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STORMWATER MANAGEMENT

AT

0 SCUGOG LINE 6,
PORT PERRY, ON

PREPARED FOR:

Wanlong Real Estate Development Ltd.
(Property Owner)

January 24th, 2024

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1. Site Background

King EPCM (the Engineer) was retained by Wanlong Real Estate Development Ltd. (the Client and Property Owner) for geotechnical and environmental engineering services, including the creation of a Stormwater Management Plan (SWM). The property is located at 0 Scugog Line 6, Lot 14, Port Perry, Ontario (the Site). It is understood that the SWM is for the sole purpose of the application and construction of a 12-lot industrial subdivision and new drive aisle. This report is to be submitted to the Township of Scugog (Port Perry), Kawartha Region Conservation Authority (KRCA), and Regional Municipality of Durham (Durham Region). The subject site location is shown in Figure 1.

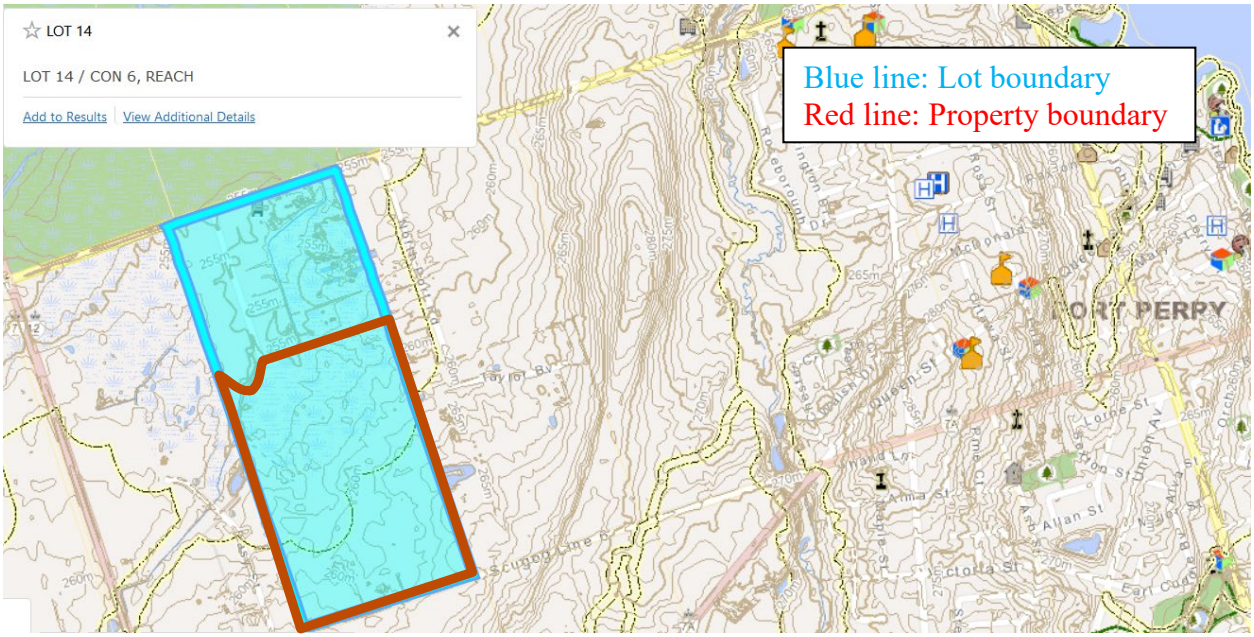


Figure 1 – Topographic map of 0 Scugog Line 6, Port Perry, ON

(Regional Municipality of Durham, 2023)

The proposed project is composed of a 12-block industrial subdivision, including new driveways, post-development TIMP (Total Impervious Area) = 22.1ha = 42.2%. New buildings and warehouses are proposed for each block with a total area of 25%, in addition to 20% for interior roads, 25% for parking lots, and 30% for landscaping. The site property is located on the north side of Scugog Line 6 Road between Easy St. and North Port Rd, east of Highway 7, in the southern half of Lot 14. The site is on agricultural land, with industrial/commercial structures to the north, west, and east, and with an agricultural area south of the Scugog Line 6.

The Durham Regional Official Plan (ROP) designated the subject property as an 'Employment Area' in the Urban System and is approximately 52.4 hectares (129 acres) in size. This site is located within the Schomberg Clay Plains physiographic region, and it is low-lying with a shallow slope from southeast to

northwest. The material of the southeastern part of the site is mainly compacted clay with low permeability, but to the northwest, there are silt/loam materials with high permeability. There are two small ditches in the northwestern and southwestern part of the site property, which drain into a tributary connected to the Nonquon River, as well two small ditches located on the northern half of the eastern boundary, which join the tributary in the north. The subject lands are currently vacant, contain key natural heritage features, and are also partially regulated by the Kawartha Region Conservation Authority (KRCA).



Figure 2 –Site Location Plan

The proposed developments are displayed in Appendix IX:
#1: Block 1 (TIMP= 16,131 m^2)
#2: Block 2 (TIMP= 15,702m^2)
#3: Block 3 (TIMP= 18,323m^2)

#4: Block 4 (TIMP= 15,343m^2)
#5: Block 5 (TIMP= 14,025m^2)
#6: Block 6 (TIMP= 14,322m^2)
#7: Block 7 (TIMP= 19,590m^2)
#8: Block 8 (TIMP= 9742m^2)
#9: Block 9 (TIMP= 10,344m^2)
#10: Block 10 (TIMP= 20,399m^2)
#11: Block 11 (TIMP= 10,266m^2)
#12: Block 12 (TIMP= 35,868m^2)
#13: Proposed Asphalt Municipal Roadway (TIMP= 20,683m^2)

The existing pervious areas which are located outside of the proposed development, containing key natural heritage features are:
#14: Several separate wetlands (Total area = 20,393m^2)
#15: Woodlands & Hedgerow (Total area =170,921m^2)

Since Blocks 14 and 15 are outside of the proposed development and are natural heritage features, no SWM facilities will be proposed for those blocks and the effective study area for the subject property would be Blocks 1-13.

Table 1 – Proposed land uses with occupied area (m^2) in each block

Block	Total Area (m^2)	Impervious (70%)			Pervious (30%)		
		Building (25%)	Parking lots (25%)	Interior roads (20%)	Bioretention/Bioswale (Area)	Septic Bed	Landscape/grass (Total pervious Area)
1	23045	5761	5761	4609	1257	5000	6913
2	22432	5608	5608	4486	1224	5000	6729
3	26176	6544	6544	5235	1428	5000	7853
4	21919	5480	5480	4384	1196	5000	6576
5	20036	5009	5009	4007	1093	5000	6011
6	20461	5115	5115	4092	1116	5000	6138
7	27985	6996	6996	5597	1526	5000	8396
8	13917	3479	3479	2783	759	3600	4175
9	14777	3694	3694	2955	806	3600	4433
10	29141	7285	7285	5828	1590	5000	8742
11	14666	3666	3666	2933	800	3600	4400
12	51239	12810	12810	10248	2795	5000	15372
13	47471	20683			17959	-	26788

The Nonquon River is the largest watershed flowing into Lake Scugog. This river is the largest tributary of Lake Scugog and is approximately 35% of the total Lake Scugog watershed contributing area. The Nonquon River headwaters are located in the Oak Ridges Moraine at an elevation of 304.8 meters above sea level (masl) in the Township of Scugog, flowing north to Lake Scugog. The majority of this watershed (80%) is located within the Peterborough Drumlin Field. Approximately 5% of this watershed lies within the Oak Ridges Moraine and 15% of the land area lies within the Schomberg Clay Plains, including properties north of Scugog Line 6 Rd., such as the subject site.

2. Site Investigation- Monitoring Wells & In-Situ Permeameter Testing

2.1. Monitoring Wells

Thirteen (13) boreholes were drilled at the site by King EPCM (O.Reg 903 License C-7691) and then developed into monitoring wells. Detailed borehole drill logs are in Appendix III, summarized in Table 2. Groundwater has been observed in all boreholes since drilling (Table 3). See Appendix IV for the relation between water table contours (groundwater piezometric surface) with topographic map contours. Groundwater flows at right angles to the equipotential lines in the same way that water flowing down a slope would flow at right angles to the contour lines. Based on this map, the higher groundwater levels are in the northwest part of the site where several waterways/creeks are flowing.

Table 2 - Borehole & Monitoring Well Summary

Borehole #	Date	Northing (UTM)	Easting (UTM)	Surface Elev (masl)	Hole Depth (m)	Screen Elevations (m)	Surface Soil type	Groundwater
101	06-Dec-2021	4,884,244.90	661,934.63	262.184	3.1	259.08-262.07	Clay	yes
102	07-Dec-2021	4,884,266.53	661,997	262.627	4	258.63-260.12	Clay	yes
103	07-Dec-2021	4,884,287.68	662,059.62	263.463	3.1	260.36-261.85	Clay	yes
104	08-Dec-2021	4,884,364.30	661,868	260.898	4	256.9-258.39	Clay	yes
105	09-Dec-2021	4,884,404.51	661,950	261.746	2.7	259.05-260.54	Clay	yes
106	15-Dec-2021	4,884,440.76	662,027	262.448	2.5	259.87-261.42	Clay	yes
107	08-Dec-2021	4,884,471.38	661,772	260.092	3.1	257-258.48	Clay	yes
108	09-dec-021	4,884,508.26	661,897.12	261.729	3	258.73-260.22	Clay	yes
109	09-Dec-2021	4,884,544.89	661,977	261.704	3	258.7-260.2	Clay	yes
110	14-Dec-2021	4,884,788.53	661,612.83	256.346	3.1	253.35-254.84	Sand	yes
111	14-Dec-2021	4,884,841.03	661,720.39	257.541	3.1	254.54-256.04	Sand	yes
112	14-Dec-2021	4,884,893.89	661,843	258.759	3	255.76-257.25	Clay	yes
113	13-Dec-2021	4,884,926.44	661,327.92	255.121	2.1	253-254.52	Silt	yes

Table 3 –Water-level measurements(m) in monitoring wells, December 2021, March, May & June 2022

Date		BH 101	BH 102	BH 103	B 104	BH 105	BH 106	BH 107	BH 108	BH 109	BH 110	BH 111	BH 112	BH 113
December 2021	GW depth (m)	0.4	3.5	2.2	1.8	2.2	2.2	2.2	2.4	2.8	1.0	1.0	1.5	0.7
	GW Level (m)	261.784	259.127	261.263	259.098	259.546	260.248	257.892	259.329	258.904	255.346	256.541	257.259	254.421
March 30, 2022	GW depth (m)	1.3	0.9	1.2	0.4	0.8	1.0	1.1	1.0	1.0	1.2	1.3	1.7	0.5
	GW Level (m)	260.884	261.727	262.263	260.498	260.946	261.448	258.992	260.729	260.704	255.146	256.241	257.059	254.621
May 30, 2022	GW depth (m)	-	-	-	-	-	-	-	-	-	1.25	1.58	-	1.35
	GW Level (m)	-	-	-	-	-	-	-	-	-	255.096	255.961	-	253.771
June 22, 2022	GW depth (m)	1.27	0.68	1.15	0.06	0.80	1.12	1.29	1.37	1.06	*	1.53	1.73	0.54
	GW Level (m)	260.914	261.947	262.313	260.838	260.946	261.328	258.802	260.359	260.644	*	255.920	257.029	254.580

** The borehole was broken.*

2.2. In-Situ Permeameter Testing

Based on two separate field visits dated December 13 and 15, 2021, "field-saturated" hydraulic conductivity, K_{fs}, was achieved using the Constant Head Well Permeameter (CHWP) method. K_{fs101} was conducted at BH101 while K_{fs106} at BH106, K_{fs107} at BH107, and K_{fs112} at BH112 using ETC Slow Soils Pask Permeameter Apparatus. The Constant Head Well Permeameter (CHWP) method was described in Appendix V in detail.

The in-situ infiltration test was not tested at the actual bottom of the LID but based on sieve size analysis and borehole soil samples. It is in the Engineer's opinion, as a geotechnical engineer, that the soils perform similarly in hydrological infiltration potential.

The ETC Pask Permeameter is a convenient and easy-to-use apparatus for ponding a constant head of water in a well, and simultaneously measuring the flow into the soil. The Kfs was calculated using Quick Field Reference Tables provided by (ETC, 2018) as:

K_{fs101}= 7.5E-9 m/sec = 7.5E-7 cm/sec
K_{fs106}= 6.2E-9 m/sec = 6.2E-7 cm/sec
K_{fs107}= 4.0E-6 m/sec = 4.0E-4 cm/sec
K_{fs112}= 6.2E-9 m/sec = 6.2E-7 cm/sec

It should be noted that three tests (K_{fs101}, K_{fs106}, K_{fs112}) have a low permeability in clayey soils, while the fourth test (K_{fs107}) has a higher permeability for a thick upper sandy layer within clay soils.

K_{fs} were then corrected for temperature (for t=4$^{\circ c}$-7$^{\circ c}$):

K_{a101}=7.07E-9 m/sec = 7.07E-7 cm/sec
K_{a106}= 6.04E-9 m/sec = 6.04E-7 cm/sec
K_{a107}= 3.77E-6 m/sec = 3.77E-4 cm/sec
K_{a112}= 5.84E-9 m/sec = 5.84E-7 cm/sec

Correlations between Perc Time (PT) and field-saturated hydraulic conductivity (K_{fs}) are often used in the development of on-site water recycling and treatment facilities that operate by infiltration into unsaturated soil. Based on OMMAH (1997) interpolation, the measured infiltration rate may be interpolated as:

PT_{101} = 48.8 min / cm (Infiltration Rate = 12.3 mm/hour)

PT_{106} = 50.8 min / cm (Infiltration Rate = 11.8 mm/hour)

PT_{107} = 9.1 min / cm (Infiltration Rate = 66 mm/hour)

PT_{112} = 51.3 min / cm (Infiltration Rate = 11.7 mm/hour)

Calculations can be found in Appendix V. Considering that most of the new blocks under development are composed of clay materials and the substrate material of the upper sandy layer is composed of low permeability clay, like other parts of the site, the slower infiltration rates from PT_{101}, PT_{106}, and PT_{107} may be used as representative of the permeability of the site. Therefore, the Engineer's opinion is to trust the values obtained from the OMMAH (1997), with an unfactored infiltration rate of 11.9 mm/hour.

For a conservative approach to infiltration speeds, the Wisconsin Department of Natural Resources (2004) method shall be used for the calculation of a factored design infiltration rate and the Engineer's opinion is that the factored engineering design infiltration rate is 4.8 mm/hour, with a safety factor of 2.5. See Appendix V for more details, calculations, and graphs provided.

3. Pre-development site conditions

The site conditions prior to development can be broken down into several groups (See Appendix IX for more details):

3.1. Topographic Elevation & Base Precipitation

- This site is located within the Schomberg Clay Plains physiographic region, and it is low lying with a shallow slope from southeast to northwest at a 1.25% gradient.
- Site survey & topographic information are in Appendix I, the Site Plan & Grading Plan in Appendix II, and the Pre-development catchment area in Appendix IX.
- Average annual precipitation reported 909.5mm at Burketon McLaughlin (derived from Environment Canada Climate Normals). The nearest station is located in the southeast of the site in which yearly rainfall and snowfall were 774.1mm and 135mm, respectively. September is generally the wettest month of the year whereas February is the driest (Appendix VI). Based on the water balance model output cited in: NONQUON RIVER SUBWATERSHED STUDY AND NONQUON INDUSTRIAL TRIBUTARY AREA, MASTER DRAINAGE PLAN, FINAL REPORT, Oct 2005; reported total precipitation is **898 mm/year** and actual evapotranspiration is **567 mm/year**. These values were used in this study. Base yearly precipitation of the site was also estimated at 881 mm/year by OFAT (Ontario Flow Assessment Tool), for the main Nonquon River Watershed and the site's sub-watershed.
- The site is within the Nonquon River Subwatershed, with Clay and Loam Hydrologic Soil Group B/D.
- The Nonquon River is the largest watershed flowing into Lake Scugog. This river is the largest tributary of Lake Scugog and is approximately 35% of the total Lake Scugog watershed contribution area.

3.2. Vegetation & Evapotranspiration

- The site soil is mostly compacted clay, except in the middle part which is loam (ML). The Hydrologic Soil Group should be B/D.
- Existing structures on the site include a single office trailer and a gravel driveway, with less than 0.1% TIMP which is negligible.
- The site has two rows of scattered trees on the eastern and northern boundaries, and a dense forest area classified as a key natural heritage is located on the western part of the site.
- 38.6ha / 74% of the site is high-intensity agriculture.
- Averaged site evapotranspiration is 567mm/year (Appendix VI)

3.3. Precipitation Surplus (Recharge + Runoff)

- Net surplus (precipitation surplus) = 898-567 = 331 mm/year for cultivated area
- Recharge for pervious area based on MOEE Hydrogeological Tech Info, 1995, Table 2: Infiltration factors
  - Topography (Rolling land) = 0.2
  - Soil (Medium combination of clay & loam) = 0.2
  - Cover = 0.1 (cultivated lands) & 0.2 (Woodlands/Wetlands)
  - MOE infiltration factor = 0.5 (Agricultural area) and 0.6 (Wetland/Woodland)
  - Recharge rate for pervious area is 0.5*331 = 165.5 mm/year (Cultivated)
  - Recharge rate for pervious area is 0.6*331 = 198.6 mm/year (Woodland/Wetland)
- Storm runoff from the cultivated area = 331 – 165.5 = 165.5 mm/year
- Storm runoff from the woodland or wetland area = 331 – 198.6 = 132.4 mm/year
- The following area-weighted calculations are based on:
  - ✓ 333,264 m^2 cultivated,

- ✓ 170,527 m^2 woodland,
- ✓ 20,393 m^2 wetland.

- Total area-weighted average precipitation surplus (recharge + runoff) = 331 mm/year.
- Total area-weighted average storm runoff = 153 mm/year
- Total area-weighted average recharge = 178 mm/year
- See Appendix X- Water Balance Calculations for more details.

3.4. Stormwater Run-off

- There are no city-maintained stormwater sewers, with stormwater generally collected by the interior ditches in the southwest, northwest, and northeast of the property which connects to a tributary of Nonquon River and flows north to Scugog Lake.
- All stormwater in ditches is generally infiltrated or moved downstream. The historical well records and the new monitoring borehole (BH 113) drilled by the Engineer in the northwest corner of the site, near the creek, confirm that the groundwater level in this part of the property is high (Table 3 above).
- Based on in-situ infiltration test using ETC Pask Permeameter Apparatus, the infiltration rate is 48.8 min/cm = 0.204 mm/min = 12.3 mm/hour below the southwest (K_{fs101} near BH101) to northeast diameter (K_{fs112} near BH112) and is 9.1 min/cm = 1.098 mm/min = 66 mm/hour on the other side (K_{fs106} & K_{fs107} near BH106 & BH107). Four permeability tests have been performed at this site, three of them in the integrated clay layer, and one in the thick upper sand layer.
- Native permeable infiltration rate should use a non-factored infiltration rate of 11.9 mm/hour because the predominant soil material of this site is composed of clay (CL) to a depth of more than 3 meters, except an upper layer of sand in the northwestern corner. A 2.5 factor of safety was applied to the native permeable infiltration rate to determine the design infiltration rate of 4.8 mm/hr. This design infiltration rate was used to guide the designs of the LIDs (bioretension & bioswale).
- Based on the Township of Scugog requirements, the Regional Storm is the Timmins Storm event, with data from Design Criteria and Standard Details Drawings of Township of Scugog, with A, B, C values of 3-parameter Chicago distribution design storm as: i = a / (t+b)^c.
- See Table 4: Chicago Distribution Design Storm Parameters and Rainfall Amounts cited in the Design Criteria and Standard Details Drawings of Township of Scugog (Drawing No. SS-600, April 1990), May 12, 2003, showing the IDF values (Appendix VII).

Table 4 - Township of Scugog Chicago Distribution Storm Parameters

Return Period	2 Year	5 Year	10 Year	25 Year	100 Year
A	645	904	1065	1243	1799
B	5	5	5	4	5
C	0.786	0.788	0.788	0.787	0.810

	Return Period (year)				
IDF Parameters	2	5	10	25	100
a	645	904	1065	1243	1799
b	5	5	5	4	5
c	0.786	0.788	0.788	0.787	0.81
Duration	Intensity (mm/hour)				
5 min	105.57	147.29	173.52	220.54	278.63
10 min	76.76	107.01	126.06	155.76	200.63
15 min	61.23	85.30	100.49	122.49	158.93
30 min	39.44	54.88	64.66	77.48	101.00
1 hr	24.24	33.70	39.70	47.10	61.17
2 hr	14.50	20.13	23.71	27.99	36.02
4 hr	8.54	11.84	13.95	16.43	20.88
6 hr	6.25	8.65	10.19	11.99	15.12
12 hr	3.64	5.04	5.93	6.98	8.67
24 hr	2.12	2.93	3.45	4.05	4.96

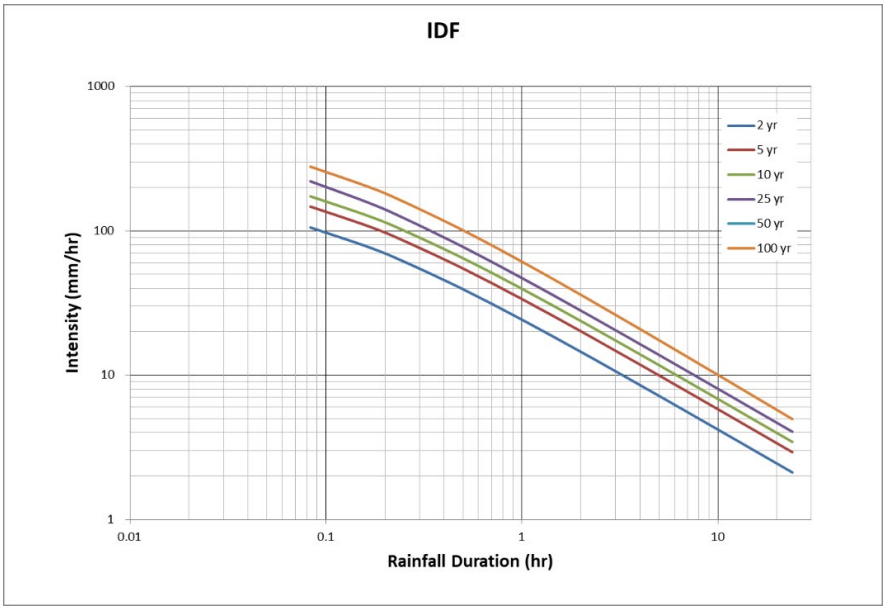


Figure 3- Intensity-Frequency-Duration (IDF) Curves calculated from Table 4 above.

According to the KRCA Technical Guidelines for Stormwater Management (Appendix A, Section 1.4), two methods are generally used to derive synthetic design storms. One method that develops the storm hyetograph from the IDF curve or using uniform rainfall methods, is named the Chicago design storm (Keifer and Chu, 1957) or the modified Rational Method. The second method develops the synthetic design storm from an analysis of historic storm events and produces rainfall distribution instead of a certain amount for peak discharge. Examples of this type of historical analysis include applying the U.S. Natural Resource Conservation Service formerly the Soil Conservation Service (SCS) design storm, and the Canadian Atmospheric Environment Service (AES) design storm.

After communication with KRCA (Iryna Shulyarenko, Hydrologist, Kawartha Conservation on March

8, 2022), manual calculations were recommended, such as the Modified Rational Method for this project. The Engineer used the Modified Rational Method for this site. This method is based on a simple empirical formula used to determine flow that results from a rainfall of specific intensity applied to an area based on an average catchment land use condition.

- Pre-development 1 in 2 years, 1 in 5 years, 1 in 10 years, 1 in 25 years, 1 in 50 years, and 1 in 100 years design storm events, peak flow conditions are calculated as follows:
- Peak flow rate Modified Rational Formula: Qp = (1/360) * A * C * Ca * i

    - A = area in m^2 = 33.3 ha**
    - C = runoff coefficient = 0.55 ** (Clay loam or Clay & 0.1% TIMP, Section 21, Appendix A, KRCA Technical Guidelines for SWM, & MTO 1997 design chart 1.07 for rural)

  ** Note: The Engineer unselected conservatively the effect of Key Natural Heritage and/or Hydrologic Features (KNHHF) in runoff coefficient estimation. This portion is located outside the development area plan and has the same effect on the runoff captured in pre- and post-development scenarios. In other words, the difference in runoff captured by this portion will be zero in both cases.

