WATER SUPPLY, WASTEWATER TREATMENT/DISPOSAL AND STORMWATER MANAGEMENT REPORT

RIVERVIEW HEIGHT SUBDIVISION – PHASE 2

TOWNSHIP OF MAPLETON
WELLINGTON COUNTY

PREPARED FOR:
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PREPARED BY:

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CONTENTS

1.0 INTRODUCTION

ABEC Engineering (ABEC) was retained by Drayton B. G. Inc. to prepare a Stormwater Management Report to support the approval for draft plan of "Riverview Heights" subdivision, phase 2. The purpose of this report is to provide detailed design for the completion of civil work.

1.1 General Description

The subject property is approximately 14.64 ha (36.17 acre) and is located on the North side of Wellington Street North/Highway 11 and Scenic Dr., which is located 11.2 km North of Drayton, Ontario. The subject property is labelled as Phase 2 of and is currently vacant.

The property is bounded by:

- Phase 1 of Riverview Heights subdivision to the South;
- Wellington Street/Highway 11 to the South East, and;
- Agricultural lands to the North, West and East.

The project will consist of the development of 28 residential single family detached dwellings, stormwater management pond, road, ditches, culverts, underground hydro and gas pipeline.

2.0 WATER SUPPLY

2.1 Existing Water Servicing for Riverview Heights Subdivision, Phase 1

The existing lots in Phase 1 of Riverview Heights subdivision are serviced with on lot wells, pump, and water storage reservoir and piping system. The wells are 45.0 to 78.0 m deep and are protected from contamination by 45.0 m of clayey and sandy silt.

2.2 Proposed Water Servicing for Riverview Heights Subdivision, Phase 2

The hydrological investigation report prepared by AGRA Earth and Environmental for phase 2 of Riverview Heights subdivision indicate that the deeper aquifer is protected by 45.0 m of overburden and the tested well (TW1) in Phase 2 are capable of producing 272 L/m at peak rate which is much more than the minimum 7.74 L/m water demand requirements for each dwelling. Testing of TW1 confirmed that water supplies of suitable quantity and quality are available for the proposed 28 lots.

3.0 WASTEWATER TREATMENT AND DISPOSAL

3.1 Existing Wastewater Treatment and Disposal System

The existing lots in Phase 1 of Riverview Heights subdivision are serviced with on lot sewage treatment and disposal system.

3.2 Proposed Wastewater Treatment and Disposal System

The geotechnical investigation report prepared by AGRA Earth and Environmental for phase 2 of Riverview Heights subdivision indicate that the area is covered by 300 mm to 400 mm topsoil. The underneath soil is Sandy silt and Clayey silt till with percolation time of 40 to 50 mm per second.

The maximum dwelling size are about 300 m^2 with 4-bedrooms. The design flow will be about 3,500 litre at maximum. The detail treatment design will be provided on a lot to lot basis according to ministry guideline before building permits are issued for each home.

4.0 STORM DRAINAGE

4.1 Existing Storm Servicing

The existing Phase 1 of Riverview Heights subdivision currently drains southerly to the road side ditch at Highway 11 for drain into the Conestogo River through a culvert at intersection of Highway 11 and 12 Line.

4.2 Storm Design Criteria

The objectives of the Servicing Study is to identify how storm water will be managed. The report identifies constrains and opportunities within the study area as they relates to the proposed development:

- Quantify stormwater management pond volume to meet current criteria for water quality and quantity.
- Review pond locations and determine area requirements and suitability for gravity storm sewer conveyance under developed conditions.
- Review pond locations and size for proposed development.
- Layout road cross-sections, with culvert locations and outfalls
- Review major storm system and design the outfall arrangements with regards to location capacity

The SWM design criteria for the Site to meet the above objectives are set out in following:

- The Township of Mapleton, Stormwater Management Guideline.

- The Ontario MOECC SWM Pond sizing guidelines for impervious area percentages to achieve TSS removal objectives (MOE, 2003).

4.3 Pre-Development Condition

The subject property is divided into five pre-developed catchment areas as follow:

- Drainage Area 101:

The drainage area 101 with 6.62 ha area is a part of watershed A. The pre-developed run-off generated in this catchment are currently drains as sheet flow northerly to watershed A that subsequently drains the run-off to Conestogo River and Lake.

- Drainage Area 102:

The area drainage 102 with 4.74 ha area is a part of watershed A. The pre-developed run-off generated in this catchment are currently drains as sheet flow northerly to watershed A that subsequently drains the run-off to Conestogo River and Lake.

