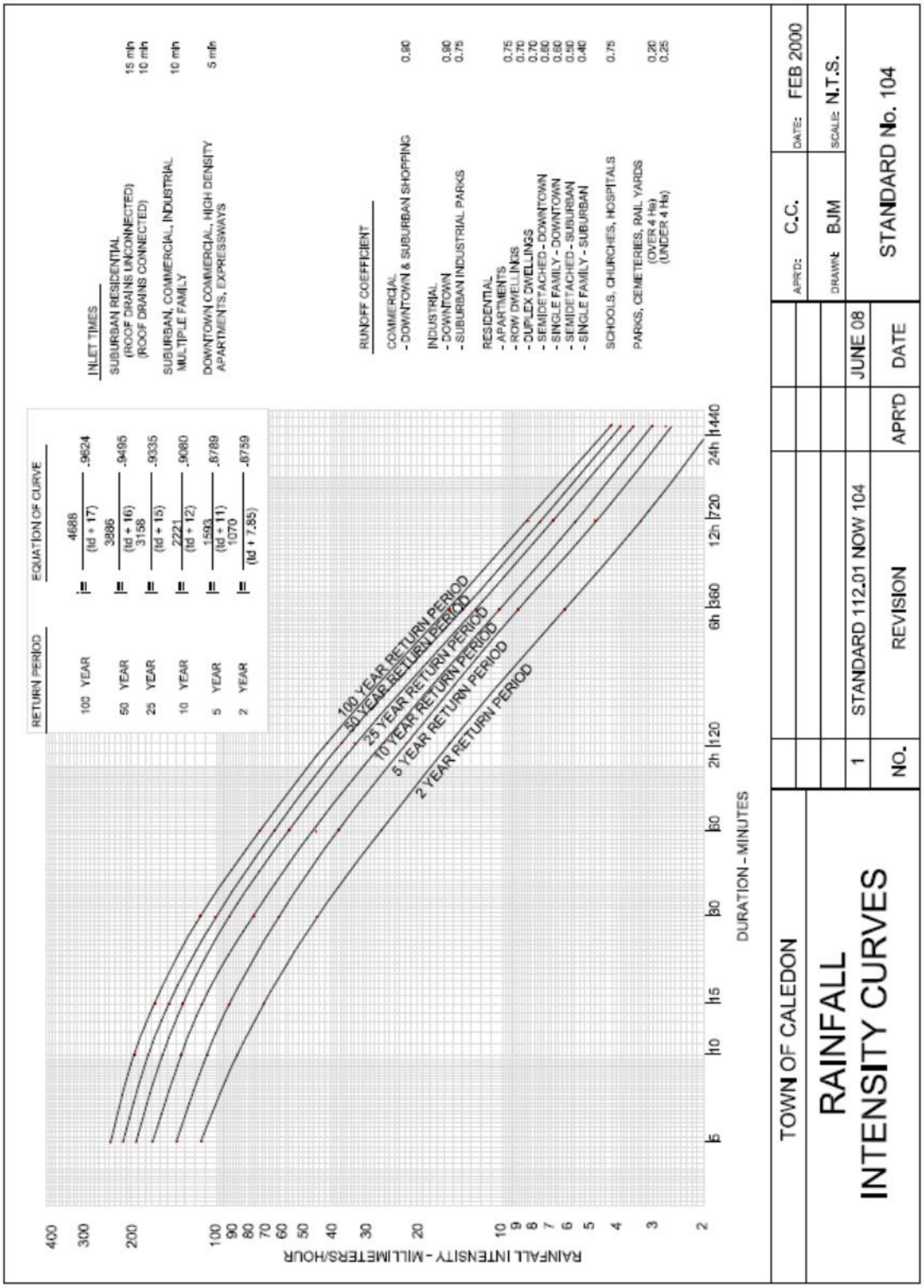
Ram Engineering Inc.