    - Ca = Antecedent Precipitation Factor = 1.0 for 2, 5 and 10 years, 1.10 for 25 years, 1.20 for 50 years, and 1.25 for 100 years.
    - i = average rainfall intensity in mm/hour (IDF for Scugog, Table 4 above)
    - See the Pre-Development catchment area in Appendix IX.

- Time of Concentration:

    - Bransby Williams Method if $C > 0.4$, $\boldsymbol{T_c = \frac{0.057 * L}{S_w^{0.2} * A^{0.1}}}$

      Where:
      T_c= time of concentration (min)
      L = catchment length, (m)
      S_w = catchment slope (%)
      A = catchment area (ha)
    - When the time of concentration is calculated as less than 10 minutes, an assumed time of concentration for those catchments is 10 minutes.

- Since each block will have its own stormwater management facilities, the pre-development peak flow rates for each block (1 – 13) are calculated in Table 5, noted that blocks 14 and 15 (Woodlot and Wetland) will remain the same in pre- and post-development conditions, and they do not contribute drainage to proposed SWM infrastructure.

Table 5– Flow Rate Calculations using Modified Rational Method

(Pre-Development Scenario)

Parameter	Block	Area (ha)	Time of Concentration (min)	Design Storm				
				2 Year	5 Year	10 Year	25 Year	100 Year
Total Runoff (Qp) in l/s **(Intensity in mm/hr)**	1	2.30	12.0	245.2 (69.6)	341.7 (97.1)	402.6 (114.3)	543.7 (140.4)	720.4 (163.7)
	2	2.24	12.8	230.2 (67.2)	320.7 (93.6)	377.9 (110.3)	509.2 (135.1)	675.0 (157.6)
	3	2.62	12.7	269.2 (67.3)	375.2 (93.8)	442.0 (110.5)	595.7 (135.4)	789.7 (158.0)
	4	2.19	10.6	249.2 (74.4)	347.4 (103.7)	409.3 (122.2)	555.1 (150.7)	734.7 (175.5)
	5	2.00	11.2	221.4 (72.3)	308.6 (100.8)	363.6 (118.8)	492.2 (146.2)	651.7 (170.3)
	6	2.05	10.7	230.9 (73.9)	321.9 (103.0)	379.2 (121.3)	514.1 (149.5)	680.5 (174.2)
	7	2.80	14.6	266.2 (62.3)	370.8 (86.7)	436.9 (102.2)	586.3 (124.7)	777.9 (145.6)
	8	1.39	10.0	163.2 (76.8)	227.5 (107.0)	268.0 (126.1)	364.3 (155.8)	481.9 (181.3)
	9	1.48	10.0	173.3 (76.8)	241.6 (107.0)	284.6 (126.1)	386.8 (155.8)	511.7 (181.3)
	10	2.92	15.6	267.0 (59.9)	371.9 (83.4)	438.1 (98.3)	586.8 (119.6)	778.9 (139.8)
	11	1.47	10.0	172.0 (76.8)	239.8 (107.0)	282.5 (126.1)	383.9 (155.8)	507.8 (181.3)
	12	5.12	17.6	435.9 (55.7)	607.1 (77.6)	715.2 (91.4)	954.5 (110.9)	1267.8 (129.6)
	13	4.75	40.6	232.3 (32.0)	323.1 (44.5)	380.7 (52.5)	499.3 (62.6)	663.1 (73.1)

4. Stormwater Management Plan

The SWM plan has been prepared in accordance with the MOE Stormwater Management Planning and Design Manual, and KRCA Stormwater Technical Guidelines as detailed in Section 13. The SWM plan is subject to review and approval by the Township, Region, and KRCA, and is presented in the following sections.

4.1. Design Criteria

The following design criteria are to be satisfied in the proposed SWM plan:

- The stormwater management plan must maintain existing stormwater runoff rates at the site outlet by restricting post-development peak flow rates to pre-development levels for the 1:2-year through 1:100-year design storms;
- The stormwater management plan must provide quantity control storage for storm events up to and including a 1:100-year design storm in addition to the 25mm storm runoff volume;
- Safe conveyance of the 2-year to 100-year peak flows through the site to the downstream drainage system must be provided for surface runoff generated within the development;
- Storage requirements for an infiltration basin for quality treatment following Table 3.2 in "Stormwater Management Planning and Design Manual" (MOECP, 2003); and
- Protect life and property from flooding and erosion.

4.2. Stormwater Management Strategy

The proposed stormwater management strategy comprises a “treatment train” approach utilizing a combination of lot level controls and Low Impact Development (LID) measures to minimize potential increases in the volume of runoff and provide, as far as practical, a natural hydrologic response. Measures are proposed to be undertaken at the source, conveyance, and end of pipe locations, and are as follows:

- recharge of rooftop stormwater by direction to bioretention basin to promote filtering and natural infiltration;
- by lot grading, as far as practical, the direction of structure envelope drainage, via sheet flow, towards bioretention and naturalized areas versus the road right of way;
- use of a roadside bioswale for road drainage versus a piped storm sewer system;
- use of an oil/grit separator for each outfall;
- use of a bioretention basin for each block to temporarily detain and slowly release stormwater to meet applicable stormwater management criteria; and
- The seasonally high-water table depth should be > 1 m below the bottom of the infiltration subdrain.

The use of bioswales along with check-dams versus a piped storm sewer system is proposed to encourage passive infiltration of stormwater, provide linear storage in the conveyance system to dampen hydrologic response, and provide pre-treatment of stormwater prior to discharge to the wetland/creek.

This study will follow the goals of the "Nonquon River Subwatershed Study and Nonquon Industrial Tributary Area Master Drainage Plan" (Greenland International Consulting Ltd. & Stantec Consulting Ltd., 2005) that will: 1) provide future direction for sustainable development in the Nonquon River Subwatershed or "Study Area"; 2) ensure the long-term health of ecosystems through protection and conservation of significant natural heritage features; and, 3) protect human life and reduce property damage from hazards such as flooding and erosion.

Based on the investigations by Greenland & Stantec, there are some issues of concern that exist in the study area, to sustain or enhance the ecosystem and which will influence the selection of resource management objectives, include the following:

- Baseflow augmentation during drought sequences to sustain wetland and other aquatic resources, as well as to maintain suitable wastewater assimilation conditions for the proposed regional structure;
- Construction of new storm drainage systems for development lands, with recognition of any Drainage Act requirements;
- Water quality (both surface and groundwater) protection and enhancement.

The main vision of the Nonquon River Subwatershed Study was to conserve natural resources within the Nonquon River Subwatershed in a co-operative, integrated manner in which human needs are met in balance with the need to sustain the natural environment (Greenland International Consulting Ltd. & Stantec Consulting Ltd., 2005). The proposed sub-division site plan meets the objectives of the subwatershed plan developed for the Nonquon River Subwatershed to enhance the conveyance function and ecological integrity of Study Area watercourses by exploring opportunities to:

- Control existing runoff volumes, peaks and quality.
- Provide relief to existing and potential flooding and erosion problems.
- Mitigate existing land-based activities that are resulting in the degradation of the streams, by using "at- source", "conveyance" and "end-of-pipe" Storm Water Management Practices (Bioretentions).
- Enhance existing vegetation communities, where practical, through the eradication of any invasive species, installation of infill and understory plantings and the establishment of linkages between isolated fragments and patches.
- Strengthen the riparian vegetation community along the specified reaches of the tributaries with the objective of enhancing the stability of the stream system and improving aquatic habitat.
- Increase the diversity of plant communities through the introduction of a range of species with unique attributes related to the provision of food or shelter for birds and wildlife.

5. Proposed SWM Plan

The Client has proposed to construct several new industrial blocks containing buildings, warehouses, parking lots, and driveways on the property while retaining the existing natural heritage (woodlands, wetlands, and ditches) outside of the developed area. Based on the high groundwater level observed during borehole drilling, monitoring wells in the last few months, and low infiltration clay soil existed in the site to a considerable depth, the Engineer proposes that rooftop downspouts along with parking lots and interior roads runoffs be captured by bioretention in each block, while driveways are surrounded by bioswales.

On the other hand, the impervious pavements (parking lot, walkway, and driveway within each block) be captured by asphalt directly and then directed to the proposed bioretention basin, in each block, and if there is any overflow, it will be directed into the roadside bioswale. It is recommended that separate Oil/Grit Separators (OGS) units be installed in each block to treat the pollutant-generating areas, such as the roadways, driveways, parking spaces, etc.

Below is a summary of the proposed site conditions:

- Total site surface area = 52.4 ha
- Total pervious surface area (Woodlands, Wetlands, and landscapes) = 30.3 ha (57.8%)
- Total impervious surface area (TIMP) = 22.1 ha (42.2%)
    - 7.15 ha – new building & Warehouse (combined building rooftop area)
    - 2.1 ha – new driveway (municipal roadway)
    - 7.15 ha– new parking lot (estimated parking area within each block)
    - 5.7 ha – new interior roads within each sub-division (estimated interior driveways within each block)
- Rooftop downspouts are discharged directly into the bioretention cell (LID 1), and parking lot and interior road runoffs are also captured by bioretention (LID 1).
- Grass Swales along both sides of the new driveway (LID 2)
- Post-development peak flow conditions are calculated as follows:
- Peak flow rate Modified Rational Formula: Qp = (1/360) * A * C * Ca * i
    - A = area in m^2 = 33.3 ha
- C = runoff coefficient = 0.69 ** (Clay loam or Clay & 42.2% TIMP, Section 21, Appendix A, KRCA Technical Guidelines for SWM, & MTO 1997 design chart 1.07 for rural)

  ** Note: Based on the MTO 1997 design chart 1.07 for rural land use, C is interpolated from 42.2 percent imperviousness to 0.51 runoff coefficient (Runoff coefficient is a dimensionless coefficient relating the amount of runoff to the amount of precipitation received). Using chart 1.07, clay is shown as a runoff coefficient of 0.55, which is the same as pre-development land use. If the effect of a key natural heritage section is not included for conservative purposes, the total imperviousness of the development area will be 70%, and using chart 1.07, C is estimated at 0.75 but using the common hydrological formula for interpolation the C coefficient:

  C = 0.2 (1-I) + 0.9 (I) = 0.2(1-0.7) + 0.9(0.7) = 0.69 (“I” is the site imperviousness ratio)

Using the first method, the C coefficient should be between 0.51 and 0.75; however, the second method estimates a value within the range closer to the upper limit. The Engineer decided to unselect the effect of Key Natural Heritage and/or Hydrologic Features (KNHHF) in runoff coefficient estimation as per the pre-development condition and input the amount interpolated from the common hydrological formula.

  - Ca = Antecedent Precipitation Factor = 1.0 for 2, 5 and 10 years, 1.10 for 25 years, 1.20 for 50 years, and 1.25 for 100 years.
  - i = average rainfall intensity in mm/hour (IDF for Scugog, Table 4 above)
  - See the post-development catchment area in Appendix IX.

- See Table 6: Post-development peak flows for the 1:2-year through 1:100-year design storms based on the Township's Rational Method along with appropriate pre-development peaks flow for each sub-catchment (block). The Modified Rational Method calculations are included in Appendix XIII for reference.

- See Table 7: To compare the Pre-development versus the Post-development peak flows for the 1:2-year through 1:100-year design storms based on the Township's Rational for each sub-catchment (block).

- See Table 8: Storage volumes (m^3) required for quantity control versus allowable (pre-development) peak flows for each block.

$$Bioretention\ Static\ Storage = (Main\ channel + subdrain) = ((B \times H \times n) + (T+B) \times 0.5) \times L \quad \text{(Eq. 1)}$$

In which:
B = bottom width (m), H = subdrain height (m), n = porosity, T = Top width (m), L = Length (m)

$$Check\text{-}dam\ Static\ Storage = (h^2 \times (h \times z + B)) / 2S \quad \text{(Eq. 2)}$$

In which:
B = bottom width (m), h = check-dam height (m), z = Channel Side slope (-), S = Ditch slope (-)

See Table 9 for volume calculations provided by check-dams in each ditch.

$$24\ hour\ Dynamic\ Storage = (A \times i \times 24) /1000 \quad \text{(Eq. 3)}$$

In which:
A = bottom Area (m^2), i = infiltration rate (mm/hr).

Table 6 – Proposed peak flows prior to LID mitigations based on the Modified Rational Method

Parameter	Block	Area (ha)	Time of Concentration (min)	Design Storm				
				2 Year	5 Year	10 Year	25 Year	100 Year
Total Runoff (Qp) in l/s (Intensity in mm/hr)	1	2.30	13.1	292.9 (66.3)	408.2 (92.4)	480.9 (108.9)	647.6 (133.3)	952.8 (172.6)
	2	2.24	14.4	269.9 (62.8)	376.1 (87.5)	443.0 (103.0)	594.8 (125.8)	876.4 (163.1)
	3	2.62	12.7	337.8 (67.3)	470.7 (93.8)	554.5 (110.5)	747.4 (135.4)	1099.1 (175.3)
	4	2.19	10.6	312.7 (74.4)	435.8 (103.7)	513.4 (122.2)	696.4 (150.7)	1020.6 (194.3)
	5	2.00	10.5	288.0 (75.0)	401.4 (104.5)	472.9 (123.2)	641.8 (151.9)	940.2 (195.9)
	6	2.05	10.6	292.0 (74.5)	407.0 (103.8)	479.5 (122.3)	650.4 (150.8)	953.2 (194.4)
	7	2.80	13.5	348.7 (65.0)	485.9 (90.6)	572.5 (106.7)	770.0 (130.5)	1133.6 (169.1)
	8	1.39	10.0	204.8 (76.8)	285.4 (107.0)	336.3 (126.1)	457.0 (155.8)	669.0 (200.6)
	9	1.48	15.4	170.9 (60.3)	238.1 (84.1)	280.5 (99.0)	375.8 (120.6)	554.2 (156.5)
	10-1	1.25	10.0	184.5 (76.8)	257.2 (107.0)	303.0 (126.1)	411.8 (155.8)	602.8 (200.6)
	10-2	1.66	17.6	177.2 (55.7)	246.9 (77.6)	290.8 (91.4)	388.2 (110.9)	573.4 (144.2)
	11	1.47	10.0	215.8 (76.8)	300.8 (107.0)	354.4 (126.1)	481.6 (155.8)	705.0 (200.6)
	12	5.12	16.1	576.1 (58.7)	802.5 (81.7)	945.5 (96.3)	1264.9 (117.1)	1866.8 (152.1)
	13-101	2.98	36.8	144.8 (34.3)	201.4 (47.7)	237.3 (56.2)	311.6 (67.1)	461.5 (87.5)
	13-102	1.77	27.9	103.6 (41.4)	144.2 (57.6)	169.9 (67.9)	224.3 (81.5)	332.3 (106.2)

Table 7 –Comparison of peak flows for pre- and post-development based on the modified Rational method.

Parameters	Block	Design Storm				
		2 Year	5 Year	10 Year	25 Year	100 Year
Post-Development Peak Flows (l/s) (***Allowable Peak Flows = Pre-Development Peak Flows in l/s***)*	1	292.2 (*245.2*)	408.2 (*341.7*)	480.9 (*402.6*)	647.6 (*543.7*)	952.8 (*720.4*)
	2	269.9 (*230.2*)	376.1 (*320.7*)	443.0 (*377.9*)	594.8 (*509.2*)	876.4 (*675.0*)
	3	337.8 (*269.2*)	470.7 (*375.2*)	554.5 (*442.0*)	747.4 (*595.7*)	1099.1 (*789.7*)
	4	312.7 (*249.2*)	435.8 (*347.4*)	513.4 (*409.3*)	696.4 (*555.1*)	1020.6 (*734.7*)
	5	288.0 (*221.4*)	401.4 (*308.6*)	472.9 (*363.6*)	641.8 (*492.2*)	940.2 (*651.7*)
	6	292.0 (*230.9*)	407.0 (*321.9*)	479.5 (*379.2*)	650.4 (*514.1*)	953.2 (*680.5*)
	7	348.7 (*266.2*)	485.9 (*370.8*)	572.5 (*436.9*)	770.0 (*586.3*)	1133.6 (*777.9*)
	8	204.8 (*163.2*)	285.4 (*227.5*)	336.3 (*268.0*)	457.0 (*364.3*)	669.0 (*481.9*)
	9	170.9 (*173.3*)	238.1 (*241.6*)	280.5 (*284.6*)	375.8 (*386.8*)	554.2 (*511.7*)
	10	361.8 (*267.0*)	504.1 (*371.9*)	593.9 (*438.1*)	800.0 (*586.8*)	1176.2 (*778.9*)
	11	215.8 (*172.0*)	300.8 (*239.8*)	354.4 (*282.5*)	481.6 (*383.9*)	705.0 (*507.8*)
	12	576.1 (*435.9*)	802.5 (*607.1*)	945.5 (*715.2*)	1264.9 (*954.5*)	1866.8 (*1267.8*)
	13	248.4 (*232.3*)	345.6 (*323.1*)	407.2 (*380.7*)	536.0 (*499.3*)	793.8 (*663.1*)

Note: (*245.2*) denotes pre-development peak flow.

* Allowable Peak Flow = Pre-dev. Peak Flow – Uncontrolled Post-dev. Peak Flow (The allowable release rate will be set to the pre-development flow as there is no Uncontrolled Post-dev. Peak Flow in this project)

Table 8– SWM Bioretention/Bioswale Storage Volumes and Release Rates

Parameters	Block	Return Period (year)					LID Length (m)	LID static storage + 24-hour dynamics infiltration volumes (m^3)	
		2	5	10	25	100			
Storage Volume (m^3) (Offsite Discharge in l/s)	1	82.5 (245.2)	96 (341.7)	113.1 (402.6)	162.2 (543.7)	259.3 (720.4)	233	293 + 80.5	373.5
	2	43.2 (230.2)	90.1 (320.7)	106.2 (377.9)	152.6 (509.2)	244.4 (675.0)	227	285 + 78.3	363.3
	3	97.1 (269.2)	113.7 (375.2)	133.9 (442.0)	192.1 (595.7)	306.5 (789.7)	264	333 + 91.4	424.4
	4	72.7 (249.2)	83.2 (347.4)	98.1 (409.3)	141.7 (555.1)	227 (734.7)	221	279 + 76.5	355.5
	5	68.8 (221.4)	79.3 (308.6)	93.4 (363.6)	134.1 (492.2)	214.8 (651.7)	202	255 + 70	325
	6	68.5 (230.9)	78.6 (321.9)	92.5 (379.2)	133.5 (514.1)	213.9 (680.5)	207	260 + 71.4	331.4
	7	112.8 (266.2)	134 (370.8)	157.8 (436.9)	224.2 (586.3)	357.4 (777.9)	283	356 + 97.7	453.7
	8	44.4 (163.2)	50.6 (227.5)	59.6 (268.0)	86.4 (364.3)	138.6 (481.9)	141	177 + 48.6	225.6
	9	47.1 (173.3)	53.7 (241.6)	63.2 (284.6)	91.7 (386.8)	147.2 (511.7)	149	188 + 51.6	239.6
	10	121.7 (267.0)	145.5 (371.9)	171.4 (438.1)	243.4 (586.8)	387.1 (778.9)	294	371 + 101.7	472.7
	11	46.8 (172.0)	53.3 (239.8)	62.8 (282.5)	91 (383.9)	146.1 (507.8)	148	187 + 51.2	238.2
	12	229.7 (435.9)	277.8 (607.1)	327.2 (715.2)	461 (954.5)	733.1 (1267.8)	518	652 + 178.9	830.9
	13	193.6 (232.3)	241.7 (323.1)	284.8 (380.7)	397.3 (499.3)	628.6 (663.1)	2806	701 + 323.3	1024.3
					Total Vol. (m^3)			4338 + 1321	5659

There is a pond in the northeast corner of the site property within the non-disturb area, and development has no effect on it. On the other hand, the northeast block (Block 10) comprised of two separate drainage catchments (i.e., 10-1 & 10-2), with sub-catchment 10-2 being adjacent to the pond and directing the pre-development flow towards the pond.

6. Proposed Low-Intensity-Development Infiltration Structures

Since the total post-development peak flows from the site exceed the pre-development levels due to TIMP increases, quantity controls are required. Due to the poorly draining clay soil surrounding the site, the Engineer proposed that all impervious areas such as rooftops, asphalt parking lots, and interior roads be connected into bioretention landscape areas, and after primary treatment and infiltration, the overflow will be discharged into the roadside bioswales. Moreover, asphalt roadways will be graded to two grass swales with flat bottoms for treatment and infiltration. See Appendix II & XII for detailed design information and the SWM flowchart (Fig. 4 below).

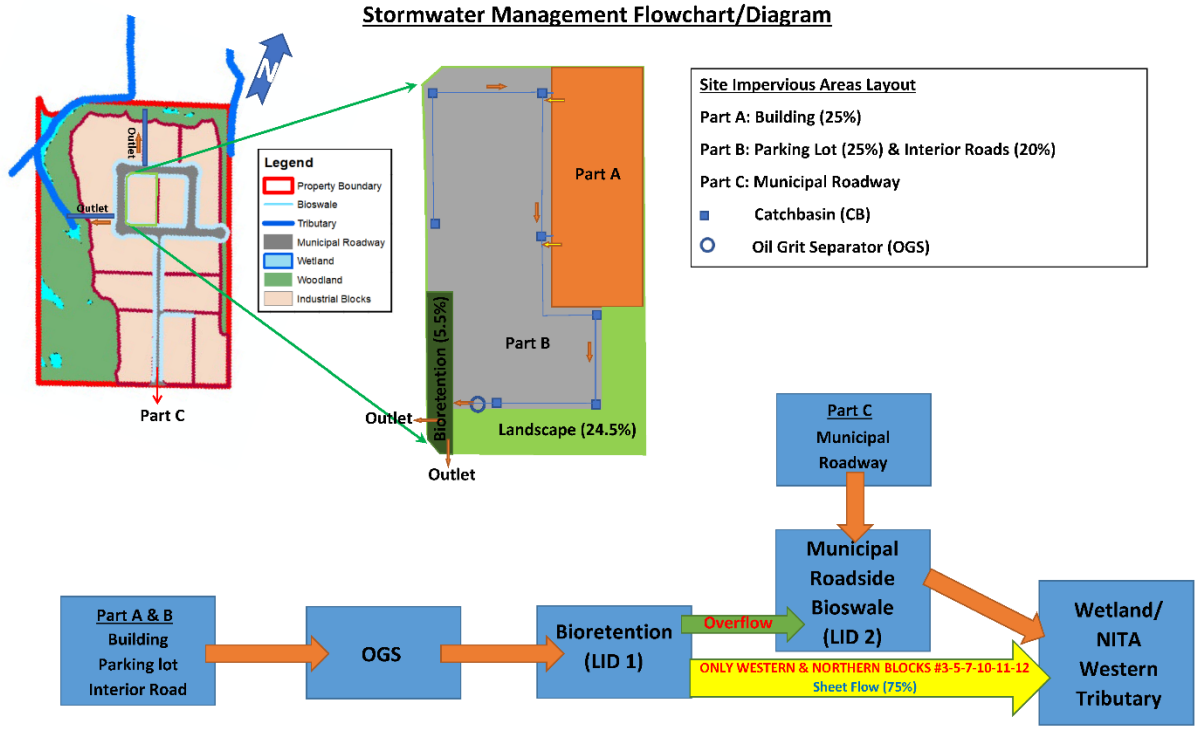


Figure 4- Stormwater Management Flowchart/Diagram

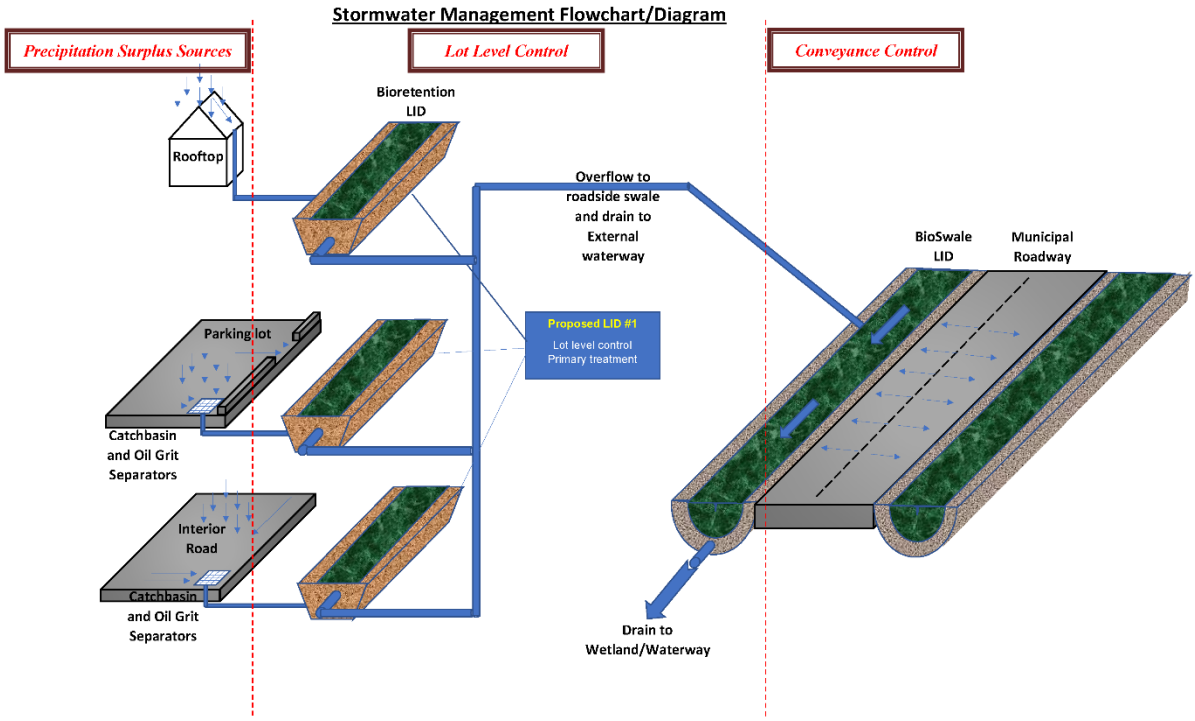


Figure 5- Low-Intensity Development – Source Control, Lot-Level Control, and Conveyance Control

6.1. Bioretention for Lot Level Control (LID 1)

Bioretention basins are landscaped depressions that intercept stormwater runoff from adjacent paved surfaces to temporarily store, filter, and infiltrate it into 0.5 – 1.0 meter of soil. They are commonly placed alongside or in parking lots, where they help remove suspended solids, heavy metals, and other pollutants from the lot's stormwater runoff through a mix of physical, biological, and chemical processes. In addition to these filtration capabilities, bioretention basins can effectively reduce peak runoff rates and volumes for relatively frequent storms.