- Drainage Area 103:

The drainage area 103 with 2.05 ha is a part of watershed B. The pre-developed run-off generated in this catchment are currently drains as sheet flow northerly to roadside ditch of Highway 11 and through a culvert under Highway 11 drains to Conestogo River and Lake.

- Drainage Area 104:

The drainage area 104 with 1.54 ha is a part of watershed B. The pre-developed run-off generated in this catchment are currently drains as sheet flow northerly to roadside ditch of Highway 11 and through a culvert under Highway 11 drains to Conestogo River and Lake.

- Drainage Area 105:

The drainage area 105 with 0.58 ha is a part of watershed C. The pre-developed run-off generated in this catchment area currently drains as sheet flow to stream C that drains southerly to roadside ditch at Highway 11 11 and through a culvert under Highway 11 drains to Conestogo River and Lake.

4.4 Post-Development Condition

The post-development discharge from the site was calculated based on the Rational Method. The entire site was divided to four catchment areas under the ID. No. 201, 202, 203, and 204 as follows:

- Drainage Area 201:

The drainage area 201 with 8.37 ha area includes lots and roadways. The post-development run-off generated in area 201 needs to be treated prior to discharge to watershed A. A dry pond is designed to catch the run-off and treat it before discharging to watershed A at 60% basic S.S. removal and below the peak 5-year pre-development flow. Watershed A subsequently discharge to Conestogo River and Lake.

- Drainage Area 202:

The drainage area 202 with 2.90 ha area includes lots and roadways. The post-development run-off generated in area 202 drains to the roadside ditches of Scenic Dr. which currently drains uncontrolled and untreated to roadside ditch of Highway 11 and through a culver under the Highway 11, it drains directly to Conestogo River.

- Drainage Area 203:

The drainage area 203 with 1.63 ha area includes lots and roadways. The post-development run-off generated in area 203 drains to the roadside ditches of Scenic Dr. which currently drains uncontrolled and untreated to roadside ditch of Highway 11 and through a culver under the Highway 11, it drains directly to Conestogo River.

- Drainage Area 204:

The drainage area 204 with 1.68 ha area includes the parkland. The post and pre-development run-off are same as the area will not be graded. The run-off is discharged uncontrolled and untreated to watershed A which drains to Conestogo River and Lake.

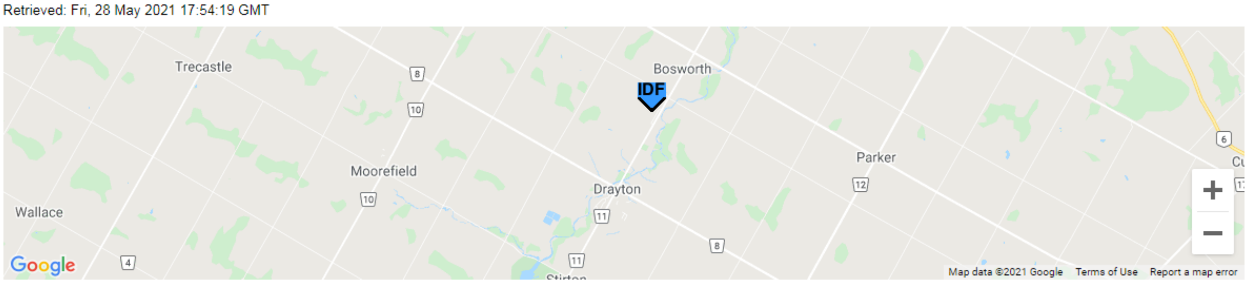
It should be noted that the site is located on high point compared to the adjacent properties and there is no any run-off from adjacent properties to this site.

The composite "C" value for each catchment area was calculated. A time of concentration of 10.0 minutes as per MOECC requirements was used for post-development condition. The rainfall intensity equation and IDF curves were obtained from MTO website for the site. The coordination of site is shown in below picture. The IDF curve for the above coordination is shown in below picture:

Active coordinate

43° 46' 15" N, 80° 39' 45" W (43.770833,-80.662500) Modify selection

Retrieved: Fri, 28 May 2021 17:54:19 GMT



Map options: Modify selection | Show/hide gauging stations | Re-center selection

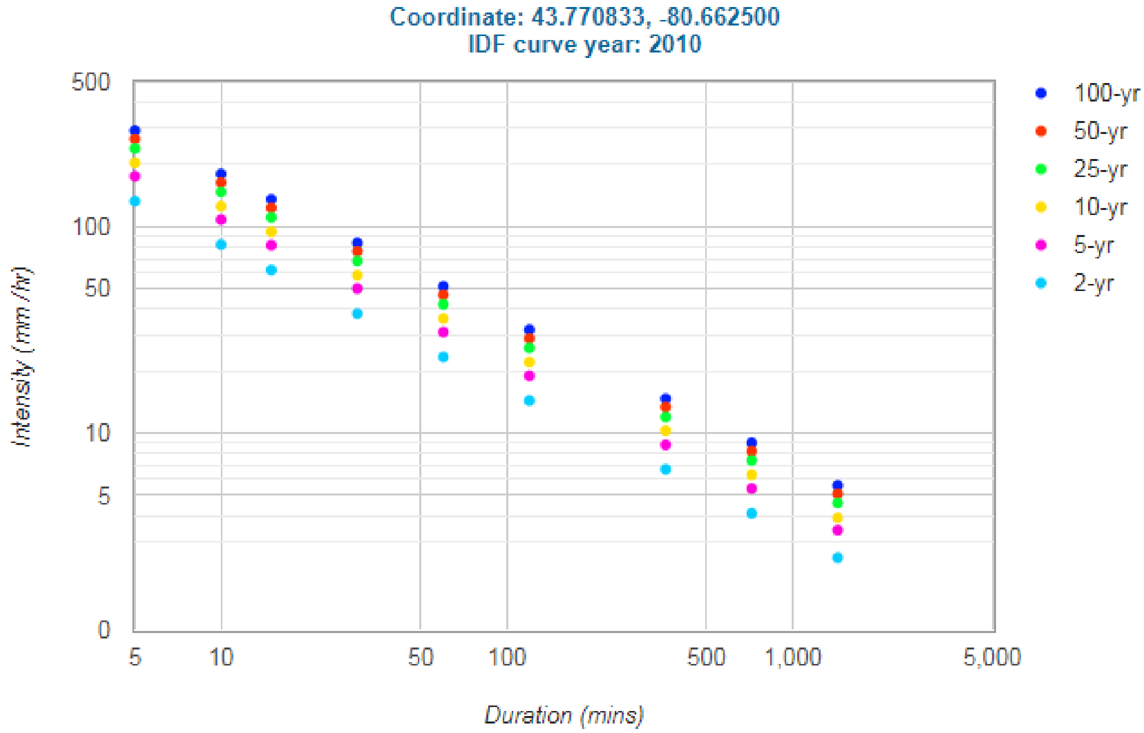
Location summary

These are the locations in the selection.

IDF Curve: 43° 46' 15" N, 80° 39' 45" W (43.770833,-80.662500)

Results

An IDF curve was found.



Switch variable: Intensity or Depth

The rainfall intensity and depth are shown in below table:

Rainfall intensity (mm hr^{-1})

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	132.9	81.9	61.7	38.0	23.4	14.4	6.7	4.1	2.5
5-yr	175.5	108.1	81.4	50.2	30.9	19.0	8.8	5.4	3.4
10-yr	203.9	125.6	94.6	58.3	35.9	22.1	10.3	6.3	3.9
25-yr	239.1	147.3	110.9	68.3	42.1	25.9	12.0	7.4	4.6
50-yr	265.8	163.7	123.3	76.0	46.8	28.8	13.4	8.2	5.1
100-yr	292.0	179.8	135.5	83.4	51.4	31.7	14.7	9.0	5.6

Rainfall depth (mm)

Duration	5-min	10-min	15-min	30-min	1-hr	2-hr	6-hr	12-hr	24-hr
2-yr	11.1	13.6	15.4	19.0	23.4	28.8	40.1	49.4	60.9
5-yr	14.6	18.0	20.4	25.1	30.9	38.1	53.0	65.3	80.4
10-yr	17.0	20.9	23.7	29.1	35.9	44.2	61.6	75.8	93.4
25-yr	19.9	24.6	27.7	34.2	42.1	51.9	72.2	88.9	109.6
50-yr	22.2	27.3	30.8	38.0	46.8	57.7	80.3	98.9	121.8
100-yr	24.3	30.0	33.9	41.7	51.4	63.3	88.1	108.6	133.8

The Rational Method was used to determine the run-off based on the peak flow to discharge area.