Appendix


TOWN OF CALEDON
RAINFALL
INTENSITY CURVES
STANDARD No. 104
DATE: FEB 2000
SCALE: N.T.S.
APR'D: C.C.
DRAWN: BJM
1
STANDARD 112.01 NOW 104
JUNE 08
NO.
REVISION
APR'D
DATE
DURATION - MINUTES
RAINFALL INTENSITY - MILLIMETERS/HOUR
100 YEAR RETURN PERIOD
50 YEAR RETURN PERIOD
25 YEAR RETURN PERIOD
10 YEAR RETURN PERIOD
5 YEAR RETURN PERIOD
2 YEAR RETURN PERIOD
INLET TIMES
RUNOFF COEFFICIENT

Old Kennedy Road - Storage for Infiltration Bed 1

Rainfall data

100year storm

A= 4688

B= 17

C= 0.9624

T= 10 minutes

Area draining to CB 1

Area (ha) = 0.0188 ha

C= 0.63

$Q_{release}$ 0.0055 m^3/s

Time (min)	Intensity (mm/hr)	Release Rate (m3/s)	Q_{post} (m3/s)	Inflow Volume (m3)	Outflow Volume (m3)	Storage (m3)
10.00	196.54	0.00546	0.007	3.92	3.54	0.38
12.00	183.47	0.0055	0.006	4.39	4.10	0.29
14.00	172.07	0.0055	0.006	4.81	4.66	0.15
15.00	166.89	0.0055	0.006	4.99	4.94	0.06
16.00	162.02	0.0055	0.005	5.17	5.22	00
17.00	157.43	0.0055	0.005	5.34	5.50	00

Storage required to control 100 year post to allowable flow rate = 0.38 m^3

Old Kennedy Road - Area draining to CB 2

Rainfall data

100year storm

A= 4688
B= 17
C= 0.9624
T= 10 minutes

Area Draining to CB 2

Area (ha) =0.0300 ha
C= 0.62

$Q_{release}$ 0.0087 m^3/s

Time (min)	Intensity (mm/hr)	Release Rate (m3/s)	Q_{post} (m3/s)	Inflow Volume (m3)	Outflow Volume (m3)	Storage (m3)
10.00	196.54	0.00871	0.010	6.12	5.60	0.51
11.00	189.78	0.0087	0.010	6.50	6.05	0.45
12.00	183.47	0.0087	0.010	6.85	6.49	0.36
13.00	177.58	0.0087	0.009	7.18	6.93	0.25
14.00	172.07	0.0087	0.009	7.50	7.38	0.12
15.00	166.89	0.0087	0.009	7.79	7.82	00

Storage required to control 100 year post to allowable flow rate = 0.51 m^3

Old Kennedy Road - Area draining to CB 3

Rainfall data

100year storm

A= 4688
B= 17
C= 0.9624
T= 10 minutes

Area draining to CB 3

Area (ha) =0.0313 ha
C= 0.63

$Q_{release}$ 0.0091 m^3/s

Time (min)	Intensity (mm/hr)	Release Rate (m3/s)	Q_{post} (m3/s)	Inflow Volume (m3)	Outflow Volume (m3)	Storage (m3)
10.00	196.54	0.00908	0.011	6.51	5.89	0.62
11.00	189.78	0.0091	0.010	6.92	6.36	0.56
12.00	183.47	0.0091	0.010	7.30	6.82	0.47
13.00	177.58	0.0091	0.010	7.65	7.29	0.36
14.00	172.07	0.0091	0.010	7.98	7.75	0.23
15.00	166.89	0.0091	0.009	8.30	8.21	0.08
16.00	162.02	0.0091	0.009	8.59	8.68	0.00

Storage required to control 100 year post to allowable flow rate = 0.62 m^3

Design for Infiltration Trench 1

Design for Infiltration bed

$V_{infiltration}$	0.38	m^3
ΔT =	36	hrs
Porosity n	0.40	
Hydraulic conductivity	6	min/cm
P (percolation rate)	20.00	mm/hr

Use

$$A = \frac{1000\ V}{Pn\ \Delta t}$$

Minimum bottom Area required	1.31	m^2
Width of bed	1.20	m
Length of bed	2.50	m
Bottom area provided area	3	m^2
Depth of storage media	1.2	m
Volume in bed	1.44	m^2