- Flat bed bioretention with underdrain pipes on the site property (LID 2), in the front portion of each block along access driveways and designed to receive 1:100 year (major storms) from rooftops, interior roads, and parking lots using Scugog's IDF curve values.
- Bioretention is designed at 0.4 m depth, 3.0 m bottom width, 5.4 m top width at 3H:1V sloped sides, cross-sectional area of 1.68 m^2.
- Receives 100% precipitation from rooftops, parking lots, and interior roads, for an impermeable area of 20 ha.
- 0.55 m depth underdrain system, consisting of a 0.4 m gravel layer, a 0.05 m sand layer, and a 0.1 m soil bed.
- The total static storage volume is 3637 m^3, and the dynamic infiltration volume per hour is approximately 41.6 m^3/hour from the channel bottom.
- Bioretention collects stormwater from all rooftops plus impermeable parking lots and interior roadways. Two different options proposed are for rainwater collection:
    - 1. Curb-cut openings that allow stormwater to enter the linear bioretention basins from the gutters or with a grass filter strip for roads without curbs, or
    - 2. Catchbasins and Oil Grit Separators to collect roadway runoff and provide the opportunity for stormwater to infiltrate.

- Drawdown time is 41.6 hours for LID 1 main channel with a maximum water depth of 0.2 m with a conservative engineering infiltration rate of 4.8 mm/hour for bioretention basin and 37.5 hours for its sub-drain media with a maximum water depth of 0.45 m with a factored engineering infiltration rate of 4.8mm/hour.

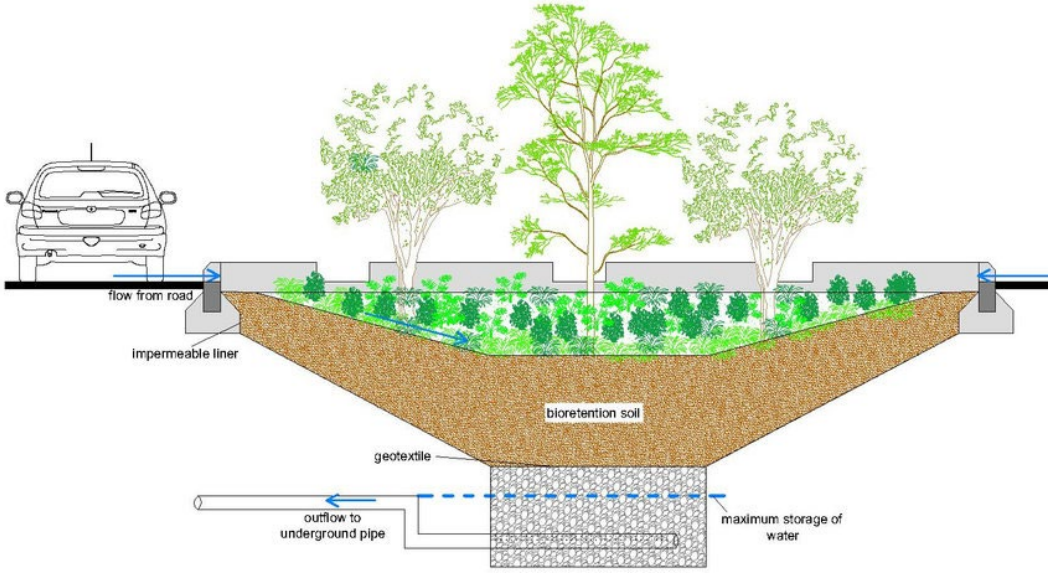


Figure 6- Sample Infiltration / Exfiltration Systems (From the Web)

6.2. Bioswale for Conveyance Control (LID 2)

Biofiltration swales are one of several LIDs for the treatment of stormwater runoff from project areas that are anticipated to produce pollutants of concern (e.g., driveways). Bioswales are vegetated, typically trapezoidal channels, which receive and convey storm water flows while meeting water quality criteria and other flow criteria.

- Flat bed bioswales with check dams and underdrain pipes on the site property (LID 2), along both sides of new municipal roadways, and designed to receive 1:100-year storms using Scugog's IDF curve values. Total number of check dams will be 63.
- Swales are designed at 0.9 m depth, 1.0 m bottom width at 3H:1V sloped sides, cross-sectional area of 2.88 $m^{2,}$ and underdrain pipe with 0.3 m diameter.
- Concrete check dams with a pond height of 0.2 m, and usually installed based on ditch grading along the swale (See Table 9 for details)
- The gravel used for the drainage layer is to be 20-50 mm (stone size), uniformly graded, clean, crushed angular stone that has a porosity of 0.4.
- Total static storage volume behind check dams (#49) is approximately 224 m^3 plus 477 m^3 of subdrain volume (2806 m length, with a 0.5 m underdrain system, consisting of a 0.3 m gravel layer, a 0.1 m choker layer, and a 0.1 m Filter media), and dynamic infiltration volume per hour is 13.5 m^3/hour from the total area of channel bottom.
- Swales grade north or west at roughly 0.8-1.7 %, and it is recommended to plant small landscaping shrubs, grasses, and forbs on the bank and the channel bed to retain their long-term shape. See the secondary plan for suggested landscaping.
- Swales collect stormwater from all impermeable municipal roadways as well as stormwater overflow from the block's bioretention practices.

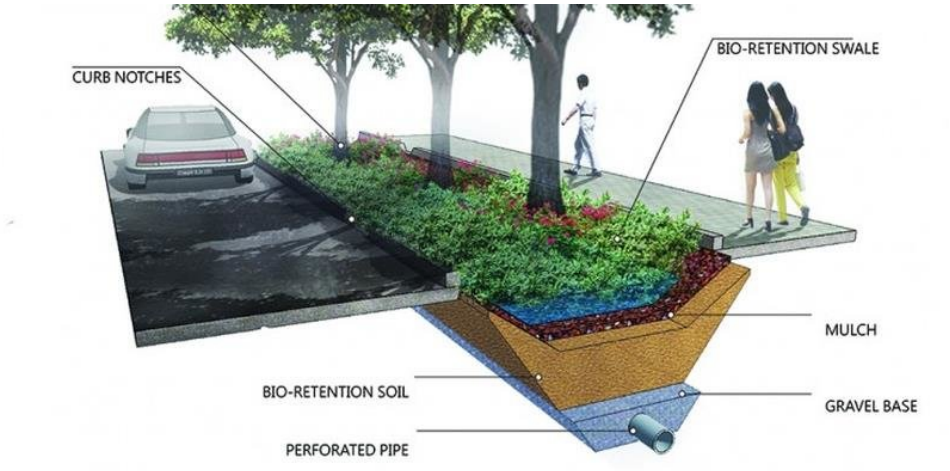


Figure 7- Sample Bioswale Section (From the Web)

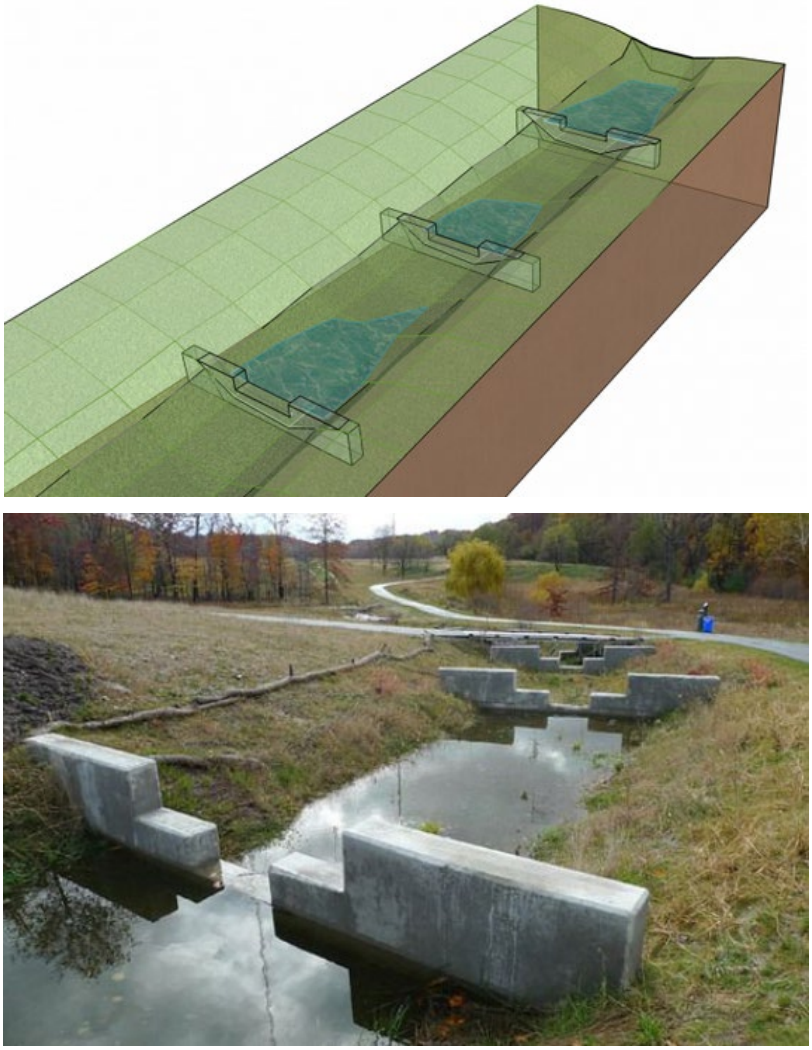


Figure 8- Sample Swale / Concrete Check Dams (From the Web)

- Assumes 56% of captured volumes are infiltrated/recharged into groundwater and the rest would be directed to downstream wetland or waterway (56% pervious area in the municipal roadway).
- Drawdown time is 41.6 hours for the LID 2 main channel with a maximum water depth of 0.2 m with a conservative engineering infiltration rate of 4.8 mm/hour for the bioswale basin and 41.6 hours for its sub-drain media with a maximum water depth of 0.5 m with a factored engineering infiltration rate of 4.8 mm/hour.

Table 9 – Design of Check Dam for Retainment

Catchment	Ditch	Length (m)	Grading (%)	Distance between check dams (m) **	Number of Check Dams (#) **	Water Volume Capacity (m^3)	Infiltration Footprint Area (m^2) *	Infiltration (m^3/hr)
101	a	196.3	0.6	67	3	16	75.5	1.1
	b	104.6	0.6	67	2	8	75.5	0.6
	c	109	0.6	67	2	9	75.5	0.6
	d	126.9	0.6	67	2	10	75.5	0.7
	e	208.3	0.5	80	3	17	90.6	1.1
	f	452.5	0.6	67	7	36	75.5	2.5
	g	180.4	0.85	47	4	14	53.3	1.0
	h	163.7	1.8	22	7	13	25.2	0.9
	i	135.5	1.4	29	5	11	32.4	0.7
	j	122.3	0.5	80	2	10	90.6	0.7
	k	92.7	0.5	80	1	7	90.6	0.5
102	a'	475.5	0.5	80	6	38	90.6	2.6
	b'	161.9	0.5	80	2	13	90.6	0.9
	c'	94.4	0.5	80	1	8	90.6	0.5
	d'	51	0.5	80	1	4	90.6	0.3
	e'	49.2	1	40	1	4	45.3	0.3
	f'	81.9	1	40	2	7	45.3	0.4
Total		**2806.1**			**49**	**224**	**1213**	**15.3**

* *Infiltration footprint area* $= (B+ 2 \times h \times (1+z^2)^{0.5}) \times Distance / 2$

**** Note: According to the ditch grading and the maximum height of the check-dams (20 cm), a large number of check-dams with extra storage volume will be provided. Therefore, the engineer decided the check-dams to be built not each one in a series, but every second.**

See Appendix XII for a detailed SWM flowchart, and Appendix XIII for LID sizing design information.

7. Water Quantity balance change

7.1. Peak flow control

Stormwater runoff peak flow discharges must be controlled to the pre-development levels for the 2-to-100-year design st4orms for the Subdivision Plan area in accordance with the KRCA's criteria for peak flow control and the guidelines of the Ministry of Environment Stormwater Management Planning and Design Manual, March 2003.

Prior to development, the property is considered nearly vacant with a total impervious site area of 0.1% near the south portion of the property (a trailer and gravel driveway), with cultivated area, trees, hedgerow, grasses, and wetlands for the remaining 99.9%. The property is graded 1.25% towards the northwest, with soil made up of primary clay, with very low infiltration rates.

Post-development has a TIMP = 42.2% and combined with the proposed construction of two (2) LIDs, there are not any expected changes in peak runoff flow rates or total volumes. The bioretention landscape areas (capturing runoff from parking lots and interior roads plus rooftop downspouts), and the bioswales surrounding the municipal roadway. Using these LIDs in multiple locations produces the required mitigation results without using a traditional SWM pond which requires a relatively large land area and has the potential breeding habitat for mosquitoes.

Traditional major development projects make use of SWM ponds to collect large volumes of stormwater to be stored on-site to allow settling to reduce erosion and increase water quality. Ponds are most useful during large storm events where increased volume produces minimal infiltration. Based on monitoring well records, groundwater at the subject site is very high (0.4 – 1.7 m), and the soil material was found to have a low infiltration rate (50.3 min/cm). The Engineer has provided their opinion that a stormwater settling pond would not be beneficial for the subject site due to the soil material and high groundwater. In lieu of a SWM pond, LID techniques have been proposed to increase the infiltration rates at bioretention and bioswale locations and provide underground storage volume for each block. These techniques provide adequate mitigation to control the peak flows to pre-development levels for all storm events up to the 100-year event. Additionally, the KRCA requires the detention of 25mm of stormwater runoff, which is 5518.5m^3 for this site. The proposed LIDs provide volume storage for this amount in addition to the 100-year event volume.

For a 1:100-year storm, there is an excess of 3375.4 m^3 of peak flow capacity when comparing pre-development and post-development without any mitigations for the whole site (13 blocks including 12 industrial blocks and one municipal roadway block). The static storage volume of the proposed LIDs is 4338 m^3, which is sufficient to fully reduce peak flow capacity. See Appendix XIII for more details on calculations.

Post-development peak flows for each catchment (block) are calculated above using the Modified Rational Method (Tables 7-8). The calculations are included in Appendix XIII.

In summary, the proposed LIDs 1, and 2 are adequately sized to reduce peak flow rates of the site property when comparing pre-development and post-development with mitigation measures.

7.2. Drawdown Time Review

The drawdown time is calculated as the time (in hours) for the depth of water in the storage at the end of a runoff event to be infiltrated into the native soils through the bottom of the LID. The depth is assumed to be the full depth of the storage layer. The seepage rate is the factored infiltration rate of the native soils. The formula for drawdown time is:

Drawdown Time (hr) = Water depth in Storage (mm) x Porosity / Seepage Rate (mm/hr) (Eq. 4)

LID 1 (Bioretention with TIMP= 200,056 m^2)

Drawdown Time (hr) = 200 (mm) / 4.8 (mm/hr) = 41.6 hr *(Main channel)*

Drawdown Time (hr) = 450 (mm) x 0.4 (fraction) / 4.8 (mm/hr) = 37.5 hr *(Sub-drain)*

Due to the 0.2 meter height of the catchbasin (max water depth), and the assumed infiltration rate of 50 mm/hr for the bioretention media, the calculated drawdown time for the proposed design (main channel) is approximately 4 hours, which is less than the typical maximum target drawdown time of 48 hours. This assumption is correct if the porous medium under the bed (sub-drain area) is not saturated; otherwise, the same conservative infiltration rate of 4.8 mm will be true, in which case the drawdown time will increase to 41.6 hours which is still less than the typical maximum target drawdown time of 48 hours. For the 0.45m sub-drain thickness and factored infiltration rate of 4.8 mm/hr, the calculated drawdown time for the proposed design (sub-drain) is approximately 37.5 hours, which is less than the typical maximum target drawdown time of 48 hours.

This infiltration system is a practice used to manage stormwater runoff and to prevent flooding and downstream erosion. It is designed to infiltrate stormwater though permeable soils into the groundwater aquifer. Infiltration systems do not discharge to a surface water body under most storm conditions but are designed with overflow structures.

LID 2 (Driveway Grass Swale with TIMP= 20,683 m^2)

Drawdown Time (hr) = 200 (mm) / 4.8 (mm/hr) = 41.6 hr *(Main channel)*

Drawdown Time (hr) = 500 (mm) x 0.4 (Fraction) / 4.8 (mm/hr) = 41.6 hr *(Sub-drain)*

Due to the 0.2 meter height of the check dam (max water depth), and the assumed infiltration rate of 50 mm/hr for the bioswale media, the calculated drawdown time for the proposed design is approximately 4 hours, which is less than the typical maximum target drawdown time of 48 hours. This assumption is correct if the porous medium under the bed (sub-drain area) is not saturated; otherwise, the same conservative infiltration rate of 4.8 mm will be true, in which case the drawdown time will increase to 41.6 hours which is still less than the typical maximum target drawdown time of 48 hours. However, the 0.50 m sub-drain thickness and factored infiltration rate of 4.8 mm/hr, resulted in the calculated drawdown time for the proposed design (sub-drain) being approximately 41.6 hours, which meets the typical maximum target drawdown time of 48 hours.

7.3. Volume control

Refer to the KRCA Technical Guidelines For Stormwater Management Submissions, for all new developments and re-developments >2 ha, 24-48 hour extended detention of the **25mm storm** is required to protect the downstream receiver of runoff from erosion.

The proposed site will have a total impervious surface area of 22.1 hectares (42.2% of the site), which is above the industry standard of stormwater volume control requirement of 2 hectares. In Table 10 the total runoff volume control target for post-development scenario (25 mm storm event) and volume capacity of proposed LIDs are shown. The total volume of stormwater runoff produced due to a 25 mm rainfall event from the total impervious area of the study site is 5,519 m^3, which is less than the storage capacity intended for storm runoff at the site 5659 m^3, as discussed above in Section 6.
Table 10 shows that runoff volume reduction is met, and that the post-construction runoff volume shall be fully captured and retained on the site from a 25 mm rainfall event from the total impervious area.

Table 10 – Runoff volume control target (25 mm) and values provided by post-development design (m^3)

Parameter	Building	Parking	Interior Road	Roadway	Total
TIMP Area (m^2)	71,448	71,448	57,159	20,683	220,738
25 mm runoff volume control (m^3)	1786.2	1786.2	1429	517.1	5,518.5
Building + Parking + Interior Road: LID 1 volume capacity for rooftop + parking lot + interior road = 4634.7 m^3 versus a volume control target of 5,001 m^3					
Municipal Roadway: LID 2 volume capacity for new roadway = 1024.3 m^3 versus a volume control target of 517 m^3					
Total LID volume capacity = 4634.7 + 1024.3 =5659 m^3 versus a site-wide volume control target of 5,519 m^3					
The drainage system (bioswale & outlets) will be designed to manage storm water for up to the 100-year design storm consistent with the Township of Scugog Design Criteria and Standard Detail Drawings (2003). Peak flows up to the 100-year design level would be contained within the LIDs prior to release to the environment. The proposed conveyance system (bioswale) to the outlet at a tributary of Nonquon River was sized to convey major flow contributing to the impervious areas of the river.					

The bioretention landscape areas (capturing runoff from rooftops, parking lots, and interior roads), and the bioswales surrounding the municipal roadway. Using these LIDs in multiple locations produces the required mitigation results along with the total volume control target (25 mm). Total LID static-storage volume = 5659 m^3 while 25 mm runoff volume control requires 5,519 m^3.

7.4. Major-minor system conveyance

This site is classified as a low infiltration area due to the presence of highly compacted clay soil in most parts of the developed area, 11.9mm/hr unfactored infiltration rate vs 1:100 year 4-hour event of 20.88mm/hr precipitation.

The minor system conveyance is for flows below a 5-year storm event (i.e., 11.84mm/hr precipitation in 4 hours). Despite the low permeability of natural clays on site, 4-hour precipitation on permeable surfaces will be fully infiltrated, while the probable runoff due to lower rainfall duration and more intensity would be drained to on-site wetlands or outlet watercourses. Similarly, all precipitation on impermeable surfaces of developed areas will be drained to their respective LIDs for storage and infiltration (bioretention and bioswale), with a small chance of discharging the overflow to wetlands and downstream tributaries.

The major system conveyance is for flows between 5- and 100-year storm events, and is the same for pre-development and post-development, generally flowing from southeast to northwest. Pervious woodlands/wetlands runoff Blocks 14-15 (undeveloped area) will follow the pre-development grades, flowing northward through the watercourses in the key natural heritage section. Impervious area major system conveyance remains the same, as all LIDs were sized to receive 1:100-year storm events.

Overland routes on the western section of the site which are undisturbed are expected to follow the pre-grading, with the south portion being a locally elevated region, and flowing northwest towards Nonquon tributary. The overflow locations from bioswales LID to the west and north wetlands (watercourses) are marked on the site grading plan.

There are no major differences between pre-development and post-development overland routes. The runoff direction in pre- and post-development scenarios has been shown in Appendix XIV and shows consistency with the direction of the land's grading to the northwest. As can be seen in Appendix XIV, this site contains small subwatersheds, all of which eventually drained into the tributary of the Nonquon River in the northwest. In the Engineer's proposed plan, three outlets have been considered, which are in accordance with the current pre-development conditions, and their localized discharge amounts are the same as in existing conditions.

7.5. Regulatory storm conveyance

The site property is serviced by different ditches along the southwest, northwest, and northeast corners of the property. The northwest ditch is the main drainage outlet for the developed area. The southwest branch is connected to the southern area of the Scugog Line 6 road through a roadway culvert. The southwestern watercourse is responsible for the safe transfer of the regulatory storm from the southern part of the Scugog Line 6 Road to the tributary of the Nonquon River, which is located completely outside the developed area and will remain unchanged. The neighbors to the east of the site drain with other northeastern ditches, and then connect to a natural watercourse that flows north to the Nonquon tributary.

7.6. Flooding Review

Greenland International Consulting (Greenland), in association with Stantec Consulting, was retained in late 2001 to undertake the “Nonquon River Subwatershed Study” (Greenland International Consulting Ltd. & Stantec Consulting Ltd., 2005). By studying the hydrologic and hydraulic processes associated with the Nonquon Industrial Tributary Area (NITA), they proposed a sustainable Master Drainage Plan (MDP) for reducing flood damages and risk to existing and future businesses and ensuring public safety.

The goal of a Master Drainage Plan (MDP) specifically for the NITA portion of the Subwatershed Plan, is to establish an environmentally sensitive approach, prior to the approval of any plan of development for the handling of storm water runoff/snowmelt from urbanizing areas. The proposed MDP for the NITA includes preliminary design details of a centralized storm water management facility, located at Regional Road 8 (Reach Street) between the existing Northport and Reach Subdivisions.