Design Flow and Hydrology

All design flows for storm and combined sewer systems will meet the level of protection return period requirements outlined in this chapter.

Run-off Calculation

Rational Method

The Rational Method is a runoff estimation method based on relating the peak flow to the discharge area, rainfall intensity, and a runoff coefficient.

The Rational Method is expressed as:

$$Q = 2.78\ C\ I\ A$$

where:

Q = flow in litres per second
A = drainage area in hectares
C = run-off coefficient, dimensionless
I = intensity of rainfall in mm/hr

The model parameters based on intensity of rainfall and depths for 2-year to 100-year storms was obtained from the MTO website for the site location and is shown as below:

Coefficient summary

Data year: 2010
IDF curve year: 2010

Click a return period in the table header for more detail.

Return period	2-yr	5-yr	10-yr	25-yr	50-yr	100-yr
A	23.4	30.9	35.9	42.1	46.8	51.4
B	-0.699	-0.699	-0.699	-0.699	-0.699	-0.699

Statistics

The composite coefficient of "C" was determined using the coefficient of "C" equal to 0.05 for grass areas, 0.90 for the roof, driveway and road as per MOECC requirements.

Type	"C"
Grass Area	0.30
Forest	0.12
Driveway, Roof and Road	0.95
Lot Size 1 acre	0.32

The pre and post-development peak flow run-off for 2 to 100-year storm was determined for drainage areas 201 as shown in below table:

Drainage Area 201	Area (ha)	"C"	A x C	Time of Con. (min)	Rainfall Intensity (mm/hr)	Peak flow Discharge (L/s)
PRE DEVELOPMENT CONDITIONS						
"C" is set at 0.5 for any pre-development condition as per MOEC guideline						
C Type Soil (Sandy Silt)	Area (m^2)	Percet.	"C"	Composite "C"		
Wood Flat	18500	0.221	0.12	0.26		
Grass Flat	65200	0.779	0.30			
Total Area	83700					
Erosion						
24-Year						
2-Year	8.37	0.26	2.178	10.0	81.9	495.7
5-Year	8.37	0.26	2.178	10.0	108.1	**654.6**
10-Year	8.37	0.26	2.178	10.0	150.5	911.4
25-year	8.37	0.26	2.178	10.0	176.5	1068.8
50-Year	8.37	0.26	2.178	10.0	196.2	1188.1
100-Year	8.37	0.26	2.178	10.0	215.5	1304.9
POST DEVELOPMENT CONDITIONS						
C Type Soil (Sandy Silt)	Area (m^2)	Percet.	"C"	Composite "C"		
Grass Flat	66600	0.796	0.30	0.43		
Hard Surface	17100	0.204	0.95			
Total Area	83700					
Erosion						
24-Year						
2-Year	8.37	0.43	3.62	10.0	81.9	824.5
5-Year	8.37	0.43	3.62	10.0	108.1	1088.8
10-Year	8.37	0.43	3.62	10.0	150.5	1515.9
25-year	8.37	0.43	3.62	10.0	176.5	1777.7
50-Year	8.37	0.43	3.62	10.0	196.2	1976.2
100-Year	8.37	0.43	3.62	10.0	215.5	**2170.4**

For drainage area 201, the peak flow run-off for 100-year storm and 5-year is 654.6 L/s. A dry pond is designed to provide adequate storage with a discharge below the 5-year pre-development peak flow of 654.6 L/s.

4.5 Basement Drainage

All building foundations drains must be drained using sump pump. All sump pump should be fitted on the floor of basement and there is no need for a raiser.

5.0 Pond Design

The SWM Pond design plans, sections and details are included in the Design Drawings. A summary of the SWM Pond dimensions and capacities for each feature are outlined in the sections below. A dry pond was designed for this site.

5.1 Quality Control

Guidelines for quality control have been taken from the Stomwater Management Practices, Planning and Design Manual (MOEE 2003), from Table 3.2 "Water Quality Storage Requirements Based on the Receiving Waters".