Design exceeds requirements

Calculations for storage for the 100 year storm

Roof in Area 1

Area 0.01270 ha C= 0.95

100 year storm

A= 4688
T= 10 minutes
B= 17
C= 0.9624
Area (ha) = 0.0127
C= 0.95

$Q_{release}$= 0.0035 m^3/s

Time (min)	Intensity (mm/hr)	Release rate (m^3/s)	Q_{rain} (m^3/s)	Inflow Volume (m^3)	Outflow Volume (m^3)	Storage (m^3)
10.00	196.54	0.0035	0.0066	3.96	2.59	1.36
11.00	189.78	0.0035	0.0064	4.20	2.78	1.42
12.00	183.47	0.0035	0.0062	4.43	2.97	1.46
13.00	177.58	0.0035	0.0060	4.65	3.16	1.48
14.00	172.07	0.0035	0.0058	4.85	3.35	1.49
15.00	166.89	0.0035	0.0056	5.04	3.54	1.49
16.00	162.02	0.0035	0.0054	5.22	3.73	1.48
17.00	157.43	0.0035	0.0053	5.39	3.92	1.46
18.00	153.10	0.0035	0.0051	5.55	4.11	1.43
19.00	149.01	0.0035	0.0050	5.70	4.30	1.39
20.00	145.13	0.0035	0.0049	5.84	4.50	1.35
21.00	141.45	0.0035	0.0047	5.98	4.69	1.29
22.00	137.96	0.0035	0.0046	6.11	4.88	1.23

Storage to control 100 yr post -development = 1.49 m^3

Calculations for storage for the 100 year storm

Flow to infiltration bed

Area 0.02240 ha   C= 0.25

100 year storm

A= 4688
T= 10 minutes
B= 17
C= 0.9624   Flow from Roof 0.0035
Area (ha) = 0.0224 $Q_{100\ flow} = Q_{landscape} + Q_{roof}$
C= 0.25

$Q_{release}$= 0.0040 m^3/s

Time (min)	Intensity (mm/hr)	Release rate (m^3/s)	Q_{rain} (m^3/s)	Inflow Volume (m^3)	Outflow Volume (m^3)	Storage (m^3)
10.00	196.54	0.0040	0.0066	3.94	2.86	1.07
15.00	166.89	0.0040	0.0061	5.49	4.01	1.48
20.00	145.13	0.0040	0.0058	6.91	5.16	1.75
25.00	128.46	0.0040	0.0055	8.25	6.32	1.93
30.00	115.28	0.0040	0.0053	9.53	7.48	2.05
30.00	115.28	0.0040	0.0053	9.53	7.48	2.05
35.00	104.59	0.0040	0.0051	10.77	8.65	2.12
40.00	95.75	0.0040	0.0050	11.98	9.82	2.16
41.00	94.16	0.0040	0.0050	12.22	10.06	2.16
42.00	92.62	0.0040	0.0049	12.45	10.29	2.16
43.00	91.14	0.0040	0.0049	12.69	10.53	2.16
44.00	89.70	0.0040	0.0049	12.93	10.76	2.16
45.00	88.31	0.0040	0.0049	13.16	11.00	2.16

Storage to control 100 yr post -development = 2.16 m^3

Calculations for storage for the 100 year storm

Flow to infiltration bed for Area 3

Area 0.02030 ha C= 0.25

100 year storm

A= 4688

T= 10 minutes

B= 17

C= 0.9624 Flow from Roof 0.0035

Area (ha) = 0.0203 $Q_{100\ flow} = Q_{landscape} + Q_{roof}$

C= 0.25

$Q_{release}$= 0.0040 m^3/s

Time (min)	Intensity (mm/hr)	Release rate (m^3/s)	Q_{rain} (m^3/s)	Inflow Volume (m^3)	Outflow Volume (m^3)	Storage (m^3)
10.00	196.54	0.0040	0.0063	3.76	2.83	0.93
15.00	166.89	0.0040	0.0059	5.27	3.97	1.29
20.00	145.13	0.0040	0.0055	6.66	5.13	1.53
25.00	128.46	0.0040	0.0053	7.97	6.29	1.68
30.00	115.28	0.0040	0.0051	9.23	7.45	1.78
30.00	115.28	0.0040	0.0051	9.23	7.45	1.78
35.00	104.59	0.0040	0.0050	10.45	8.62	1.83
40.00	95.75	0.0040	0.0049	11.64	9.79	1.85
41.00	94.16	0.0040	0.0048	11.88	10.03	1.85
42.00	92.62	0.0040	0.0048	12.11	10.26	1.85
43.00	91.14	0.0040	0.0048	12.35	10.50	1.85
44.00	89.70	0.0040	0.0048	12.58	10.73	1.85
45.00	88.31	0.0040	0.0047	12.81	10.97	1.84

Storage to control 100 yr post -development = 1.85 m^3