The recommended MDP identifies possible locations and contributing areas for a total of six (6) future storm water management facilities, which would be constructed as the NITA lands become further developed in the future. In addition to SWM facilities, the MDP also makes recommendations on the

implementation of other structural and nonstructural strategies to mitigate current nuisance flooding conditions within the NITA lands including the installation of additional culverts under Reach Street to increase the conveyance of storm water toward the Nonquon River through the PSW (Provincially Significant Wetland).

Storm water runoff is conveyed by two main tributaries within NITA, as shown in Appendix XV and Figures 12-13, which cross 0 Scugog in the northwest and northeast corners. The tributary empties into the wetland located on the north side of Reach Street. According to the last study (Greenland International Consulting Ltd. & Stantec Consulting Ltd., 2005), the hydraulic computer modeling (HEC-RAS) analysis was used to assess in-stream hydraulic conditions along watercourses, including potential water level increases at culvert/bridge crossings where rating curve relationships were calculated (Figure 12 shows the layout of the old Hac-RAS model and cross sections along the creek). Furthermore, a post-development HEC-RAS model was developed to estimate water surface elevations in the NITA and the change associated with the implementation of the new facility using post-development flows. Use of the 5-year post-development flow rate assumes full development has occurred and six (6) SWMFs are operational within the NITA lands. Results are presented in Table 5.10 (Appendix XV) for the condition of high (254.27 m) and low (252.71 m) water surface elevations in the downstream PSW. Figure 5.5 in that report indicates cross-section locations.

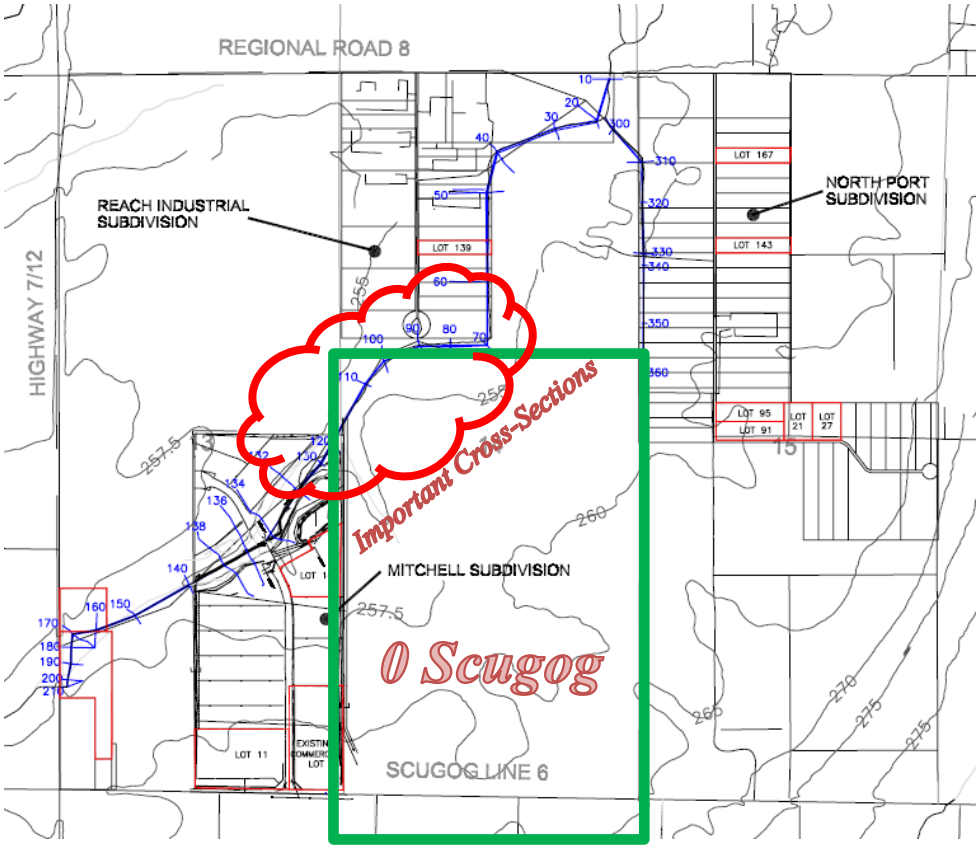


Figure 12- Old HEC-RAS model layout for NITA with cross-sections locations (Greenland International Consulting Ltd. & Stantec Consulting Ltd., 2005)

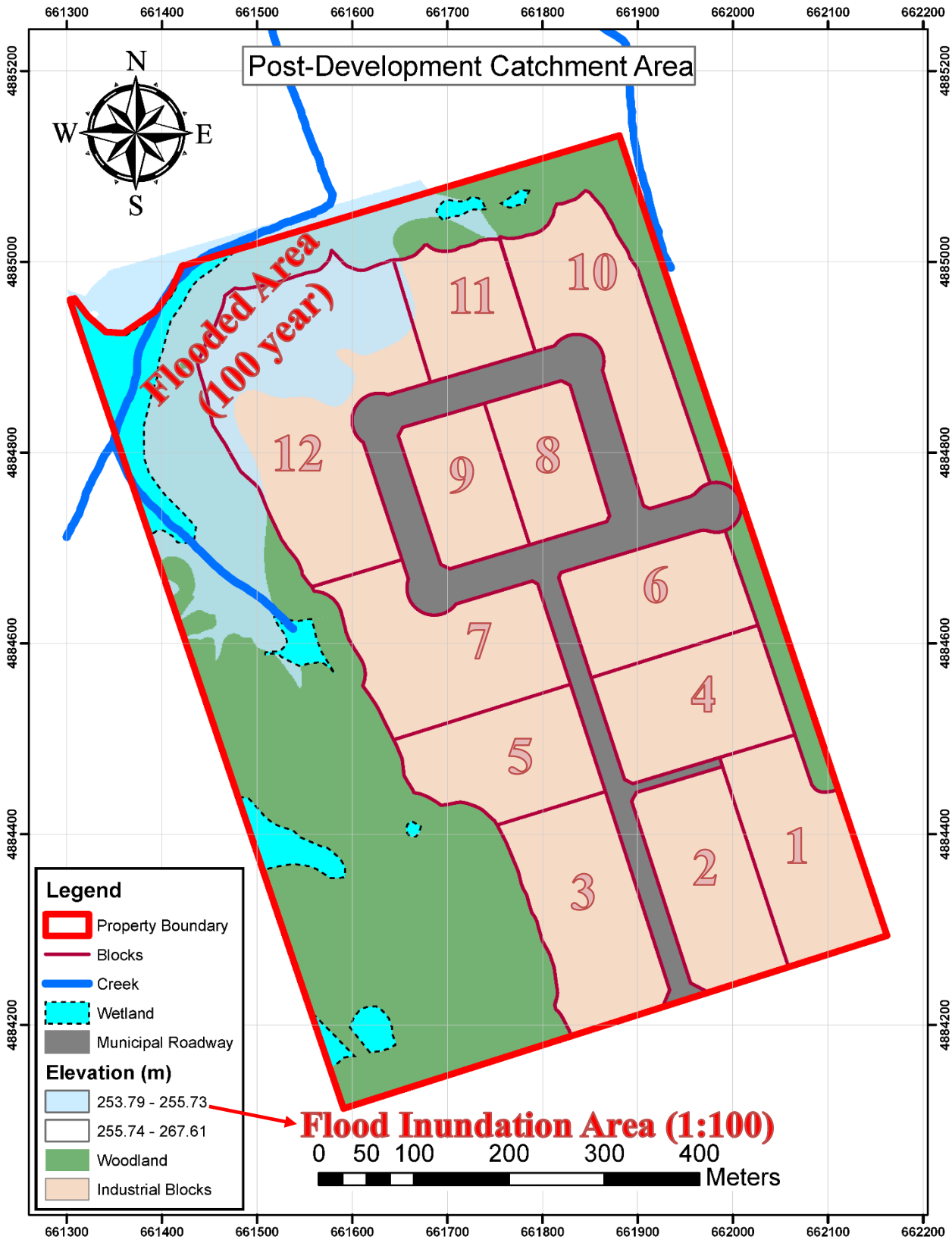


Figure 13- Estimated flooding elevation line (100-year flood)

Appendix XV showed 7 cross-sections of the Main NITA tributary located in the northwest corner of the site. The output results of the previous model for these sections in the 100-year storm events are presented in Appendix XV and Figure 13, as well. As the results show, the water level in the northwest corner will be 255.73 m, which is within the desired flooded zone shown on the new development plan. Only about half of Block No. 12 in the northern part could experience flooding due to a 100-year event. Looking at Figure 13, the depth of flooding in most of this area is less than 0.3 m which is allowable for ponding within the parking lot, prior to overflowing and in the rest of this zone will be a maximum of 0.5 m.

8. Water Quality

8.1. Total Suspended solids

Stormwater quality controls will be implemented in accordance with requirements of the Township of Scugog and Kawartha Region Conservation Authority (KRCA) with a targeted Total Suspended Solids (TSS) removal of 80% in accordance with provincial policy.

The proposed post-development has all impervious areas draining the entirety of the minor and major storm event volumes into the twelve (12) independent LIDs (bioretention) and their overflows will be discharged into another LID (bioswale). Based on MOE 2003 SWM PDM Table 3.2, Enhanced 80% S.S. Removal by infiltration requires 30 m^3/ha for 55% imperviousness. For the subject site, TIMP is 42.2%, and required storage volumes are obtained through interpolation of Table 3.2, about 26.8 m^3/ha. Using 26.8m^3/ha to meet the required 80% TSS removal and the area of the site 33.3 ha, a minimum of 892.4m^3 of storage must be provided. The proposed site has 5659m^3 of infiltration storage; therefore 80% TSS removal has already been achieved through stormwater quantity control measures.

When considering a series of proposed LIDs as described by the New Jersey Stormwater Best Management Practices Manual, 2004, Chapter 4, it is recommended to calculate TSS reduction based on the removal rate of the second BMP (Best Management Practice) or LID (Low-intensity development) applied to the fraction of the TSS load remaining after the runoff has passed through the first BMP/LID. If there is a third LID, the calculation continues in the same way.

In this study, there are two infiltration LIDs in series in each block along with a third LID in the municipal roadway to receive the overflow from the upstream LIDs with the following individual TSS removal rates:

Bioretention basin (LID 1) & Bioswale (LID 2) = 90% TSS removal rate

Figure 14 shows schematics of proposed treatment trains for different areas with Low Impact Development options. The municipal roadway only has a single LID treatment (LID 2), and individual parking lots, interior roads, and rooftops have two treatments (LIDs 1-2).

1# Municipal Roadway (= 47,471 m^2)

Initial TSS Load Upstream of Bioswale (LID 2) = 1.0
Municipal Roadway TSS Removal Rate = 1.0 X 90% Removal Rate = 0.9 or 90%

2# Rooftops (= 71,448 m^2) + Parking Lots (= 71,448 m^2) + Interior Roads (= 57,159 m^2)

Initial TSS Load Upstream of Bioretention (LID 1) = 1.0
TSS Load Removed by Bioretention (LID 1) = 1.0 X 90% Removal Rate = 0.9
Remaining TSS Load Downstream of Bioretention which drained to bioswales = 1.0 – 0.9 = 0.1
TSS Load Removed by Bioswales (LID 2) = 0.1 X 90% Removal Rate = 0.09
Final TSS Load Downstream of Bioswales which drained to wetlands/creek = 0.1 – 0.09= 0.01
Rooftops & Parking Lots & Interior Roads TSS Removal Rate = 1.0 – 0.01 = 0.99 or 99%

Finally,

The averaged area TSS removal rate =
(0.9 × 47,471 + 0.99 × 71,448 + 0.99 × 71,448 + 0.99 × 57,159) / 247,526

The averaged area TSS removal rate = 0.973 or 97.3%

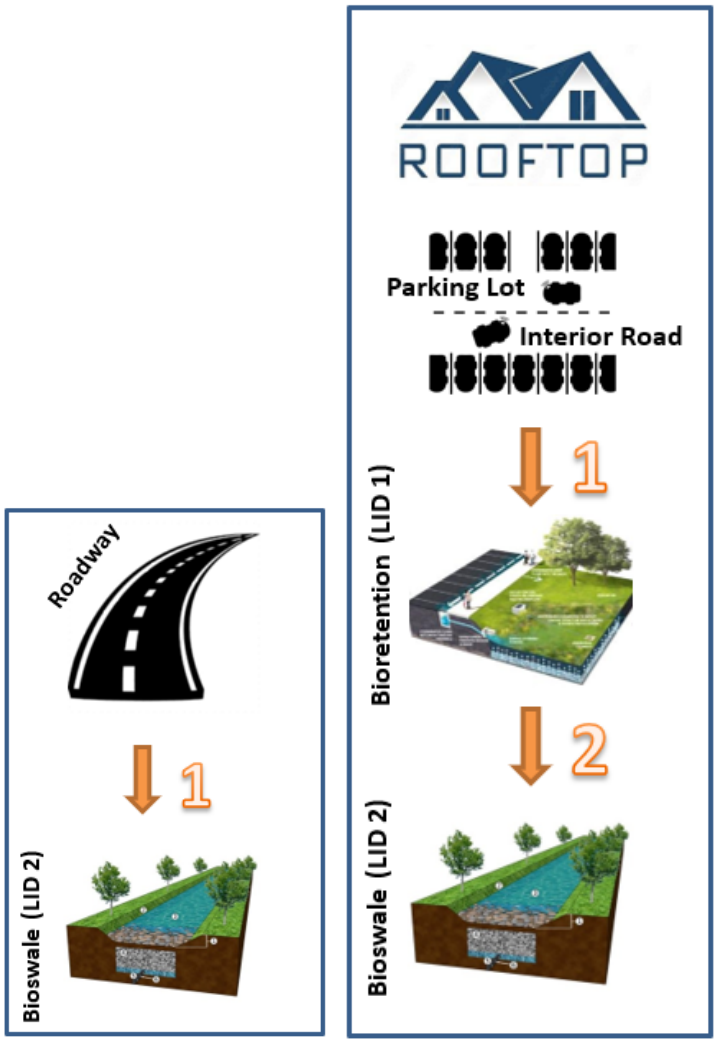


Figure 14- Schematic showing proposed treatment trains for different areas

Based on the calculation method as referenced in New Jersey Stormwater Best Management Practices Manual, 2004, Chapter 4, the KRCA 80% TSS reduction on "developed areas / impervious areas" is met.

8.2. Total Phosphorus Budget

Kawartha Conservation requires a net zero increase of the annual Total Phosphorus (TP) load from all major development areas. Full infiltration and volume reduction were selected to meet the targets for the site property.

The proposed development site has been modeled using the NVCA Phosphorus Loading Development Tool (PLDTv2, Last update on October 19, 2021) developed by the NVCA, Nottawasaga Valley Conservation Authority as a modification of a tool developed by the Province of Ontario for the Lake Simcoe Watershed in 2012.

PLDT was modeled for the following modules:

Module 1 – Estimates pre-development phosphorus loads for representative land uses (categorized in separate Natural Heritage, Urban, and Cropland sub-types) contained within the study site,
Module 2 – Estimates post-development phosphorus loads that are representative of the proposed land uses for the study site without BMPs (Best Management Practice) and LID (Low-intensity development) techniques to reduce phosphorus loads,
Module 3 – Estimates the reduction in phosphorus loads from the post-development scenario with the implementation of BMPs and LID techniques, and
Module 4 – Provides a checklist for users to guide the selection and implementation of BMPs for the construction phase of development to minimize sediment loss and resultant phosphorus export.

The NVCA Tool is based on an export coefficient approach in recognition that export coefficients address the export of both dissolved and particulate phase phosphorus and that recent estimates were available for the Lake Simcoe watershed. Export coefficients were modified to reflect site-specific soil and runoff characteristics for those land use classes that are most often changed with urbanization (cropland, high-intensity residential, industrial, commercial, and transportation).

Phosphorus export coefficients for the following land use classes were derived as the mean phosphorus export for all 'monitored' Lake Simcoe subwatersheds (n = 7) using phosphorus loads and land use areas from CANWET modeling:

- Forest, Transition, Wetland, Open Water, Turf/Sod, Hay/Pasture, Low-Intensity Residential, Unpaved Roads, Open Water

Phosphorus export coefficients for cropland were derived from the relationship between CANWET-derived phosphorus export for Lake Simcoe subwatersheds and soil loss (A) as estimated using the Universal Soil Loss Equation (USLE):

- Phosphorus Export (kg/ha/yr) = (0.16 x A) +0.16

Phosphorus export coefficients for urban land use classes (high intensity residential, commercial,

industrial, and transportation) were derived from an equation using a standard value for phosphorus concentrations in urban runoff (TPi), annual depth of precipitation (Precip), that fraction of precipitation producing runoff (Pi) and a runoff coefficient derived for impervious surfaces (Rv):

- Phosphorus Export (kg/ha/yr) = TPi x Precip x Pj x Rv x 10^{-2}

These estimates are recommended for use in Modules 1 and 2 of the NVCA Tool to estimate phosphorus export for Pre-Development and Post-Development land uses. Module 3 addresses means to mitigate and reduce phosphorus loads by implementing approved BMPs and LID techniques. Taken together, Modules 1, 2, and 3 allow the user to calculate and compare pre-development phosphorus export to post-development export, with and without BMP/LID implementation.

Results were modeled to show Total Phosphorus runoff from the new development impervious areas was reduced from 60.61 kg TP/year to 20.57 kg TP/year using LIDs 1 to 2 (Appendix XI).

Pre-development with TIMP = 0.1%, outgoing TP of 38.75 kg/year
Post-development with TIMP = 42.2%, outgoing TP of 60.67 kg/year
Post-development with TIMP = 42.2% draining into LID's, outgoing TP of 21.47 kg/year

When comparing the effective TP removal, the Engineer used the following formulas:

i. Changes in controlled phosphorus concentrations compared with existing conditions should be represented by:

100 x [(Controlled – Existing)/Existing]
100 x [(21.47 – 38.75)/38.75]
100 x [– 0.45]
45% reduction in Total Phosphorus concentrations versus existing condition

j. ii. The effective reduction of Total Phosphorus in the post-development with mitigation conditions should be represented by:

100 x [(Controlled – Uncontrolled)/Uncontrolled]
100 x [(21.47 – 60.67)/60.67]
100 x [– 0.65]
65% reduction in Total Phosphorus concentrations versus Post-development

Without any mitigation, average annual runoffs would release a total of 60.67 kg of phosphorus due to the proposed development. When LID filtration and mitigation measures are made, the additional release of phosphorus from the controlled catchment area is brought down to 21.47 kg. In summary, there is a 45% reduction in total phosphorus discharge when comparing post-development with mitigation (controlled) and pre-development (existing) scenarios while 65% reduction in total phosphorus discharge when comparing post-development with mitigation measures (controlled) and post-development (uncontrolled) scenarios using the NVCA Phosphorus Loading Development Tool.

8.3. Winter salt

Post-development winter salt use is anticipated to be higher than pre-development. This is primarily a concern for the driveways, parking lots, and interior roads with a total asphalt area equal to 16.05ha (30.6% of total area). Design practices that help reduce winter salt use include:

- Minimal trees or use of deciduous plants are considered for landscaping near driveways/roads, to reduce winter shading.
- Effective parking lot grading can minimize the freezing of wet pavement surfaces as well as prevent melt water from ponding and refreezing, reducing the need for re-application of salts.
- Effective drainage management can reduce salt use over the life of the facility. Proposed drainage LID systems will lead to reduced ice accumulation, and as such, reduce salt usage.
- Proposed bioswales and bioretention with curb cut inlets collect and retain melt water runoff, reducing melt water ponding and refreezing, and thus, salt usage will reduce.
- Parking lots are designed with catch basins located near the perimeter so melt water from snow storage areas does not have to flow a long way across paved surfaces where it could freeze and require excessive salting.
- Effective drift control can reduce snow and ice control efforts including salt use. Drifting can be controlled through the erection of drift control devices such as snow fences (structural and living) and snow ridges and the strategic placement of buildings.
- By designing sidewalks that are wide enough for mechanical clearing and anticipating pedestrian flow, the number of walkways and salt required on them is minimized.
- All driveways are graded up towards block entrances, preventing ice build-up at entrances.
- Recommend the client use sand & gravel for traction instead of salt.

8.4. Temperature

Since groundwater is very high and the soil material has a low infiltration rate, no stormwater settling ponds are designed or to be built, as LID proposals are full infiltration, with minimal overflow in case of interference of precipitation event with the time of complete drawdown of the last event. Therefore, temperature is not an issue because there are no open areas of water with a negative impact on aquatic life.

8.5. Other contaminates

The site property is not serviced by the township storm sewer system. Post-development contaminates generated in an industrial estate land-use setting would be captured by infiltration into the ground. Stormwater runoff from rooftops, interior roads, and parking areas will require water quality treatment to an 80% TSS removal standard, as per the KRCA criteria. This will be accomplished mainly through the installation of oil-grit separator (OGS) units at each discharge point for buildings and sewer services. Each OGS will be sized based on its contributing catchment area and the overall imperviousness of the catchment. Furthermore, Oil-grit separators (OGS) are designed to be installed in outflow directions as part of a treatment train to remove heavy particulates, floating debris, and hydrocarbons from stormwater. See the SWM plan in Appendix II to find the marked location of the proposed OGS (EGR 1.3).

9. Stream Erosion Analysis

There are two small watercourses within a key natural heritage that drain the site land to the northwest part of the site and join the main tributary flows northeast to Lake Scugog through the Nonquon River.

Other ditches were located near the east boundary of the site property which are connected to the main tributary at the north side. The Nonquon River is the largest watershed flowing into Lake Scugog. The internal small ditches (bioswales) would be covered with dense vegetation along with interval check dams for retainment, the lengths are very short, and extreme surface flow does not pass through them; therefore, stream erosion is not anticipated.

Furthermore, based on KRCA's Technical Guideline for SWM, Section 2.4, for areas larger than 2 ha, KRCA will require that the runoff from a 25 mm design storm (4 hours, Chicago distribution) be detained and released over a period of at least 24 hours. In this site, the total LID volume capacity (infiltration capacity) is equal to 5659 m^3 versus a site-wide volume control target of 5,519 m^3. Therefore, outlet speed will be reduced and related erosion. So, LID facilities can reduce the peak flow rate, delayed peak flow time, outflow volume, and rainfall outflow ratio of stormwater runoff in this project; and they can effectively collect rainwater and become a supplement to the landscape water system of the site.

10. Water Balance

Based on Burketon Mclaughlin Climate Data for Nonquon River Subwatershed, precipitation is 898mm/year, and evapotranspiration is mostly crops and trees in hydrologic soil group B/D, 567mm/year (Appendix X).

Pre-development site conditions have low groundwater infiltration/recharge, as the site is primarily clay soil with slopes <2% grade and a large part of the runoff flows into the tributary of the Nonquon River through the northwest, southwest and northeast waterways/ditches. On a 52.4 ha with 0.1% TIMP, there is an estimated 93,072m^3 of groundwater recharge per year. Runoff is 153 mm/year = 80,433 m^3/yr.

Pre-development condition parameters are shown in Table 11, below, as well as detailed in Section 3 of this report. The site is within the Nonquon River Subwatershed, with clay and loam Hydrologic Soil Group B/D, 0.1% TIMP, containing 33.3 ha (63.6%) agricultural land, 2.04 ha (3.9%) of wetlands and 17.05 ha (32.5%) of woodland and hedgerow.

The proposed post-development TIMP is 22.1 ha / 42.2% and, without any LID treatments, the site recharge is estimated at an equivalent of 116mm/year, or 60,797 m^3 (-34.7% as compared to pre-development).

The proposed development includes 22.1 ha total impervious surface area (TIMP), 42.2% of the total drainage area. Agricultural lands are reduced through the development of the property, which causes a reduction in evapotranspiration of 38.8% from pre-development levels. Additionally, it causes an 80-85% (assumed) of the precipitation surplus within the TIMP area of industrial blocks which is managed for recharge and infiltration (+117.9%) but the outflow from asphalt roadway would be discharged to wetlands or creeks (10,190 m^3 which is equivalent to 12.7% of total runoff in pre-development scenario or 13.8% of total runoff in post-development with mitigation). The results show that the amount of external outflow has increased compared to the pre-development state (153mm to 435mm or ΔV= +135,634m^3). Proposed mitigation measures are discussed above LID 1 for developed industrial blocks and LID 1 for municipal roadway.

Post-development with mitigation, the total rate of infiltration has increased from 178 mm/year to 387 mm/year or 93,072 m^3 to 202,816 m^3 (+117.9%). Detailed calculations of water balance for each scenario are presented in Appendix X.