Table 3.2 Water Quality Storage Requirements based on Receiving Waters[1, 2]

Protection Level	SWMP Type	Storage Volume (m^3/ha) for Impervious Level 35%	55%	70%	85%
Enhanced 80% long-term S.S. removal	Infiltration	25	30	35	40
	Wetlands	80	105	120	140
	Hybrid Wet Pond/Wetland	110	150	175	195
	Wet Pond	140	190	225	250
Normal 70% long-term S.S. removal	Infiltration	20	20	25	30
	Wetlands	60	70	80	90
	Hybrid Wet Pond/Wetland	75	90	105	120
	Wet Pond	90	110	130	150
Basic 60% long-term S.S. removal	Infiltration	20	20	20	20
	Wetlands	60	60	60	60
	Hybrid Wet Pond/Wetland	60	70	75	80
	Wet Pond	60	75	85	95
	Dry Pond (Continuous Flow)	90	150	200	240

For a basic 60% long-term S.S. removal, a dry pond and a 35% impervious level for the proposed site, extrapolation indicates that a storage volume of 90 m^3/ha is required.

Therefore, for the 8.37 ha site, a total storage volume of approximately 754 m^3 is required. The following table provides the design values for the dry pond and compares these values to the minimum or preferred criteria as per Table 4.6 of the MOECC Manual:

Design Element	**Design Value**	**Comparison to MOEE Criteria**
Drainage Area	8.37 ha	Meets Preferred Criteria
Treatment Volume	Table 3.2 minimum: 754 m^3 Provided: 3,000 m^3	Exceeds Preferred Criteria
Forebay	Minimum depth: 1.5 m Designed depth: 1.8 m	Meets Preferred Criteria
Length-to-Width Ratio	From 4:1 to 5:1 Designed length to width ratio: 4:1	Meets Preferred Criteria
Active Storage Detention Time	Desired: 48 hours Designed value: 51 hours	Meets Preferred Criteria
Inlet	Minimum 450mm Pipe Designed: 525 mm Pipe Pipe slope more than 1.0% Designed pipe slope 1.5%	Meets Preferred Criteria
Outlet	Minimum 450mm Pipe Designed: 450 mm Pipe Pipe slope more than 1.0% Designed pipe slope 1.5% Minimum orifice size: 100 mm Designed orifice size : 100 mm	Meets Preferred Criteria

5.2 Quantity Control

5.3 Stage-Storage-Discharge Table

The stage-stotrage-discharge table is shown in below table:

Stage-Storage-Discharge	**Elevation (m)**	**Volume (m^3)**	**Δh (m)**	**Orifice Diameter (m)**	**Pressure (Pa)**	**Pipe Diameter (mm)**	**Release Rate (L/s)**
Bottom of dry Pond	423.50	0.00	0.00	0.00	0.00	450	0.00
Top of 25mm event	423.90	495	0.40	0.10	1005	450	51.6
Top of active storage (5-year)	424.00	654	1.00	0.10	10052	450	392.4
Top of active storage (100-year)	425.50	1302	1.50	0.10	15078	450	561.9
Top of 0.3 m emergency weir	425.80	1827	1.80	0.10	18093	450	632.1

The discharge from pond under 100-year storm is 632.1 L/s which is below pre-developemtn peak flow of 654.6 L/s. Thereofre, the objective of stormwater management to design a pond that control, store and treat the 100-yaer post-developemnt stormwater peak run-off to that of below the 5-year pre-development is satisfied.

It should be noted that the designed pond for this site is a dry pond with a continmous discharge flow. Therefore, the pond will be dry between each storm event. The discharge from pond is below the pre-development peak flow.

5.4 Drawdown Time of Pond

The drawdown time in the pond can be estimated using Equation 4.10. The drawdown time for the water quality volume of the pond storage was calculated using the drawdown time equation from the MOECC guideline. The drawdown calculation exceeds the 48 hour draw down time recommended by MOECC guideline for dry pond.

$$t = \frac{2\,A_p}{C\,A_o(2g)^{0.5}}\left(h_1^{0.5} - h_2^{0.5}\right)$$ **Equation 4.10: Drawdown Time**

or if a relationship between A_p and h is known (i.e., $A = C_2 h + C_3$)

$$t = \frac{0.66\,C_2 h^{1.5} + 2\,C_3 h^{0.5}}{2.75\,A_o}$$ **Equation 4.11**

where t = drawdown time in seconds
A_p = surface area of the pond (m^2)
C = discharge coefficient (typically 0.63)
A_o = cross-sectional area of the orifice (m^2)
g = gravitational acceleration constant (9.81 m/s^2)
h_1 = starting water elevation above the orifice (m)
h_2 = ending water elevation above the orifice (m)
h = maximum water elevation above the orifice (m)
C_2 = slope coefficient from the area-depth linear regression
C_3 = intercept from the area-depth linear regression