Table 11 –Water Balance Summary

Characterstic	Site				
	Pre-Development	Post-Development	Change (Pre- to Post-)	Post-Development with Mitigation	Change (Pre- to Post- with Mitigation)
Inputs (Volumes)					
Precipitaiton (m^3/yr)	470,717	470,717	0.0%	470,717	0.0%
Run-On (m^3/yr)	0	0	0.0%	0	0.0%
Other Inputs (m^3/yr)	0	0	0.0%	0	0.0%
Total Inputs (m^3/yr)	**470,717**	470,717	0.0%	470,717	0.0%
Outputs (Volumes)					
Precipitation Surplus (m^3/yr)	173,505	288,752	66.4%	288,752	66.4%
Net Surplus (m^3/yr)	173,505	288,752	66.4%	288,752	66.4%
Evapotranspiratin (m^3/yr)	297,212	181,965	-38.8%	181,965	-38.8%
Infiltration (m^3/yr)	93,072	60,797	-34.7%	154,054	65.5%
Rooftop Infiltration (m^3/yr)	0	0	0.0%	48,762	0.0%
Total Infiltration (m^3/yr)	93,072	60,797	-34.7%	202,816	117.9%
Runoff Pervious Area (m^3/yr)	80,433	37,712	-53.1%	37,712	-53.1%
Runoff Impervious Area (m^3/yr)	0	178,355	0.0%	36,335	0.0%
Total Runoff (m^3/yr)	80,433	216,067	168.6%	74,048	-7.9%
Total Outputs (m^3/yr)	470,717	470,717	0.0%	470,717	0.0%

11. Erosion and Sediment Control During Construction

In general, the guiding principles of the ESC Plan are according to the KRCA

1. Minimizing soil erosion at the source;
2. Containing sediment on site;
3. Treating sediment-laden runoff; and
4. Being proactive, not reactive.

1.1. Minimizing soil erosion at the source

Based on the proposed developments, the site needs to replace the clay material with crushed stone/gravel to construct infiltration facilities in each block. Since earthworks generally cause significant erosion, the main area of concern for ESC is the construction of the new proposed infiltration trenches for bioretention/bioswales. This area will be used as a catchment infiltration basin due to its large footprint area and high permeability materials replaced. All temporary grading will have slopes directed into the excavated area. This type of basin can store water as it runs off or is generated from upstream land. While this water is being held or ponded or

infiltrated, either managing surface water and solids and some contaminants can be settled out of the water column. Water with less sediment in it is less erosive.

1.2. Containing sediment on site
Where sediments are not directed into the excavation area of each block, it is a requirement to install silt fences as per the Township of Scugog Sediment Control Fence SS-351 along with pinned silt socks at the site boundary with surrounding wetlands and woodlands.

1.3. Treating sediment-laden runoff
The proposed cut & fill plan is significant and requires careful management to prevent sediment-laden storm runoff from being conveyed off-site and into nearby wetlands and streams. Based on the low-infiltration rate of the site (clay soil & infiltration rate = 50 min/cm), it is estimated that part of the runoff will leave the site, however, the underground LID space for detaining runoff is larger than the calculated 100-year rainfall. Assuming that silt fences/socks are adequately installed and maintained, any temporary ponding of sediment-laden runoff will be infiltrated into the natural soils and runoff outflow from ditches will be captured by a series of straw bale check dams to slow sediment-laden water and allow sediment to settle before flowing downstream.

1.4. Being proactive, not reactive (Best Management Engineering Practice)
The project proposes cut & fill operations. Topsoil stripping of the fill location must be prepared prior to cut operations. This allows the fill portion to occur as soon as feasible, with minimal stockpiling. Backfilled areas are required to be seeded as soon as possible.

This project can be separated into four (4) separate phases as follows:

Phase 1 – Site Preparations: Driveway construction and the preparation for future cut & fill operations requiring topsoil stripping. Silt fences along with silt socks are installed in this phase. All topsoil shall be stockpiled at the south side of the property, behind the silt fences, to act as a loose permeable filtration mound. To reduce tracking of sediment to the municipal roads, temporary tracking pads like mud mat (Township of Scugog Stone Pad Construction Entrance SS-354) is required at the site entrance. Local on-site CB/MH also needs to have filter fabric/sediment catchers that remove sediment.

Phase 2 – Cut & Fill Operations: Once site preparation is completed, the actual cut & fill operation within each block can begin. The native clay soil will be replaced in each block to construct new bioretention LIDs in this phase (5.5% footprint area for bioretention in each block). It is a requirement that all excavated materials are to be trucked off-site immediately, as there does not seem to be sufficient surface area for local backfill & grading. At the edges of the excavation void/cut area, all surface grading must slope into the excavation void, such that all stormwater and drainage are captured by the excavation itself. Stripped topsoil needs to be transported and stockpiled at the south property area, while excess soils can be locally used as backfill or as a base layer for driveway construction. Emergency spill kits are required to be available at the site at all times.

Phase 3 –Main construction: The proposed buildings and warehouses for each block will be constructed in this phase. Septic holding tanks will be installed and buried in this phase, along with other site services, such as propane tanks and buried electrical cables. All excavated soils must be fully backfilled, graded,

covered with topsoil, and reseeded as soon as possible.

Phase 4 – Final Grading & Auxiliary Structures: After the overall rear backfill is completed, the physical construction may be completed, as well as keeping a small amount of soil for final grading. Auxiliary structures will be constructed at this time; paving parking lots, interior roads and roadways, planting, and landscaping. Previously stockpiled topsoil would be mechanically processed and then used to cover all exposed soils.

12. Reliance & Signature

This report is the intellectual property of King EPCM, and has been prepared for the sole use of Wanlong Real Estate Development Ltd. (Property Owner and Client). King EPCM accepts no liability for claims arising from the use of this report, or from actions taken or decisions made as a result of this report, by parties other than the Client. The Client may submit this report to the Township of Scugog (Port Perry), Kawartha Region Conservation Authority (KRCA), and Regional Municipality of Durham (Durham Region) in regards to the Client's industrial development project at 0 Scugog Line 6, Port Perry, ON.

Respectfully,



Amir Samadi, PhD, EIT
Senior Engineer – Water Resources
King EPCM

Supervised and reviewed by:



Yu Tao (Tony) Wang, P. Eng
Principal Engineer
King EPCM



13. References

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APPENDIX I – SITE SURVEY PLAN

SURVEYOR'S REAL PROPERTY REPORT
PART 1 – PLAN OF
PART OF LOT 14
CONCESSION 6
GEOGRAPHIC TOWNSHIP OF REACH, COUNTY OF ONTARIO
NOW IN THE
TOWNSHIP OF SCUGOG
REGIONAL MUNICIPALITY OF DURHAM

SCALE 1:1500

75m 37.5 0 75 METRES

MANDARIN SURVEYORS LIMITED, O.L.S. ©

METRIC

DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048

LEGEND

□	DENOTES MONUMENT SET
■	DENOTES MONUMENT FOUND
SIB	DENOTES STANDARD IRON BAR
IB	DENOTES IRON BAR
SSIB	DENOTES SHORT STANDARD IRON BAR
P1	DENOTES PLAN 40R–18803
P2	DENOTES PLAN 40R–27161
P3	DENOTES PLAN 40R–10590
P4	DENOTES PLAN 40R–18167
P5	DENOTES PLAN 40R–12137
CAL'D	DENOTES CALCULATED FROM P1 AND P2
WT	DENOTES WITNESS
PIN	DENOTES PROPERTY IDENTIFIER NUMBER
M	DENOTES MEASURED
N,S,E,W	DENOTES NORTH, SOUTH, EAST, WEST
W.F.	DENOTES WIRE FENCE
C.S.P.	DENOTES CORRUGATED STEEL PIPE
INV.	DENOTES INVERT ELEVATION
1183	DENOTES H. F. GRANDER Co. LTD., O.L.S.
1174	DENOTES B.T. DAVIES, O.L.S.
NI	DENOTES NOT IDENTIFIABLE
D.S.	DENOTES FINISHED SILL ELEVATION AT ENTRY
O.H.W.	DENOTES OVERHEAD WIRE
T.O.B.	DENOTES TOP OF BANK
H.T.	DENOTES HYDRO TRANSFORMER
M.H.	DENOTES MANHOLE
F.H.	DENOTES FIRE HYDRANT
P.A.	DENOTES POLE ANCHOR
U.P.	DENOTES UTILITY POLE
U.P.L.S.	DENOTES UTILITY POLE WITH LIGHT STANDARD
W.V.	DENOTES WATER VALVE
C.B.	DENOTES CATCH BASIN
	DENOTES CONIFEROUS TREE
	DENOTES DECIDUOUS TREE

ALL TIES TO CONCRETE FOUNDATION, UNLESS NOTED OTHERWISE.

BEARINGS ARE UTM GRID, DERIVED FROM OBSERVED REFERENCE POINTS, BY REAL TIME NETWORK OBSERVATIONS, UTM ZONE 17, NAD 83.

BENCHMARK NOTE

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO THE TOWNSHIP OF SCUGOG BENCHMARK No.386–71 HAVING AN ELEVATION OF 259.346 M. DATUM: CGVD28:78. LOCATED AT TWO–STOREY RED BRICK HOUSE ON WEST SIDE OF HWY 7 AND 12, 1.9 KM NORTH OF JCT OF HWY 7 AND 12 AND HWY 7A AT MANCHESTER, 0.5 KM NORTH OF SCUGOG 6TH LINE AND 29.6M WEST OF CENTRELINE OF HWY 7 AND 12. TABLET IS SET HORIZONTALLY IN NORTH FACE OF STONE FOUNDATION, 1.40 M WEST OF NORTHEAST CORNER AND 43 CM BELOW BRICKWORK.

THIS REPORT WAS PREPARED FOR WANLONG REAL ESTATE DEVELOPMENT LTD. AND THE UNDERSIGNED ACCEPTS NO RESPONSIBILITY FOR ITS USE BY OTHER PARTIES.

PART 2 (SURVEY REPORT)

1. REGISTERED EASEMENTS AND/OR RIGHT OF WAYS: TOGETHER WITH AN EASEMENT AS IN LT936579. SUBJECT TO AN EASEMENT IN FAVOR OF ONTARIO HYDRO AS SET OUT IN INST. No. RH17533 AND RH17659.
2. ADDITIONAL COMMENTS: NOTE THE LOCATION OF THE FENCES AROUND THE WESTERLY LIMITS OF THE SUBJECT PROPERTY. NOTE ALSO THE LOCATION OF THE OVERHEAD WIRE AND POLE ANCHORS.
3. THIS PLAN DOES NOT CERTIFY COMPLIANCE WITH ZONING BY–LAWS.

SURVEYOR'S CERTIFICATE

I CERTIFY THAT:

1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT, THE SURVEYORS ACT AND THE REGULATIONS MADE UNDER THEM.
2. THE SURVEY WAS COMPLETED ON THE 08th DAY OF OCTOBER , 2021

NOVEMBER 1 , 2021
DATE

Z. ZENG
ONTARIO LAND SURVEYOR

MANDARIN SURVEYORS LIMITED
ONTARIO LAND SURVEYOR CANADA LANDS SURVEYOR
WWW.MANDARINSURVEYOR.COM
2400 MIDLAND AVE., #121 PHONE: (647)430–1366 FAX: (647)799–4068
SCARBOROUGH, ONTARIO, M1S 1X7 E–MAIL: MANDARINSURVEYOR@GMAIL.COM

SURVEY BY: S.Z. | CAD No: 21–491SRPR | JOB No: 2021–491

ASSOCIATION OF ONTARIO LAND SURVEYORS
PLAN SUBMISSION FORM
2180421
THIS PLAN IS NOT VALID UNLESS IT IS AN EMBOSSED ORIGINAL COPY ISSUED BY THE SURVEYOR.
In accordance with Regulation 1026, Section 29(3).

DETAIL "A" (NOT TO SCALE)

DURHAM ROAD No. 8

NORTH PORT ROAD

CONCESSION 6

TAYLOR BLVD

EASY STREET (BY RP 40M–2100)

REACH INDUSTRIAL PARK

SCUGOG LINE (ROAD ALLOWANCE BETWEEN CONCESSIONS 5 AND 6) (BY TOWNSHIP OF SCUGOG BY–LAW No. 66–90, INST. No. D386197)

LOT 14 CONCESSION 6

LOT 15, CONCESSION 6

PART 1 PLAN 40R – 27614

PART 1 PLAN 40R – 18803

PIN 26805–0606(LT)

PIN 26805–0611

PIN 26805–0287

No.15251 VACANT LAND

No.1401 SCUGOG LINE 6 2 STOREY CONCRETE/GLASS BUILDING F.F.E.=268.73

No.1 EASY STREET 2 STOREY BRICK BUILDING

GARBAGE BIN

POND

CREEK

SWAMP

TREE AREA

DRIPLINE WAS MEASURED ON OCTOBER 08, 2021

CENTER OF CREEK

CENTER OF DITCH

EDGE OF PAVEMENT

SUBJECT TO EASEMENT IN FAVOR OF ONTARIO HYDRO AS SET OUT IN INST. No. RH17521

SUBJECT TO EASEMENT IN FAVOR OF ONTARIO HYDRO AS SET OUT IN INST. No. RH17519

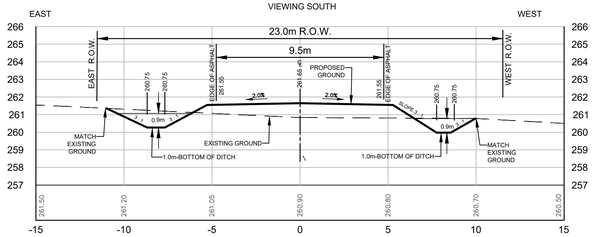
SUBJECT TO EASEMENT IN FAVOR OF ONTARIO HYDRO AS SET OUT IN INST. No. RH17533 AND RH17659

SUBJECT TO EASEMENT AS IN D279360

APPENDIX II – SITE GRADING PLAN

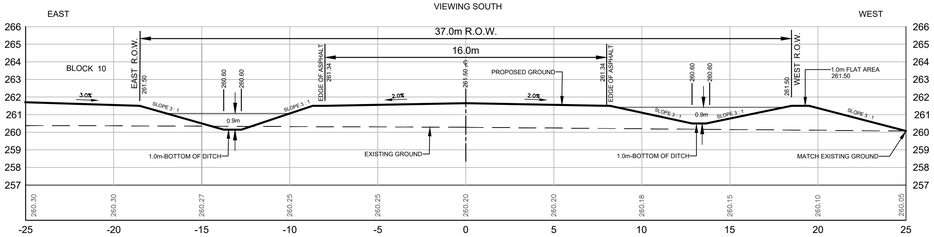
CROSS SECTION A - A' (TYPICAL)

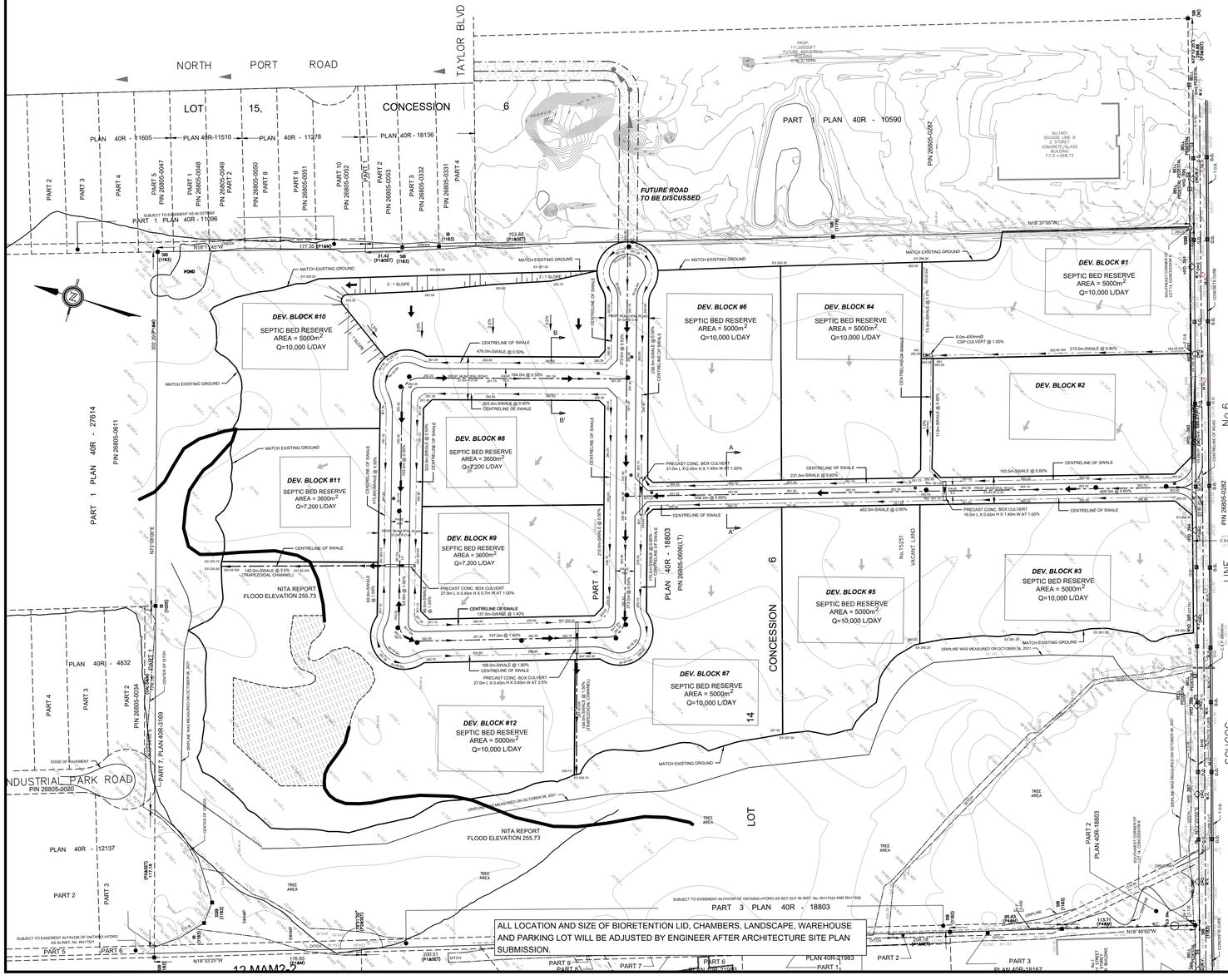
SCALE: N.T.S.
VIEWING SOUTH



CROSS SECTION B - B' (TYPICAL)

SCALE: N.T.S.
VIEWING SOUTH





LEGEND:

- x 315.58 — EXISTING ELEVATION
- x315.58 — PROPOSED ELEVATION
- x 315.58 INV — EXISTING PIPE INVERT ELEVATION
- x 315.58 INV — PROPOSED PIPE INVERT ELEVATION
- EXISTING SURFACE FLOW ROUTE
- PROPOSED SURFACE FLOW ROUTE
- x 315.00 — EXISTING ELEVATION CONTOUR LINES
- -2.0% — SWALE DRAINAGE
- PROPOSED SANITARY MANHOLE
- BOUNDARY LINE

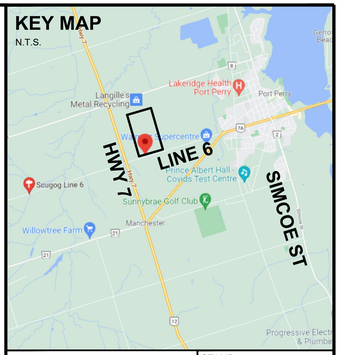
GENERAL NOTES:

1. CONTRACTOR TO BE RESPONSIBLE TO ENSURE THAT THIS LOT AND ADJOINING AREAS OF SITE ARE KEPT CLEAN AND FREE OF CONSTRUCTION DEBRIS, AND THAT ROADWAYS ARE KEPT CLEAN OF MUD AND DEBRIS
2. SEE "EROSION AND SEDIMENT CONTROL PLAN" FOR CONTROL INSTALLATION PRIOR TO CONSTRUCTION AND STOCKPILE MANAGEMENT
3. ALL MEASUREMENTS STATED IN METERS, PIPE SIZES IN MILLIMETERS, UNLESS OTHERWISE SPECIFIED EQUIVALENTS
4. GRASS SWALES SHALL BE CONSTRUCTED WITH 3:1 SIDE SLOPES, WITH AN AVERAGE DEPTH OF 600MM BELOW DRIVEWAY. SWALES SHALL HAVE A MINIMUM OF 50MM OF TOPSOIL, THEN SEEDED WITH LAWN GRASS DURING MAY - SEPTEMBER, WATERED WEEKLY UNTIL FULLY ROOTED, DETAILS REFER TO EGR 1.5
5. CONTRACTOR SHALL CHECK AND VERIFY ALL GIVEN GRADE ELEVATIONS AND DRAINAGE PRIOR TO CONSTRUCTION
6. MAIN PROPERTY SURVEY ELEVATIONS ARE DERIVED FROM "MANDARIN SURVEYORS LIMITED" ON NOV. 01 2021; ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE REFERRED TO THE TOWNSHIP OF SCUGOG BENCHMARK No.386-71 HAVING AN ELEVATION OF 259.346 M. DATUM: CGVD28:78. LOCATED AT TWO-STOREY RED BRICK HOUSE ON WEST SIDE OF HWY 7 AND 12, 1.9 KM NORTH OF JCT OF HWY 7 AND 12 AND HWY 7A AT MANCHESTER, 0.5 KM NORTH OF SCUGOG 6TH LINE AND 29.6M WEST OF CENTRELINE OF HWY 7 AND 12. TABLET IS SET HORIZONTALLY IN NORTH FACE OF STONE FOUNDATION, 1.40 M WEST OF NORTHEAST CORNER AND 43 CM BELOW BRICKWORK. 1401 SCUGOG LINE 6 ELEVATIONS ARE DERIVED FROM "A.R.(SANDY) WAKELING SURVEYING TECHNICAL SERVICES" ON AUG. 17, 2021; ELEVATIONS SHOWN HERE ON ARE GEODETIC IN ORIGIN AND ARE REFERENCED TO GEODETIC BENCH MARK NUMBER 00819828097 CGVD 28:78 ELEVATION=269.263 COORDINATES HAVE BEEN DERIVED FROM GPS OBSERVATIONS ON THE POWER NET GPS REAL TIME NETWORK, UTM ZONE 17, NAD83 (CSRS), EPOCH (2010).
10. DRIVEWAY SHALL BE CONSTRUCTED USING THE FOLLOWING:
    - 100MM HL8 BASECOURSE ASPHALT
    - 50MM HL3 SURFACE COURSE ASPHALT
    - 450MM GRANULAR "B" AND 150MM GRANULAR "A"
    - GRANULAR SHALL BE 98% COMPACTION

SCALE 1:2000
1cm = 20m
0m 100m 200m

KEY MAP

N.T.S.



DRAWN: KL

STAMP:

DATE: JAN 18, 2024

KING EPCM
3780 14th Ave., Unit 211
Markham, ON, L3R 9Y5
www.KingEPCM.com
647-459-5647

CLIENT

WANLONG REAL ESTATE DEVELOPMENT LTD.