<u>Drawdown Calculation:</u>

Orifice Diameter: 100 mm

Area of the Pond: 1,500 m^2

Depth of Pond: 1.8 m

$$t = \frac{2 \; x \; 1500 \; x 1.341}{0.63 \; x \; 0.00785 \; x \; 4.43} = 51.0 \text{ hours}$$

5.5 Forebay Calculation

The forebay length should be greater than or equal to the larger of the lengths given by Equation 4.5 and Equation 4.6 for settling and dispersion length as per MOECC guideline. The dispersion length for the forebay is calculated as below which is equal to 5.0m and the settling length is calculated as 20.0m. The forebay length at the bottom is 25.0m which exceeds both settling and dispersion lengths.

$$\text{Dist} = \sqrt{\frac{r Q_p}{V_s}}$$

Equation 4.5: Forebay Settling Length

where Dist = forebay length (m)
r = length-to-width ratio of forebay
Q_p = peak flow rate from the pond during design quality storm
V_s = settling velocity (dependent on desired particle size to settle). It is recommended that a value of 0.0003 m/s be used in most cases.

<u>Settling Length Calculation:</u>

Forebay length-to-width ratio: 4

Peak flow arte from the pond: 0.029 m^3/s

Settling velocity: 0.0003 m/s

Dist : 20.0 m

$$\text{Dist} = \frac{8Q}{dV_f}$$ **Equation 4.6: Dispersion Length**

where Dist = length of dispersion (m)
Q = inlet flowrate (m^3/s)
d = depth of the permanent pool in the forebay (m)
V_f = desired velocity in the forebay (m/s)

The depth of the permanent pool in the forebay in Equation 4.6 reflects the deep section (> 1 m) of the forebay required to minimize re-suspension and scour. A guideline for the minimum bottom width of this deep zone is given by Equation 4.7:

$$\text{Width} = \frac{\text{Dist}}{8}$$ **Equation 4.7: Minimum Forebay Deep Zone Bottom Width**

<u>Dispersion Length Calculation:</u>

V_f : Desired velocity in the forebay : 0.5 m/s

Inlet flow rate: 0.31 m^3/s

Depth of Permanent Pool in forebay: 1.0 m

Dist : 5.0 m

5.6 Quality Control During Construction

The Urban Drainage Design Guidelines recommends that 125 m^3 per hectare of storage be provided for quality control during construction. The amount of storage required during construction assuming that the complete drainage area 201 is developed is (125 m^3 x 8.37 = 1,047 m^3). The amount of storage provided is 3,000 m^3 which exceeds the required storage.

5.7 Temporary Erosion and Sediment Control During Construction

The erosion potential of study area was assessed using methods described in the guidelines of Erosion and Sediment Control or Urban Construction (May 1987). This assessment was based on the following parameters:

Parameter	Measure	Class
Average Slope	1%	Gentle
Runoff Length	700 metres	Long
Soil Type	silty clay	Medium
Erodibility		

Taken together, the above parameters indicate a "moderate erosion potential" based ont eh above guideline.

During construction of the subdivision various temporary measures will be prevented to prenet the discharge of the sediment lean stormwater from the site. This measure include silt fencing, checkdams, catchbasin buffers and mudmats as shown on Sediment Control Plans and Details. Siltation control for this site is provided by siltation fencing as shown on the plan.

6.0 CONCLUSION

This report illustrates that the proposed development is feasible from municipal servicing and stormwater management perspectives.

The proposed domestic water demands is supplied by drilling deep well with suitable quantity and quality of water for use of the proposed 28 lots. The water demand for fire can be obtained from the pond.

The lot sizes, water table level and type of the soil at the bottom of base cut indicates that a regular type 4 filter sand bed with a septic tank and a holding tank will be sufficient to service the wastewater disposal for each dwelling. The leaching bed does not need to be raised as the water table is deep in the site.

An extended detention dry pond is designed that control the peak post-development flow to below peak pre-development flow. Drawdown time of pond is 51 hour that exceeds the required 48 hour as per MOECC guideline.

The quality control during construction is achieved by the use of silt fencing, mudmats, checkdams in conjunction with the proposed stormwater management pond. The proposed pond active storage capacity exceeds the required storage capacity of 1,047 m^3 (125 m^3 per hectare).

We trust that this report addresses your current needs. Should you have any questions, or require additional information, please do not hesitate to contact the undersigned.

Respectfully Submitted,

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ABEC Engineering