PROJECT NAME

INDUSTRY DEVELOPMENT BLOCKS LAND PLAN & SITE SERVICE

PROJECT LOCATION

SCUGOG LINE 6, PORT PERRY, ON PART-1, PLAN 40R - 18803

PRINT TITLE

ROAD GRADING PLAN

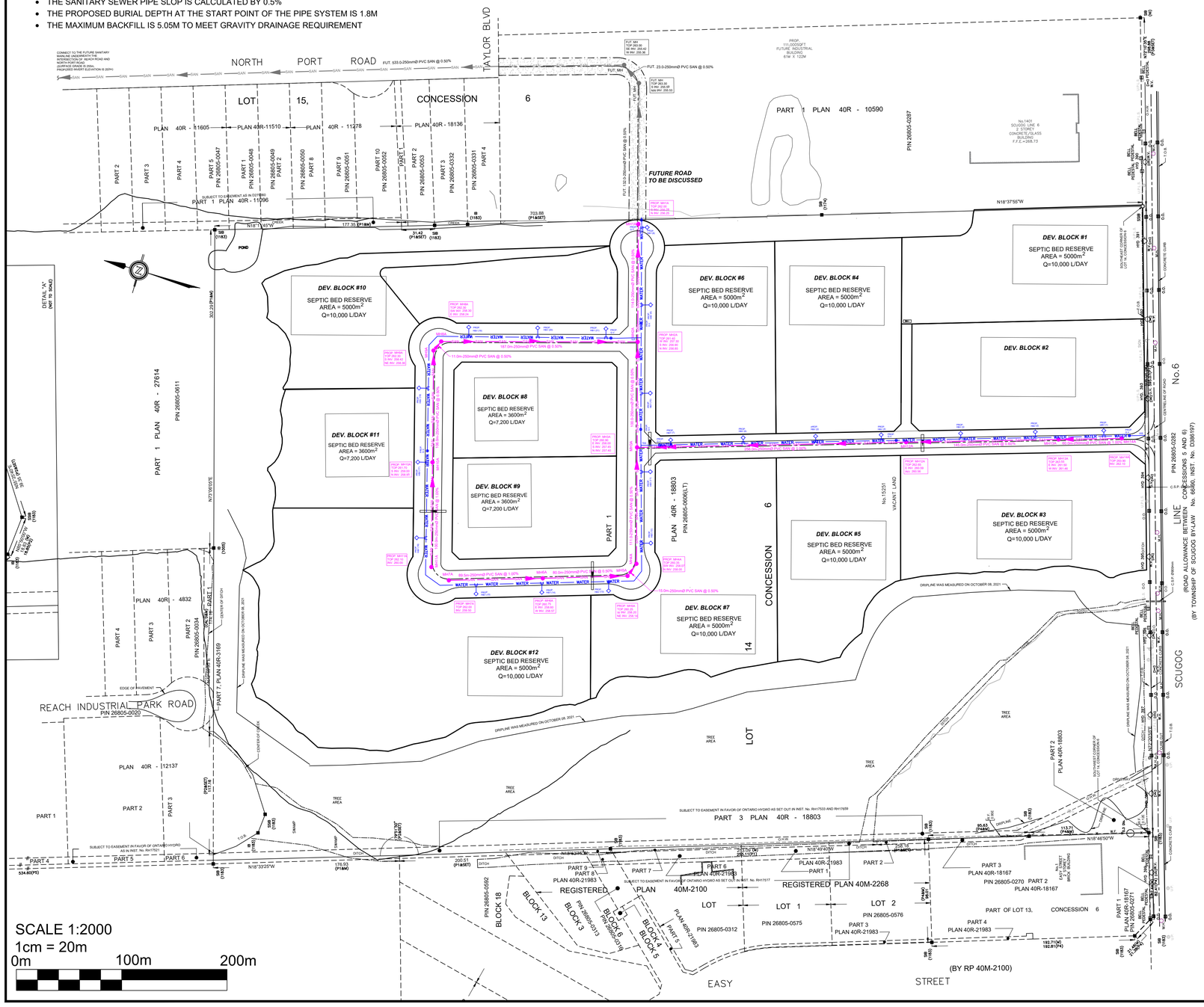
FILE No.

EGR 1.2

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V1	ISSUED FOR PERMITS	AUG. 25, 2022	ZW	TW
V20	ISSUED FOR PERMITS	JAN 18, 2024	KL	TW

GENERAL NOTES

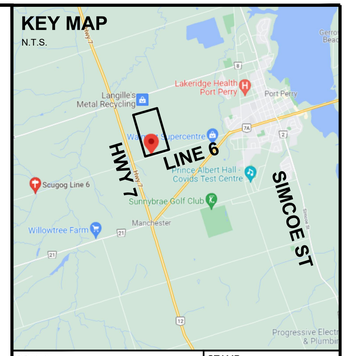
- THE SANITARY SEWER OUTLET CONNECTS TO THE FUTURE SEWER MAINLINE UNDERNEATH THE INTERSECTION OF REACH ROAD AND NORTH PORT ROAD
- THE BURIAL DEPTH OF THE OUTLET PIPE IS ASSUMED AS 3.0M
- THE SANITARY SEWER PIPE SLOP IS CALCULATED BY 0.5%
- THE PROPOSED BURIAL DEPTH AT THE START POINT OF THE PIPE SYSTEM IS 1.8M
- THE MAXIMUM BACKFILL IS 5.05M TO MEET GRAVITY DRAINAGE REQUIREMENT



LEGEND:

- PROPOSED SANITARY MANHOLE
- FUTURE SANITARY MANHOLE
- PROPOSED HYDRANT
- G.V. PROPOSED VALVE AND BOX
- SAN — SAN — PROPOSED SANITARY SEWER
- SAN — SAN — FUTURE SANITARY SEWER
- WATER — WATER — PROPOSED WATERMAIN
- INV. 315.58 PIPE INVERT ELEVATION
- BOUNDARY LINE

KEY MAP
N.T.S.



DRAWN: K.L

STAMP:

DATE: JAN 18, 2024

KING EPCM
3780 14th Ave., Unit 211
Markham, ON, L3R 9Y5
www.KingEPCM.com
647-459-5647

CLIENT

WANLONG REAL ESTATE DEVELOPMENT LTD.

PROJECT NAME

INDUSTRY DEVELOPMENT BLOCKS LAND PLAN & SITE SERVICE

(ROAD ALLOWANCE BETWEEN CONCESSIONS 5 AND 6) (BY TOWNSHIP OF SCUGOG BYLAW No. 66480, INST. No. D380197)

PROJECT LOCATION

SCUGOG LINE 6, PORT PERRY, ON PART-1, PLAN 40R - 18803

PRINT TITLE

SITE SERVICE PLAN SANITARY SEWER

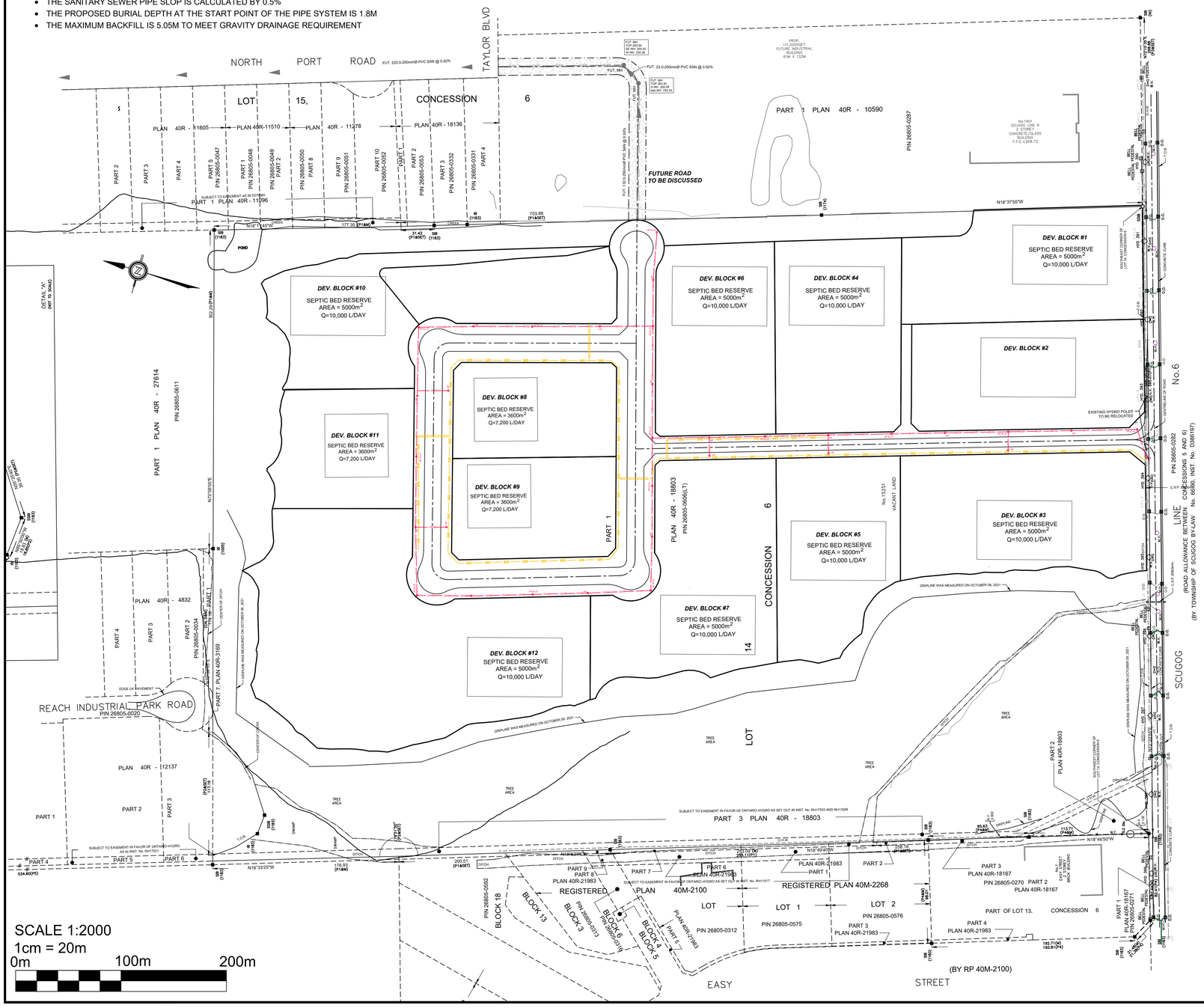
FILE No.

EGR 1.3

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V19	ISSUED FOR PERMITS	JUL. 10, 2023	QC	TW
V20	ISSUED FOR PERMITS	JAN 18, 2024	KL	TW

GENERAL NOTES

- THE SANITARY SEWER OUTLET CONNECTS TO THE FUTURE SEWER MAINLINE UNDERNEATH THE INTERSECTION OF REACH ROAD AND NORTH PORT ROAD
- THE BURIAL DEPTH OF THE OUTLET PIPE IS ASSUMED AS 3.0M
- THE SANITARY SEWER PIPE SLOP IS CALCULATED BY 0.5%
- THE PROPOSED BURIAL DEPTH AT THE START POINT OF THE PIPE SYSTEM IS 1.8M
- THE MAXIMUM BACKFILL IS 5.05M TO MEET GRAVITY DRAINAGE REQUIREMENT

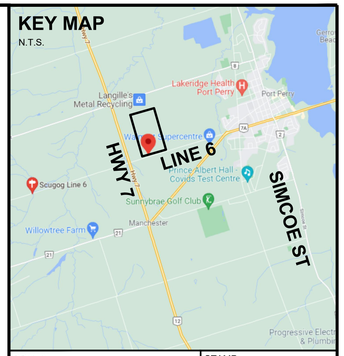


LEGEND:

HP & LS	PROPOSED HYDRO POLE WITH LIGHT STAND
U.P.L.S.	EXISTING HYDRO POLE WITH LIGHT STAND
GAS GAS	PROPOSED GASMAIN
O/H	EXISTING OVERHEAD WIRES
OHW OHW	PROPOSED ELECTRICAL OVERHEAD WIRES
	BOUNDARY LINE

KEY MAP

N.T.S.



DRAWN: K.L

STAMP:

DATE: JAN 18, 2024

KING EPCM

3780 14th Ave., Unit 211
Markham, ON, L3R 9Y5
www.KingEPCM.com
647-459-5647

CLIENT

WANLONG REAL ESTATE DEVELOPMENT LTD.

PROJECT NAME

INDUSTRY DEVELOPMENT BLOCKS LAND PLAN & SITE SERVICE

PROJECT LOCATION

SCUGOG LINE 6, PORT PERRY, ON PART-1, PLAN 40R - 18803

PRINT TITLE

UTILITY COORDINATION PLAN

FILE No.

EGR 1.4

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V19	ISSUED FOR PERMITS	JUL. 10, 2023	QC	TW
V20	ISSUED FOR COORDINATION	JAN 18, 2024	KL	TW

STORMWATER MANAGEMENT:

- 4.9% OF THE AREA OF EACH DEVELOPMENT BLOCK AS AN INFILTRATION CHAMBER WITH ATTACHED CHARACTERISTICS (REFER TO DETAIL D), PREFERRED LOCATION AT HIGH ELEVATION AREA. ALL BUILDING RUNOFF MUST BE TRANSFERRED DIRECTLY TO UNDERGROUND CHAMBERS.
- CAPTURED RUNOFFS BY PARKING LOTS AND INTERIOR ROADS IN EACH BLOCK SHOULD FIRST ENTER LANDSCAPE BIORETENTION (35% OF LANDSCAPE FOOTPRINT), PREFERRED LOCATION AT LOW ELEVATION AREA, AND THEN BE OVERFLOWED TO INFILTRATION CHAMBERS. OVERFLOW FROM THE CHAMBERS MUST ALSO RE-ENTER THE ROADSIDE BIOSWALES SUB-DRAIN AND BE DIVERTED TO EXTERNAL WATERWAYS.
- ROADSIDE BIOSWALES BE AVE. 600MM DEPTH, MIN. 400MM AND MAX. 900MM DEPTH, SIDE SLOPE WITH 1V: 3H, 800MM WIDTH FLAT BOTTOM. SUB-DRAIN PERFORATED PIPE SHALL BE INSTALLED.
- ALL EXCAVATION ACTIVITY SHALL BE HIGHER THAN UNDERGROUND WATER LEVEL, INCLUDING SUB-DRAIN PIPE. ALL STORMWATER MANAGEMENT INCLUDING SUB-DRAIN IS CONSIDERED ONLY FOR IMPERMEABLE AREA AND GROUND SURFACE, WITHOUT UNDERGROUND WATER DE-WATERING.

LEGEND:

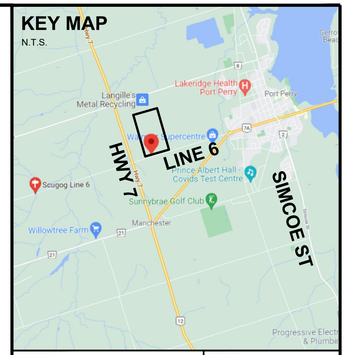
- EXISTING SURFACE FLOW ROUTE
- PROPOSED SURFACE FLOW ROUTE
- 315.00 el — EXISTING ELEVATION CONTOUR LINES
- -2.0% — SWALE DRAINAGE
- 2.55HA / 0.69 — DRAINAGE AREA / RUNOFF COEFFICIENT

SCALE 1:2000
1cm = 20m
0m 100m 200m

ROAD GRADING & SITE SERVICE REFER TO EGR 1.2.
ROAD CUT & BACKFILL VOLUME CALCULATION REFER TO EGR 1.4.

ALL LOCATION AND SIZE OF BIORETENTION LID, CHAMBERS, LANDSCAPE, WAREHOUSE AND PARKING LOT WILL BE ADJUSTED BY ENGINEER AFTER ARCHITECTURE SITE PLAN SUBMISSION.

KEY MAP
N.T.S.



DRAWN: KL
STAMP:
DATE: JAN 18, 2024

KING EPCM
3780 14th Ave., Unit 211
Markham, ON, L3R 9Y5
www.KingEPCM.com
647-459-5647

CLIENT

WANLONG REAL ESTATE DEVELOPMENT LTD.

PROJECT NAME

INDUSTRY DEVELOPMENT BLOCKS LAND PLAN & SITE SERVICE

PROJECT LOCATION

SCUGOG LINE 6, PORT PERRY, ON PART-1, PLAN 40R - 18803

PRINT TITLE

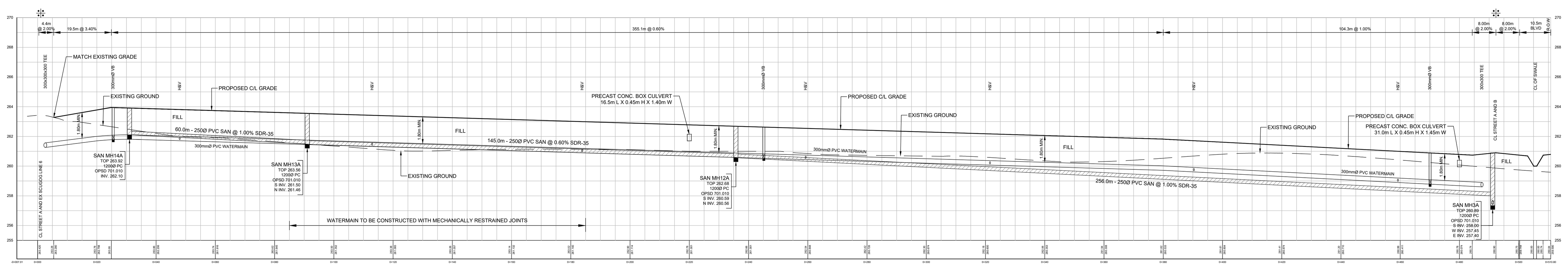
STORM DRAINAGE PLAN

FILE No.

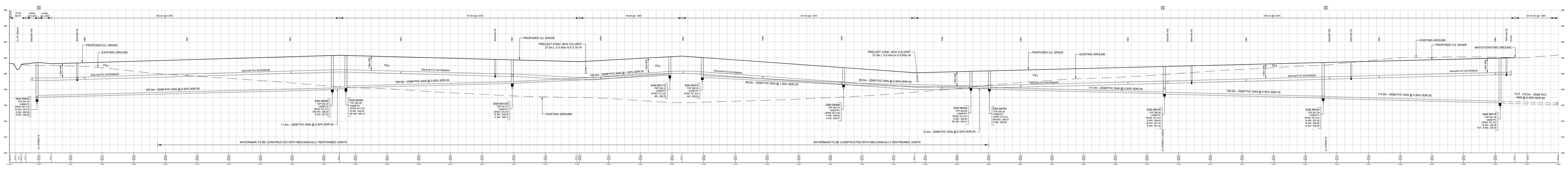
EGR 1.4

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V1	ISSUED FOR PERMITS	AUG. 25, 2022	ZW	TW
V20	ISSUED FOR PERMITS	JAN 18, 2024	KL	TW

STREET A PROFILE



STREET B PROFILE



APPENDIX III – BOREHOLE DRILL LOG

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GROUNDWATER MONITORING Well BH101

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/06/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3.1 m
DIAMETER 1.5 inch

COORDINATES 4884,244,899N, 661,934,627E
COORD SYS UTM-17
SURFACE ELEVATION 262.184 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Southeast corner of the dwelling

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			262
0.2		USCS: CL	Lean clay, damp to wet depths	Groundwater measured at 0.4 m deep when drilling		261.8
0.4						261.6
0.6						261.4
0.8						261.2
1						261
1.2						260.8
1.4						260.6
1.6						260.4
1.8						260.2
2						260
2.2						259.8
2.4						259.6
2.6						259.4
2.8						259.2
3						259
			Termination Depth at: 3.1 m			

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GROUNDWATER MONITORING Well BH102

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/07/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 4 m
DIAMETER 1.5 inch

COORDINATES 4884,266,525N, 661,996,529E
COORD SYS UTM-17
SURFACE ELEVATION 262.627 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS South of the dwelling

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation BH101	Elevation (m)
0			Topsoil			262.6
0.2						262.4
0.4						262.2
0.6		USCS: CL	Brown damp clay			262
0.8						261.8
1						261.6
1.2						261.4
1.4						261.2
1.6						261
1.8						260.8
2						260.6
2.2						260.4
2.4						260.2
2.6						260
2.8						259.8
3						259.6
3.2						259.4
3.4						259.2
3.6		USCS: CL	Grey wet clay	Groundwater measured at 3.5 m deep when drilling		259
3.8						258.8
4			Termination Depth at: 4 m			258.6

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GROUNDWATER MONITORING Well BH103

PROJECT NUMBER -----	**DRILLING COMPANY** King EPCM	**COORDINATES** 4884,287,682N, 662,059,624E
PROJECT NAME Scugog Line 6, Port Perry	**DRILLER** Pincheng Zhao, Yung Chong	**COORD SYS** UTM-17
CLIENT Wanlong Real Estate Development Ltd.	**DRILL RIG**	**SURFACE ELEVATION** 263.463 AMSL
ADDRESS 0 Scugog Line 6, Port Perry	**DRILLING METHOD** Continuous Flight Auger	**WELL TOC** None
DRILLING DATE 12/07/2021	**TOTAL DEPTH** 3.1 m	**LOGGED BY** Pincheng Zhao
LICENCE NO. C-7691	**DIAMETER** 1.5 inch	**CHECKED BY** Tony Wang, P Eng, Principal Engineer

COMPLETION **CASING** 1.25 inch **SCREEN** 1.25 inch

COMMENTS Southwest of the dwelling

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			263.4
0.2						263.2
0.4						263
0.6		USCS: CL	Brown damp clay			262.8
0.8						262.6
1						262.4
1.2						262.2
1.4						262
1.6						261.8
1.8						261.6
2		USCS: CL	Grey wet clay			261.4
2.2				Groundwater measured at 2.2 m deep when drilling		261.2
2.4						261
2.6						260.8
2.8						260.6
3						260.4
			Termination Depth at: 3.1 m			

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GROUNDWATER MONITORING Well BH104

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry
DRILLING DATE 12/08/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 4 m
DIAMETER 1.5 inch

COORDINATES 4884,364,300N, 661,867,534E
COORD SYS UTM-17
SURFACE ELEVATION 260.898 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Southeast of the dwelling

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			260.8
0.2						260.6
0.4		USCS: CL	Brown damp clay			260.4
0.6						260.2
0.8						260
1						259.8
1.2						259.6
1.4						259.4
1.6		USCS: CL	Grey wet clay			259.2
1.8				Groundwater measured at 1.8 m deep when drilling		259
2						258.8
2.2						258.6
2.4						258.4
2.6						258.2
2.8						258
3						257.8
3.2						257.6
3.4						257.4
3.6						257.2
3.8						257
4			Termination Depth at: 4 m			256.8

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GROUNDWATER MONITORING Well BH105

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/09/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 2.7 m
DIAMETER 1.5 inch

COORDINATES 4884,404,509N, 661,950,440E
COORD SYS UTM-17
SURFACE ELEVATION 261.746 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Middle of the fieild

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			261.6
0.2		USCS: CL	Brown damp clay			261.4
0.4						261.2
0.6						261
0.8						260.8
1						260.6
1.2						260.4
1.4						260.2
1.6		USCS: CL	Grey wet clay			260
1.8						259.8
2						259.6
2.2				Groundwater measured at 2.2 m deep when drilling		259.4
2.4						259.2
2.6						259
2.8			Termination Depth at: 2.7 m	Bearing capacity tested at 2.7m deep is 1200 KPa at without displacement		258.8

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GROUNDWATER MONITORING Well BH106

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 1401 Scugog Line 6, Port Perry
DRILLING DATE 12/15/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 2.5 m
DIAMETER 1.5 inch

COORDINATES 4884,440,761N, 662,026,623E
COORD SYS UTM-17
SURFACE ELEVATION 262.448 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Middle of the fieild

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			262.4
0.2		USCS: CL	Grey damp clay			262.2
0.4						262
0.6						261.8
0.8						261.6
1						261.4
1.2						261.2
1.4						261
1.6		USCS: CL	Grey wet clay			260.8
1.8						260.6
2						260.4
2.2				Groundwater measured at 2.2 m deep when drilling		260.2
2.4						260
2.6			Termination Depth at: 2.5 m			259.8

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GROUNDWATER MONITORING Well BH107

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/08/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3.1 m
DIAMETER 1.5 inch

COORDINATES 4884,471,381N, 661,771,905E
COORD SYS UTM-17
SURFACE ELEVATION 260.092 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS East of the dwelling

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			260
0.2						259.8
0.4		USCS: CL	Brown damp clay			259.6
0.6						259.4
0.8						259.2
1						259
1.2						258.8
1.4						258.6
1.6						2584
1.8						258.2
2		USCS: CL	Grey wet clay			258
2.2				Groundwater measured at 2.2 m deep when drilling		257.8
2.4						257.6
2.6						257.4
2.8						257.2
3						257
			Termination Depth at: 3.1 m			

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GROUNDWATER MONITORING Well BH108

PROJECT NUMBER -----
PROJECT NAME Scugog line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/09/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3 m
DIAMETER 1.5 inch

COORDINATES 4884,508,262N, 661,897,120E
COORD SYS UTM-17
SURFACE ELEVATION 261.729 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Middle of the fieild

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			261.6
0.2						
0.4		USCS: CL	Grey damp clay			261.4
0.6						261.2
0.8						261
1						260.8
1.2		USCS: CL	Grey wet clay			260.6
1.4						260.4
1.6						260.2
1.8						260
2						259.8
2.2						259.6
2.4				Groundwater measured at 2.4 m deep when drilling		259.4
2.6						259.2
2.8						259
3			Termination Depth at: 3 m	Bearing capacity tested at 3 m deep is 1000 KPa at without displacement		258.8
3.2						258.6

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GROUNDWATER MONITORING Well BH109

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/09/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3 m
DIAMETER 1.5 inch

COORDINATES 4884,544,885N, 661,976,834E
COORD SYS UTM-17
SURFACE ELEVATION 261.704 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Middle & west of the fieild

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			261.6
0.2						
0.4		USCS: CL	Grey damp clay			261.4
0.6						261.2
0.8						261
1						260.8
1.2						260.6
1.4						260.4
1.6						260.2
1.8						260
2		USCS: CL	Grey wet clay			259.8
2.2						259.6
2.4						259.4
2.6						259.2
2.8				Groundwater measured at 2.8 m deep when drilling		259
3			Termination Depth at: 3 m	Bearing capacity tested at 3m deep is 1200 KPa at 40mm displacement		258.8
						258.6

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GROUNDWATER MONITORING Well BH110

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanglong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/14/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3.1 m
DIAMETER 1.5 inch

COORDINATES 4884,788,529N, 661,612,830E
COORD SYS UTM-17
SURFACE ELEVATION 256.346 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Northeast corner of the field

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			256.2
0.2		USCS: ML	Dark-yellow sand			256
0.4						255.8
0.6						255.6
0.8						255.4
1		USCS: CL	Grey damp clay	Groundwater measured at 1 m deep when drilling		255.2
1.2						255
1.4						254.8
1.6						254.6
1.8						254.4
2		USCS: CL	Grey wet clay			254.2
2.2						254
2.4						253.8
2.6						253.6
2.8						253.4
3			Termination Depth at: 3m	Bearing capacity tested at 3m deep is 1000 KPa at less than 40cm displacement		

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GROUNDWATER MONITORING Well BH111

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry
DRILLING DATE 12/14/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3.1 m
DIAMETER 1.5 inch

COORDINATES 4884,841,025N, 661,720,391E
COORD SYS UTM-17
SURFACE ELEVATION 257.541 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS North and middle location of the field

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			257.4
0.2		USCS: ML	Dark-yellow sand			257.2
0.4						257
0.6						256.8
0.8		USCS: CL	Grey damp clay			256.6
1				Groundwater measured at 1 m deep when drilling		256.4
1.2						256.2
1.4						256
1.6		USCS: CL	Grey wet clay			255.8
1.8						255.6
2						255.4
2.2						255.2
2.4						255
2.6						254.8
2.8						254.6
3			Termination Depth at: 3m	Bearing capacity tested at 3m deep is 1600 KPa at no displacement		

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GROUNDWATER MONITORING Well BH112

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/14/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 3 m
DIAMETER 1.5 inch

COORDINATES 4884,893,893N, 661,842,834E
COORD SYS UTM-17
SURFACE ELEVATION 258.759 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS North and middle location of the field

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			258.6
0.2						
0.4		USCS: CL	Grey damp clay			258.4
0.6						258.2
0.8						258
1						257.8
1.2						257.6
1.4						257.4
1.6		USCS: CL	Grey wet clay	Groundwater measured at 1.5 m deep when drilling		257.2
1.8						257
2						256.8
2.2						256.6
2.4						256.4
2.6						256.2
2.8						256
3			Termination Depth at: 3m	Bearing capacity tested at 3m deep is 1700 KPa at no displacement		255.8

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GROUNDWATER MONITORING Well BH113

PROJECT NUMBER -----
PROJECT NAME Scugog Line 6, Port Perry
CLIENT Wanlong Real Estate Development Ltd.
ADDRESS 0 Scugog Line 6, Port Perry, ON
DRILLING DATE 12/13/2021
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Pincheng Zhao, Yung Chong
DRILL RIG
DRILLING METHOD Continuous Flight Auger
TOTAL DEPTH 2.1 m
DIAMETER 1.5 inch

COORDINATES 4884,926,436N, 661,327,919E
COORD SYS UTM-17
SURFACE ELEVATION 255.121 AMSL
WELL TOC None
LOGGED BY Pincheng Zhao
CHECKED BY Tony Wang, P Eng, Principal Engineer

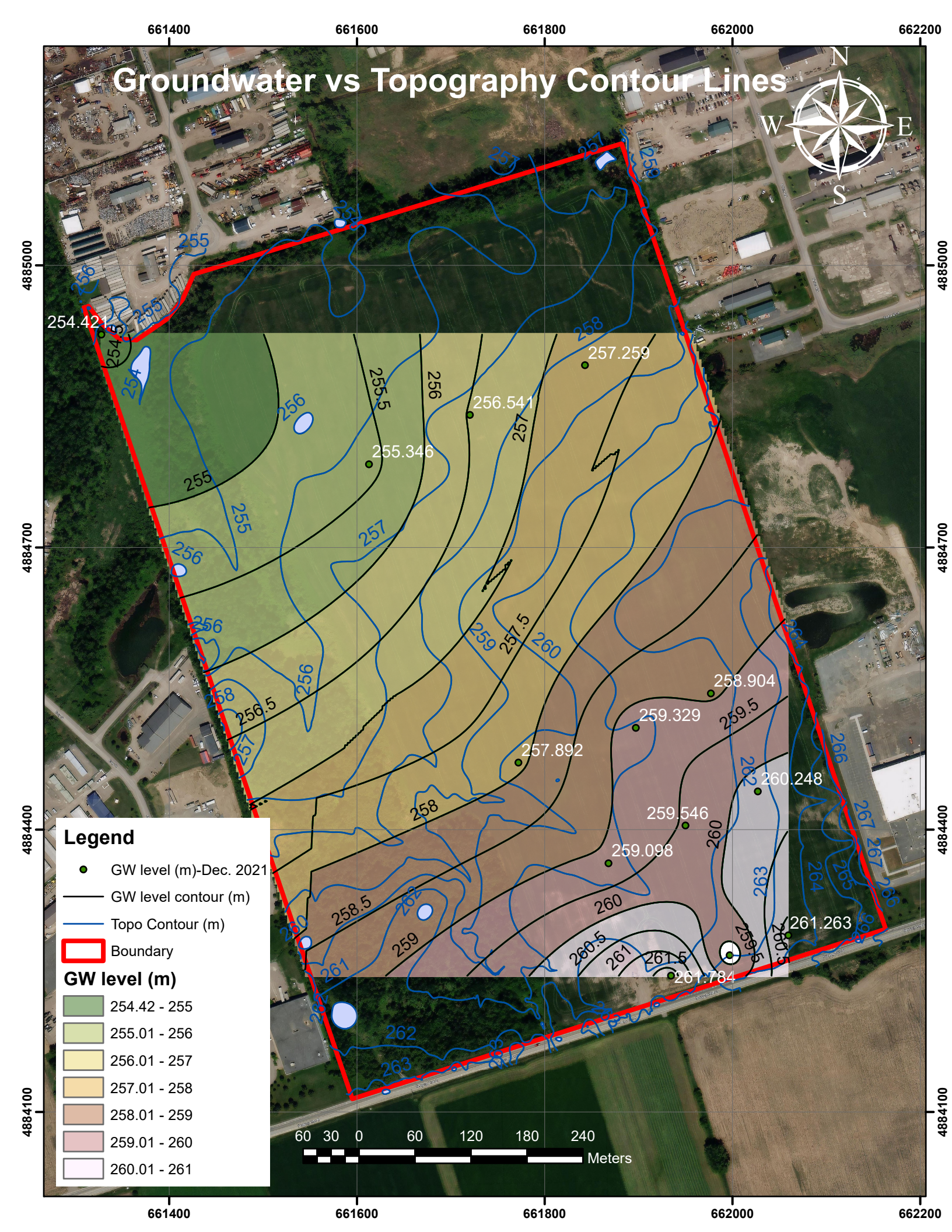
COMPLETION
CASING 1.25 inch
SCREEN 1.25 inch

COMMENTS Northeast of the dwelling near the swamp

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Well Installation	Elevation (m)
0			Topsoil			255.1
0.3		USCS: ML	Silt with low plasticity	Groundwater measured at 0.7 m deep when drilling		254.8
2.1			Termination Depth at: 2.1 m			253

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APPENDIX IV – GW vs TOPOGRAPHY


Groundwater vs Topography Contour Lines
N
W
E
S
661400
661600
661800
662000
662200
4885000
4884700
4884400
4884100
254.421
257.259
256.541
255.346
258.904
259.329
257.892
260.248
259.546
259.098
261.263
261.784
Legend
GW level (m)-Dec. 2021
GW level contour (m)
Topo Contour (m)
Boundary
GW level (m)
254.42 - 255
255.01 - 256
256.01 - 257
257.01 - 258
258.01 - 259
259.01 - 260
260.01 - 261
60 30 0 60 120 180 240
Meters

APPENDIX V – FIELD PERMEABILITY TEST

In-situ Measurement of Field Saturated Hydraulic Conductivity

1. Field Permeability Test

The "Constant Head Well Permeameter" (CHWP) method (Reynolds, 1993; Elrick and Reynolds, 1986) is based on the observation that when a constant height or "head" of water is ponded in a borehole or "well" augured into unsaturated soil, a "bulb" of field-saturated soil is gradually established around the base of the well. The K_{fs} value calculated through this method can be less than or equal to half of K_s (Saturated hydraulic conductivity) due to partial blocking of soil pores by air bubbles. K_{fs} is preferred over Ks for the design of on-site stormwater LID infiltration design, because drainage through the soil should be designed to occur at less than complete soil saturation.

The in-situ measurements were measured using the ETC Slow Soils Pask Permeameter, an extended single-head analysis method and calculation procedure based on the work of W.D. Reynolds and D.E. Elrick formerly of the University of Guelph, Ontario, Canada.

The ETC Pask Permeameter is a convenient and easy to use apparatus for ponding a constant head of water in a well, and simultaneously measuring the flow into the soil. The rate of fall (R) of the water level in the Permeameter reservoir and reservoir cross-sectional area (X) allows determination of quasi steady water flow rate (Q) into the soil (i.e Q = XR). K_{fs} is then calculated using Equation 1* (Reynolds, 1993):

$$K_{fs} = CQ / [2\pi H^2 + C\pi a^2 + (2\pi H/\alpha^*)] \qquad \text{(Eq. 1)}$$

In which:
K_{fs} = calculated permeability from the field test

* Note: Alternatively, Quick Field Reference Tables have been prepared by ETC for the specific Permeameter characteristics and typical well hole diameter (8.3cm) produced by the 7cm (2-3/4") AMS Riverside auger supplied with the kit. The tables give K_{fs} for various rates of fall (R) and $\boldsymbol{\alpha}^*$ values.

Table 1. Parameters used

Parameter	**Description**	**BH**			
		101	**106**	**107**	**112**
Soil Texture Factor (**α***) in cm^{-1}	Most structured and medium textured materials; including structured clayey and loamy soils, as well as unstructured medium single-grain sands (0.12) and compacted, structureless, clayey materials such as landfill caps and liners, lacustrine or marine sediments (0.01).	0.01	0.01	0.12	0.01
R in cm/min	Quasi steady state (constant) rate of fall of water in permeameter reservoir (measured in the site)	0.03	0.025	3.1	0.025
μ_k/μ_a	Temperature Correction Factor	0.942	0.974	0.942	0.942
C	Shape factor	1.27	1.27	1.36	1.27
X in cm^2	Cross-sectional area of Permeameter reservoir	12.80			
H in cm	Height of air inlet hole from bottom of the test hole	15			
a in cm	Well hole radius	4.15			

Based on data described in the above table and using Pask Permeameter ETC Quick Field Reference Tables for Standard Soils, the K_{fs} was calculated as:

K_{fs101}= 7.5E-9 m/sec = 7.5E-7 cm/sec
K_{fs106}= 6.2E-9 m/sec = 6.2E-7 cm/sec
K_{fs107}= 4.0E-6 m/sec = 4.0E-4 cm/sec
K_{fs112}= 6.2E-9 m/sec = 6.2E-7 cm/sec

It should be noted that the third test (K_{fs107}) has a higher permeability than the other tests due to the location of the well bottom in the sandy layer.

And then the temperature corrected permeability would be calculated using Equation 2 as follows:

$K_a = K_{fs} \times \mu_k/\mu_a$ (Eq. 2)

In which:
K_a = corrected permeability adjusted for design temperature conditions

Then using the temperature correction factor (for t = 4°c - 7°c) from the manual:
K_{a101} = 7.07E-9 m/sec = 7.07E-7 cm/sec

K_{a106} = 6.04E-9 m/sec = 6.04E-7 cm/sec
K_{a107} = 3.77E-6 m/sec = 3.77E-4 cm/sec
K_{a112} = 5.84E-9 m/sec = 5.84E-7 cm/sec

The field permeability data sheets are appended to this report.

2. Percolation time/infiltration rate for design (Reynolds et al., 2015)

Correlations between Perc Time (PT) and field-saturated hydraulic conductivity (K_{fs}) are often used in development of on-site water recycling and treatment facilities that operate by infiltration into unsaturated soil. The physically based PT versus K_{fs} expression in Reynolds et al. (2015) for cylindrical test holes in unsaturated soil can be simplified to Equation 3.

$$PT = \frac{\Delta t}{\Delta H} = mK_{fs}^{-1} \qquad \text{(Eq. 3)}$$

Where:

$$m = \frac{\bar{C}a^2}{\left[2\bar{H}^2 + \bar{C}a^2 + \frac{2\bar{H}[1-\exp(\alpha\,\psi_\alpha)]}{\alpha}\right]}; 0 < m < 1 \qquad \text{(Eq. 4)}$$

And *a* is test hole radius, $\bar{H}$ is average water level (ponding depth) in the test hole over time interval, Δt, ψα is the antecedent or background pore water pressure head in the soil surrounding the test hole, α may be viewed as the "integrally correct" slope of the soil's unsaturated hydraulic conductivity versus pore water pressure head relationship, K(ψ) and $\bar{C}$ is a "shape function" (Reynolds et al., 2015).

Conversion of CHWP K_{fs}/k_a to equivalent Perc Time, PT for this site using m= 5.39E-06 (moderate capillarity category) and m= 1.05E-06 (very strong category):

$$PT_{101} = mK_{a101}^{-1} = \frac{(1.05E-6)}{(7.07E-9)} = 148.6\ \text{min/cm}$$ (Infiltration Rate =4.04 mm/hour)

$$PT_{106} = mK_{a106}^{-1} = \frac{(1.05E-6)}{(6.04E-9)} = 173.9\ \text{min/cm}$$ (Infiltration Rate = 3.45 mm/hour)

$$PT_{107} = mK_{a107}^{-1} = \frac{(5.39E-6)}{(3.77E-6)} = 1.43\ \text{min/cm}$$ (Infiltration Rate = 419.4 mm/hour)

$$PT_{112} = mK_{a112}^{-1} = \frac{(1.05E-6)}{(5.84E-9)} = 179.8\ \text{min/cm}$$ (Infiltration Rate = 3.34 mm/hour)

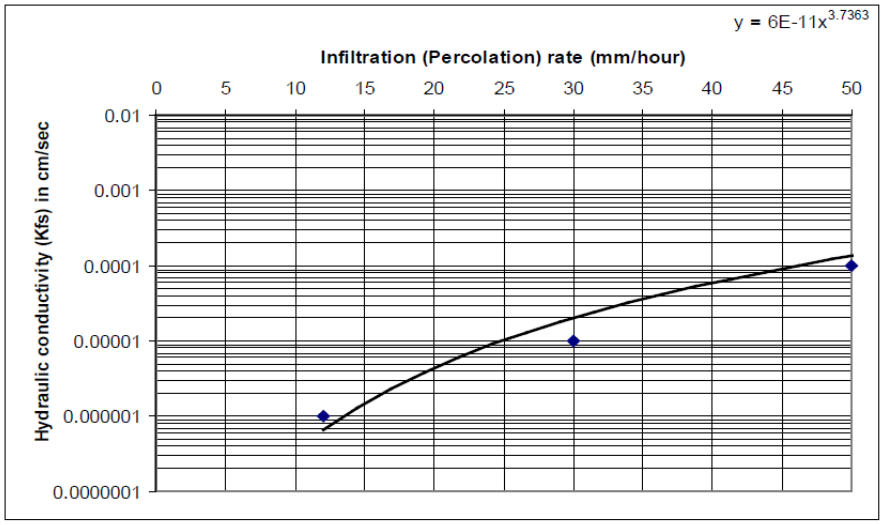
3. Percolation time/infiltration rate for design (OMMAH, 1997)

Despite the newer academic papers published by Reynolds et al. (2015), TRCA and other Conservation Authorities often still review design of infiltration basins based on historic trends. Below are two TRCA (2012) design criteria that describe the relationship between K_{fs}, PT, and infiltration rates, based on the 1997 (OMMAH) supplementary guidelines to OBC (1997).

Table 2. Approximate relationships between hydraulic conductivity, percolation time and infiltration rate

Hydraulic Conductivity, K_{fs} (centimetres/second)	Percolation Time, T (minutes/centimetre)	Infiltration Rate, 1/T (millimetres/hour)
0.1	2	300
0.01	4	150
0.001	8	75
0.0001	12	50
0.00001	20	30
0.000001	50	12

***Source**: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.*



***Source**: Ontario Ministry of Municipal Affairs and Housing (OMMAH). 1997. Supplementary Guidelines to the Ontario Building Code 1997. SG-6 Percolation Time and Soil Descriptions. Toronto, Ontario.*

Figure 1. Approximate relationship between infiltration rate and hydraulic conductivity

Based on OMMAH interpolation from Table 2 and Figure 1 above, the measured K_{fs} may be interpolated as:

PT_{101} = 48.8 min / cm (Infiltration Rate = 12.3 mm/hour)

PT_{106} = 50.8 min / cm (Infiltration Rate = 11.8 mm/hour)

PT_{107} = 9.1 min / cm (Infiltration Rate = 66 mm/hour)

PT_{112} = 51.3 min / cm (Infiltration Rate = 11.7 mm/hour)

Although the results for three slow soils (BH101, BH106 & BH112) are similar with a 5 percent difference, when comparing the OMMAH result with that obtained by Reynolds et al. (2015) formula (PT= 1.43-179.8 min/cm), the two methods of conversion are completely different results. It seems that the values of the first method have underestimated the infiltration rate and Perc Time. The Engineer has decided to use values obtained from the second method (OMMAH, 1997), as per the TRCA Stormwater Management Criteria Guideline. This method was proposed by TRCA to convert measured hydraulic conductivity to percolation time or infiltration rate. For the subject site an unfactored infiltration rate of 11.7 – 66 mm/hour was calculated.

The slower infiltration rates from PT_{101}, PT_{106}, and PT_{107} may be used as representative of the permeability of the site.

4. Factored Engineering Design Infiltration Rate (Wisconsin Department of Natural Resources, 2004)

For a conservative approach to infiltration speeds, the Wisconsin Department of Natural Resources (2004) method shall be used for the calculation of a factored design infiltration rate. The overall soil formation is compacted clay soils, except in the middle of the site (< 5%) which was loam (ML), with an unfactored infiltration rate = 66 mm/hour in contrast to the rest of the site, 11.9 mm/hour on average. However, the infiltration rate used to design an infiltration BMP must incorporate a safety correction factor that compensates for potential reductions in soil permeability due to compaction or smearing during construction, gradual accumulation of fine sediments over the lifespan of the BMP and uncertainty in measured values when less permeable soil horizons exist within 1.5 meters below the proposed bottom elevation of the BMP. As discussed above, the predominant soil material of this site is composed of clay (CL) to a depth of more than 3 meters, which has low permeability.

Based on borehole data, the soil layer remains consistent of CL soil types, including the soil layers 1.5 meters below the proposed bottom of the BMP, except in the middle of the site composed of ML. This means that based on Table 3, the measured infiltration rate should be divided by a safety correction factor to calculate the design infiltration rate. Thus the mean infiltration rate measured at the proposed bottom elevation of the BMP is 11.9 mm/hour, and the mean infiltration rate measured in the slowest underlying soil horizon is 4.8 mm/hour, and the ratio of infiltration rates is 2.5.

Table 3. Safety correction factors for calculating design infiltration rates

Ratio of Mean Measured Infiltration Rates[1]	Safety Correction Factor[2]
≤ 1	2.5
1.1 to 4.0	3.5
4.1 to 8.0	4.5
8.1 to 16.0	6.5
16.1 or greater	8.5

***Source**: Wisconsin Department of Natural Resources. 2004. Conservation Practice Standards. Site Evaluation for Stormwater Infiltration (1002). Madison, WI.*

***Notes**:*

1. *Ratio is determined by dividing the geometric mean measured infiltration rate at the proposed bottom elevation of the BMP by the geometric mean measured infiltration rate of the least permeable soil horizon within 1.5 metres below the proposed bottom elevation of the BMP.*
2. *The design infiltration rate is calculated by dividing the geometric mean measured infiltration rate at the proposed bottom elevation of the BMP by the safety correction factor.*

Field Permeability Test Sheet

Engineering Technologies Canada Ltd.

OWNER'S NAME:

SITE LOCATION: 1401 Scugog Line 6, Port Perry

PID #:

TEST PIT #: BH101

TECHNICIAN: Pincheng Zhao, Young Chong

DATE: 2021/12/13

WEATHER/TEMPERATURE: Sunny, 4-7℃

FIELD PERMEABILITY TEST #:

D – reservoir diameter (cm)	8.25	**Soil Texture**	Clay
d – well hole diameter (cm)	8.3	**Soil Structure**	Strongly Compacted
H – height of water in well (cm)	15	**α* (cm-1)**	0.01
Depth below ground surface (cm)	38	**C – Factor**	1.27

TIME (min)	(1) CHANGE IN TIME (min)	RESERVOIR WATER LEVEL (WL) (cm)	(2) CHANGE IN WL (cm)	(2) ÷ (1) RATE OF FALL (R) (cm/min)
0	0	36	0	
20	20	35.7	0.3	0.2
40	20	35	0.7	0.035
60	20	34.4	0.6	0.03
80	20	33.8	0.6	0.03
100	20	33.2	0.6	0.03
120	20	32.6	0.6	0.03

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Revision Date: May 2018

Engineering Technologies Canada Ltd.

OWNER'S NAME:

SITE LOCATION: 1401 Scugog Line 6, Port Perry

PID #:

TEST PIT #: BH106

TECHNICIAN: Pincheng Zhao, Young Chong

DATE: 2021/12/15

WEATHER/TEMPERATURE: Raining, 5°C

FIELD PERMEABILITY TEST #:

D – reservoir diameter (cm)	8.25	**Soil Texture**	Clay
d – well hole diameter (cm)	8.3	**Soil Structure**	Strongly Compacted
H – height of water in well (cm)	15	**α* (cm-1)**	0.01
Depth below ground surface (cm)	38	**C – Factor**	1.27

TIME (min)	(1) CHANGE IN TIME (min)	RESERVOIR WATER LEVEL (WL) (cm)	(2) CHANGE IN WL (cm)	(2) ÷ (1) RATE OF FALL (R) (cm/min)
0	0	36.5	0	
20	20	36	0.5	0.025
40	20	35.5	0.5	0.025
60	20	35	0.5	0.025
80	20	34.5	0.5	0.025
100	20	34	0.5	0.025

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Engineering Technologies Canada Ltd.

OWNER'S NAME:

SITE LOCATION: 1401 Scugog Line 6, Port Perry

PID #:

TEST PIT #: BH107

TECHNICIAN: Pincheng Zhao, Young Chong

DATE: 2021/12/13

WEATHER/TEMPERATURE: Sunny, 4-7℃

FIELD PERMEABILITY TEST #:

D – reservoir diameter (cm)	8.25	**Soil Texture**	Loam
d – well hole diameter (cm)	8.3	**Soil Structure**	Moderate Blocky
H – height of water in well (cm)	15	**α* (cm-1)**	0.12
Depth below ground surface (cm)	50	**C – Factor**	1.36

TIME (min)	(1) CHANGE IN TIME (min)	RESERVOIR WATER LEVEL (WL) (cm)	(2) CHANGE IN WL (cm)	(2) ÷ (1) RATE OF FALL (R) (cm/min)
0	0	46	0	
1	1	42	4	4
2	1	39	3	3
3	1	35.8	3.2	3.2
4	1	32.8	3	3
5	1	30	2.8	2.8
6	1	27	3	3
7	1	23.8	3.2	3.2
8	1	20.8	3	3

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Engineering Technologies Canada Ltd.

OWNER'S NAME:

SITE LOCATION: 1401 Scugog Line 6, Port Perry

PID #:

TEST PIT #: BH112

TECHNICIAN: Pincheng Zhao, Young Chong

DATE: 2021/12/15

WEATHER/TEMPERATURE: Raining, 5°C

FIELD PERMEABILITY TEST #:

D – reservoir diameter (cm)	8.25	Soil Texture	Clay
d – well hole diameter (cm)	8.3	Soil Structure	Strongly Compacted
H – height of water in well (cm)	15	α* (cm-1)	0.01
Depth below ground surface (cm)	38	C – Factor	1.27

TIME (min)	(1) CHANGE IN TIME (min)	RESERVOIR WATER LEVEL (WL) (cm)	(2) CHANGE IN WL (cm)	(2) ÷ (1) RATE OF FALL (R) (cm/min)
0	0	38.5	0	
20	20	37	1.5	0.075
40	20	36.5	0.5	0.025
60	20	36	0.5	0.025
80	20	35.5	0.5	0.025
100	20	35	0.5	0.025

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APPENDIX VI –CLIMATE DATA TABLE

APPENDIX E

Water Balance Model Output

Cited in:
NONQUON RIVER SUBWATERSHED STUDY AND NONQUON INDUSTRIAL TRIBUTARY AREA
MASTER DRAINAGE PLAN
FINAL REPORT

Prepared by:
GREENLAND INTERNATIONAL CONSULTING LTD. & STATNTEC CONSULTING LTD.

OCTOBER 2005

Burketon McLaughlin WATER BUDGET MEANS FOR THE PERIOD 1969-2001 DC20492

LAT.... 44.03 WATER HOLDING CAPACITY...132 MM HEAT INDEX... 34.58
LONG... 78.13 LOWER ZONE..... 79 MM A..... 1.048

DATE	TEMP (C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
31- 1	-8.0	59	15	15	1	1	0	29	58	129	295
28- 2	-6.8	46	16	24	1	1	0	38	63	130	342
31- 3	-1.9	57	37	59	7	7	0	87	24	132	400
30- 4	5.5	73	69	28	32	32	0	66	0	131	471
31- 5	12.4	84	84	0	77	77	0	20	0	118	555
30- 6	17.0	90	90	0	108	108	0	7	0	93	645
31- 7	19.9	75	75	0	129	121	-8	0	0	47	720
31- 8	19.1	86	86	0	114	99	-15	1	0	33	807
30- 9	14.6	91	91	0	75	70	-5	7	0	47	901
31-10	8.2	82	82	0	38	37	-1	7	0	85	82
30-11	2.0	89	81	6	12	12	0	39	3	120	170
31-12	-4.6	67	29	12	2	2	0	28	29	131	237
AVE	6.3 TTL	898	755	144	596	567	-29	329			

Burketon McLaughlin STANDARD DEVIATIONS FOR THE PERIOD 1969-2001 DC20492

DATE	TEMP (C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
31- 1	2.7	32	19	15	1	1	0	30	39	10	59
28- 2	2.5	28	19	25	1	1	0	36	49	9	61
31- 3	2.3	27	28	37	5	5	0	49	53	1	61
30- 4	1.8	30	29	53	8	8	0	51	0	3	74
31- 5	1.9	31	31	0	13	13	0	24	0	18	87
30- 6	1.4	48	48	0	9	9	0	21	0	35	99
31- 7	1.1	30	30	0	7	11	11	1	0	42	110
31- 8	1.2	34	34	0	7	18	19	4	0	37	110
30- 9	1.2	38	38	0	6	12	11	27	0	39	119
31-10	1.6	34	34	0	7	7	3	14	0	43	35
30-11	1.6	35	33	5	5	5	0	39	6	20	58
31-12	2.8	24	25	9	2	2	0	29	26	4	62

Burketon McLaughlin WATER BUDGET VALUES FOR THE PERIOD 1969-2001
DC20492

YR MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1969- 1	-7.9	.0	.0	.0	.0	.0	.0	.0	.0	79.0	.0
1969- 2	-6.1	20.3	.0	.0	.0	.0	.0	.0	20.3	79.0	20.3
1969- 3	-4.1	34.3	30.0	20.3	2.9	2.9	.0	.0	4.3	126.4	54.6
1969- 4	6.0	95.4	83.0	16.7	33.1	33.1	.0	61.0	.0	132.0	150.0
1969- 5	10.9	95.9	95.9	.0	67.2	67.2	.0	28.7	.0	132.0	245.9
1969- 6	15.3	66.8	66.8	.0	96.9	96.9	.0	.0	.0	101.9	312.7
1969- 7	19.3	116.0	116.0	.0	125.0	125.0	.0	.0	.0	93.0	428.7
1969- 8	20.1	60.0	60.0	.0	120.4	120.4	.0	.0	.0	32.5	488.7
1969- 9	15.3	7.4	7.4	.0	79.0	36.9	-42.1	.0	.0	3.0	496.1
1969-10	7.9	69.3	69.3	.0	37.5	37.5	.0	.0	.0	34.9	69.3
1969-11	2.1	120.5	101.9	.8	12.8	12.8	.0	.0	17.8	124.8	189.8
1969-12	-7.4	66.2	1.0	.0	.0	.0	.0	.0	83.0	125.8	256.0
1970- 1	-7.9	.0	.0	.0	.0	.0	.0	.0	83.0	125.8	256.0
1970- 2	-6.7	.0	.0	.0	.0	.0	.0	.0	83.0	125.8	256.0
1970- 3	-4.1	46.1	22.6	29.8	1.0	1.0	.0	45.3	76.7	132.0	302.1
1970- 4	5.9	81.6	76.5	81.8	34.5	34.5	.0	123.8	.0	132.0	383.7
1970- 5	11.8	48.3	48.3	.0	72.7	72.7	.0	.0	.0	107.6	432.0
1970- 6	16.7	46.0	46.0	.0	105.7	105.7	.0	.0	.0	47.9	478.0
1970- 7	19.8	88.2	88.2	.0	128.0	112.4	-15.7	.0	.0	23.8	566.2
1970- 8	19.4	52.5	52.5	.0	116.3	71.7	-44.6	.0	.0	4.6	618.7
1970- 9	14.9	88.1	88.1	.0	76.5	76.5	.0	.0	.0	16.2	706.8
1970-10	9.9	94.8	94.8	.0	45.5	45.5	.0	.0	.0	65.5	94.8
1970-11	3.2	75.0	73.7	1.3	15.0	15.0	.0	.0	.0	125.5	169.8
1970-12	-7.2	77.7	14.8	3.3	1.8	1.8	.0	9.8	59.6	132.0	247.5
1971- 1	-10.7	46.0	.0	.0	.0	.0	.0	.0	105.6	132.0	293.5
1971- 2	-6.5	95.9	38.6	10.7	.4	.4	.0	49.0	152.2	132.0	389.4
1971- 3	-4.5	32.0	6.1	24.9	.9	.9	.0	30.0	153.2	132.0	421.4
1971- 4	3.1	36.3	35.8	153.7	19.2	19.2	.0	170.3	.0	132.0	457.7
1971- 5	11.2	31.0	31.0	.0	68.9	68.9	.0	.0	.0	94.1	488.7
1971- 6	17.4	154.9	154.9	.0	110.6	110.6	.0	6.4	.0	132.0	643.6
1971- 7	18.2	98.0	98.0	.0	117.3	117.3	.0	.0	.0	112.7	741.6
1971- 8	18.0	58.6	58.6	.0	107.5	107.5	.0	.0	.0	63.8	800.2
1971- 9	16.4	65.6	65.6	.0	85.3	81.5	-3.8	.0	.0	47.9	865.8
1971-10	12.2	38.2	38.2	.0	56.5	49.3	-7.2	.0	.0	36.9	38.2
1971-11	1.1	55.2	37.7	8.6	9.6	9.6	.0	.0	8.9	73.6	93.4
1971-12	-3.3	85.3	55.3	15.7	1.9	1.9	.0	10.6	23.2	132.0	178.7
1972- 1	-7.4	46.9	14.5	.0	.0	.0	.0	14.5	55.6	132.0	225.6
1972- 2	-9.3	74.3	.0	.0	.0	.0	.0	.0	129.9	132.0	299.9
1972- 3	-5.5	64.1	15.3	20.1	.6	.6	.0	34.8	158.6	132.0	364.0
1972- 4	1.6	42.8	39.8	161.6	16.6	16.6	.0	184.8	.0	132.0	406.8
1972- 5	13.1	57.7	57.7	.0	81.6	81.6	.0	.0	.0	108.1	464.5
1972- 6	15.2	102.4	102.4	.0	96.0	96.0	.0	.0	.0	114.5	566.9
1972- 7	19.4	52.4	52.4	.0	125.5	125.5	.0	.0	.0	41.3	619.3
1972- 8	17.7	111.0	111.0	.0	105.1	105.1	.0	.0	.0	47.2	730.3
1972- 9	14.8	72.1	72.1	.0	76.1	74.5	-1.6	.0	.0	44.8	802.4
1972-10	5.4	93.8	92.5	1.3	25.7	25.7	.0	.0	.0	113.0	93.8
1972-11	.1	61.3	60.0	1.3	6.1	6.1	.0	36.1	.0	132.0	155.1
1972-12	-4.8	106.4	9.7	18.2	.5	.5	.0	27.5	78.5	132.0	261.5
1973- 1	-6.3	44.1	11.2	43.9	1.3	1.3	.0	53.8	67.5	132.0	305.6
1973- 2	-8.8	31.0	5.1	9.9	.3	.3	.0	14.7	83.5	132.0	336.6
1973- 3	2.3	117.6	113.5	87.6	15.6	15.6	.0	185.4	.0	132.0	454.2
1973- 4	5.4	76.4	76.4	.0	30.7	30.7	.0	45.7	.0	132.0	530.6

Burketon McLaughlin WATER BUDGET VALUES FOR THE PERIOD 1969-2001
DC20492

YR MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1973- 5	10.2	110.8	110.8	.0	62.8	62.8	.0	48.0	.0	132.0	641.4
1973- 6	17.8	67.5	67.5	.0	113.1	113.1	.0	.0	.0	86.4	708.9
1973- 7	20.2	57.2	57.2	.0	131.2	131.2	.0	.0	.0	12.5	766.1
1973- 8	21.1	56.5	56.5	.0	126.5	67.5	-59.0	.0	.0	1.4	822.6
1973- 9	15.1	50.6	50.6	.0	78.3	51.1	-27.2	.0	.0	.9	873.2
1973-10	10.3	119.5	119.5	.0	47.7	47.7	.0	.0	.0	72.8	119.5
1973-11	1.7	105.2	94.7	10.5	9.3	9.3	.0	36.7	.0	132.0	224.7
1973-12	-5.6	64.3	22.2	11.5	2.4	2.4	.0	31.3	30.6	132.0	289.0
1974- 1	-7.0	57.4	27.7	32.5	.9	.9	.0	59.3	27.8	132.0	346.4
1974- 2	-9.6	42.8	10.6	17.7	.6	.6	.0	27.7	42.3	132.0	389.2
1974- 3	-3.2	60.6	32.0	45.9	2.9	2.9	.0	75.0	25.0	132.0	449.8
1974- 4	6.5	101.9	101.9	25.0	36.5	36.5	.0	90.4	.0	132.0	551.7
1974- 5	9.6	127.4	127.4	.0	59.0	59.0	.0	68.4	.0	132.0	679.1
1974- 6	16.1	93.3	93.3	.0	101.8	101.8	.0	.0	.0	123.5	772.4
1974- 7	19.5	82.1	82.1	.0	126.4	126.4	.0	.0	.0	79.1	854.5
1974- 8	19.1	43.5	43.5	.0	114.2	114.2	.0	.0	.0	8.4	898.0
1974- 9	13.0	69.6	69.6	.0	66.5	66.5	.0	.0	.0	11.5	967.6
1974-10	6.0	42.2	42.2	.0	28.2	28.2	.0	.0	.0	25.5	42.2
1974-11	1.7	85.1	83.6	.0	12.0	12.0	.0	.0	1.5	97.1	127.3
1974-12	-2.5	46.9	20.8	15.0	.6	.6	.0	.2	12.6	132.0	174.2
1975- 1	-5.4	64.1	36.8	25.7	1.2	1.2	.0	61.2	14.3	132.0	238.3
1975- 2	-5.7	84.1	56.6	26.6	.8	.8	.0	82.4	15.1	132.0	322.4
1975- 3	-4.1	85.8	53.9	33.9	1.7	1.7	.0	86.2	13.1	132.0	408.2
1975- 4	1.3	70.3	44.6	38.8	17.3	17.3	.0	66.1	.0	132.0	478.5
1975- 5	15.3	66.5	66.5	.0	95.8	95.8	.0	.0	.0	102.7	545.0
1975- 6	17.6	71.2	71.2	.0	111.9	111.9	.0	.0	.0	61.9	616.2
1975- 7	20.5	71.3	71.3	.0	133.0	119.6	-13.3	.0	.0	13.6	687.5
1975- 8	19.6	60.3	60.3	.0	117.6	70.2	-47.5	.0	.0	3.7	747.8
1975- 9	13.0	82.8	82.8	.0	66.3	66.3	.0	.0	.0	20.3	830.6
1975-10	9.5	33.3	33.3	.0	44.0	36.0	-7.9	.0	.0	17.5	33.3
1975-11	5.4	87.0	74.3	12.7	23.1	23.1	.0	.0	.0	81.4	120.3
1975-12	-6.4	71.1	22.2	17.5	1.1	1.1	.0	.0	31.4	120.0	191.4
1976- 1	-10.9	87.3	.0	.0	.0	.0	.0	.0	118.7	120.0	278.7
1976- 2	-4.1	60.0	36.1	62.4	2.1	2.1	.0	84.4	80.2	132.0	338.7
1976- 3	-1.2	110.2	68.0	122.4	10.5	10.5	.0	179.9	.0	132.0	448.9
1976- 4	6.8	58.3	58.3	.0	37.9	37.9	.0	20.4	.0	132.0	507.2
1976- 5	10.1	133.5	133.5	.0	61.9	61.9	.0	71.6	.0	132.0	640.7
1976- 6	18.6	119.4	119.4	.0	118.4	118.4	.0	1.0	.0	132.0	760.1
1976- 7	19.3	116.9	116.9	.0	124.8	124.8	.0	.0	.0	124.1	877.0
1976- 8	17.7	51.0	51.0	.0	105.8	105.8	.0	.0	.0	69.3	928.0
1976- 9	13.7	87.8	87.8	.0	70.3	70.3	.0	.0	.0	86.8	1015.8
1976-10	5.6	63.9	63.9	.0	27.0	27.0	.0	.0	.0	123.7	63.9
1976-11	-1.0	27.0	19.6	7.4	3.7	3.7	.0	15.0	.0	132.0	90.9
1976-12	-9.2	39.1	.0	.0	.0	.0	.0	.0	39.1	132.0	130.0
1977- 1	-13.1	70.2	.0	.0	.0	.0	.0	.0	109.3	132.0	200.2
1977- 2	-7.4	31.0	2.3	13.9	.3	.3	.0	15.9	124.1	132.0	231.2
1977- 3	1.0	86.0	72.6	137.5	14.1	14.1	.0	196.0	.0	132.0	317.2
1977- 4	6.9	89.5	83.7	5.8	41.5	41.5	.0	48.0	.0	132.0	406.7
1977- 5	14.4	31.9	31.9	.0	90.0	90.0	.0	.0	.0	73.9	438.6
1977- 6	16.2	82.6	82.6	.0	102.5	101.2	-1.3	.0	.0	55.4	521.2
1977- 7	19.7	60.6	60.6	.0	127.6	107.6	-20.1	.0	.0	8.4	581.8
1977- 8	17.8	126.7	126.7	.0	106.2	106.2	.0	.0	.0	28.9	708.5
1977- 9	14.4	138.0	138.0	.0	74.0	74.0	.0	.0	.0	92.9	846.5
1977-10	7.4	83.7	83.7	.0	33.3	33.3	.0	11.3	.0	132.0	83.7
1977-11	2.9	126.9	116.4	.3	16.5	16.5	.0	100.2	10.2	132.0	210.6
1977-12	-5.9	93.4	8.6	15.8	.8	.8	.0	23.6	79.2	132.0	304.0

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YR	MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1978-	1	-10.0	116.4	21.4	4.6	.1	.1	.0	25.9	169.7	132.0	420.4
1978-	2	-10.5	18.8	.0	.0	.0	.0	.0	.0	188.5	132.0	439.2
1978-	3	-4.9	49.7	29.3	24.3	.9	.9	.0	52.7	184.5	132.0	488.9
1978-	4	3.3	33.0	33.0	184.5	19.9	19.9	.0	197.6	.0	132.0	521.9
1978-	5	13.0	99.6	99.6	.0	81.8	81.8	.0	17.8	.0	132.0	621.5
1978-	6	16.3	41.7	41.7	.0	103.3	103.3	.0	.0	.0	70.4	663.2
1978-	7	19.6	65.6	65.6	.0	126.7	120.0	-6.7	.0	.0	16.0	728.8
1978-	8	19.1	80.2	80.2	.0	114.1	87.1	-27.1	.0	.0	9.1	809.0
1978-	9	13.8	115.3	115.3	.0	71.1	71.1	.0	.0	.0	53.3	924.3
1978-	10	7.3	72.0	72.0	.0	33.4	33.4	.0	.0	.0	91.9	72.0
1978-	11	2.3	47.5	45.0	2.5	14.5	14.5	.0	.0	.0	124.9	119.5
1978-	12	-3.9	47.5	25.1	15.3	.6	.6	.0	32.6	7.1	132.0	167.0
1979-	1	-8.7	81.0	.0	2.3	.0	.0	.0	2.3	85.9	132.0	248.0
1979-	2	-12.5	23.2	.0	2.3	.0	.0	.0	2.2	106.8	132.0	271.2
1979-	3	.9	67.3	62.9	111.2	12.4	12.4	.0	161.7	.0	132.0	338.5
1979-	4	4.6	103.0	92.6	10.4	27.9	27.9	.0	75.1	.0	132.0	441.5
1979-	5	11.3	80.0	80.0	.0	70.0	70.0	.0	10.0	.0	132.0	521.5
1979-	6	16.9	66.8	66.8	.0	106.9	106.9	.0	.0	.0	91.9	588.3
1979-	7	20.2	30.4	30.4	.0	130.9	122.3	-8.6	.0	.0	.0	618.7
1979-	8	17.8	81.4	81.4	.0	106.1	81.4	-24.7	.0	.0	.0	700.1
1979-	9	14.5	50.0	50.0	.0	74.6	50.0	-24.6	.0	.0	.0	750.1
1979-	10	7.4	103.6	103.6	.0	34.3	34.3	.0	.0	.0	69.3	103.6
1979-	11	3.3	110.9	107.9	1.4	14.0	14.0	.0	32.7	1.6	132.0	214.5
1979-	12	-3.0	80.8	64.2	18.2	2.4	2.4	.0	80.0	.0	132.0	295.3
1980-	1	-8.0	10.6	2.6	.0	.0	.0	.0	2.6	8.0	132.0	305.9
1980-	2	-8.1	35.0	2.0	8.6	.2	.2	.0	10.4	32.4	132.0	340.9
1980-	3	-3.1	103.1	85.1	50.4	5.0	5.0	.0	130.5	.0	132.0	444.0
1980-	4	5.9	118.3	118.3	.0	32.0	32.0	.0	86.3	.0	132.0	562.3
1980-	5	13.6	47.4	47.4	.0	84.4	84.4	.0	.0	.0	95.0	609.7
1980-	6	13.9	148.6	148.6	.0	87.5	87.5	.0	24.1	.0	132.0	758.3
1980-	7	19.8	136.2	136.2	.0	127.9	127.9	.0	8.3	.0	132.0	894.5
1980-	8	20.2	46.6	46.6	.0	121.5	121.5	.0	.0	.0	57.1	941.1
1980-	9	13.9	79.6	79.6	.0	71.9	71.9	.0	.0	.0	64.8	1020.7
1980-	10	6.2	117.6	117.6	.0	28.2	28.2	.0	22.2	.0	132.0	117.6
1980-	11	.2	57.6	55.4	.0	5.2	5.2	.0	50.2	2.2	132.0	175.2
1980-	12	-8.2	69.6	49.2	4.0	1.3	1.3	.0	51.9	18.6	132.0	244.8
1981-	1	-12.3	15.0	.0	9.1	.3	.3	.0	8.8	24.5	132.0	259.8
1981-	2	-3.0	86.0	51.6	58.9	5.9	5.9	.0	104.6	.0	132.0	345.8
1981-	3	-.6	29.0	15.8	13.2	10.6	10.6	.0	18.4	.0	132.0	374.8
1981-	4	6.2	64.4	64.4	.0	33.5	33.5	.0	30.9	.0	132.0	439.2
1981-	5	11.8	91.6	91.6	.0	73.3	73.3	.0	18.3	.0	132.0	530.8
1981-	6	16.9	87.0	87.0	.0	107.2	107.2	.0	.0	.0	111.8	617.8
1981-	7	20.2	69.4	69.4	.0	131.0	131.0	.0	.0	.0	50.2	687.2
1981-	8	18.8	135.2	135.2	.0	112.9	112.9	.0	.0	.0	72.5	822.4
1981-	9	13.9	101.9	101.9	.0	72.1	72.1	.0	.0	.0	102.3	924.3
1981-	10	5.9	105.4	105.4	.0	26.4	26.4	.0	49.3	.0	132.0	105.4
1981-	11	2.5	64.0	64.0	.0	12.8	12.8	.0	51.2	.0	132.0	169.4
1981-	12	-4.3	23.0	5.2	3.0	.6	.6	.0	7.6	14.8	132.0	192.4
1982-	1	-11.5	48.6	3.4	8.1	.2	.2	.0	11.3	51.9	132.0	241.0
1982-	2	-7.5	19.2	.0	6.3	.1	.1	.0	6.1	64.9	132.0	260.2
1982-	3	-3.0	32.7	21.7	75.9	3.8	3.8	.0	93.8	.0	132.0	292.9
1982-	4	3.8	36.8	36.0	.8	28.0	28.0	.0	8.8	.0	132.0	329.7
1982-	5	14.5	72.6	72.6	.0	90.0	90.0	.0	.0	.0	114.6	402.3
1982-	6	14.7	118.6	118.6	.0	92.8	92.8	.0	8.4	.0	132.0	520.9
1982-	7	19.8	49.2	49.2	.0	127.8	127.8	.0	.0	.0	53.4	570.1
1982-	8	17.0	58.4	58.4	.0	101.2	87.3	-13.9	.0	.0	24.5	628.5
1982-	9	14.3	84.2	84.2	.0	73.6	73.6	.0	.0	.0	35.1	712.7

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YR MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1982-10	9.2	61.4	61.4	.0	42.3	42.3	.0	.0	.0	54.2	61.4
1982-11	3.2	136.2	125.0	11.2	15.3	15.3	.0	43.1	.0	132.0	197.6
1982-12	-.9	98.6	93.4	5.2	9.0	9.0	.0	89.6	.0	132.0	296.2
1983- 1	-5.8	43.5	11.7	21.2	.6	.6	.0	32.2	10.6	132.0	339.7
1983- 2	-4.7	42.0	31.4	21.2	1.5	1.5	.0	51.1	.0	132.0	381.7
1983- 3	-.3	52.4	49.6	2.8	9.4	9.4	.0	43.0	.0	132.0	434.1
1983- 4	4.6	92.8	92.6	.2	24.9	24.9	.0	67.9	.0	132.0	526.9
1983- 5	9.6	137.4	137.4	.0	58.7	58.7	.0	78.7	.0	132.0	664.3
1983- 6	17.8	30.6	30.6	.0	113.2	113.2	.0	.0	.0	49.4	694.9
1983- 7	21.0	65.6	65.6	.0	136.5	110.0	-26.5	.0	.0	5.1	760.5
1983- 8	20.3	65.6	65.6	.0	122.1	69.2	-52.9	.0	.0	1.4	826.1
1983- 9	16.6	61.8	61.8	.0	86.1	62.2	-23.9	.0	.0	1.0	887.9
1983-10	8.7	124.2	124.2	.0	40.3	40.3	.0	.0	.0	84.9	124.2
1983-11	1.8	91.0	85.4	5.0	10.6	10.6	.0	32.7	.6	132.0	215.2
1983-12	-7.6	93.2	26.2	12.5	.2	.2	.0	38.5	55.1	132.0	308.4
1984- 1	-10.6	43.6	3.0	.0	.0	.0	.0	3.0	95.7	132.0	352.0
1984- 2	-2.4	60.2	35.4	107.9	4.7	4.7	.0	138.6	12.6	132.0	412.2
1984- 3	-6.1	40.8	13.4	39.4	1.4	1.4	.0	51.4	.6	132.0	453.0
1984- 4	6.7	111.0	111.0	.6	36.2	36.2	.0	75.4	.0	132.0	564.0
1984- 5	9.6	80.4	80.4	.0	58.7	58.7	.0	21.7	.0	132.0	644.4
1984- 6	17.5	42.4	42.4	.0	110.8	110.8	.0	.0	.0	63.6	686.8
1984- 7	19.2	45.6	45.6	.0	124.1	108.8	-15.3	.0	.0	.4	732.4
1984- 8	20.1	104.4	104.4	.0	120.8	104.5	-16.4	.0	.0	.3	836.8
1984- 9	13.2	71.2	71.2	.0	67.5	67.5	.0	.0	.0	4.0	908.0
1984-10	10.0	35.6	35.6	.0	45.6	36.1	-9.5	.0	.0	3.5	35.6
1984-11	1.7	87.2	86.6	.6	10.3	10.3	.0	.0	.0	80.3	122.8
1984-12	-1.8	46.1	16.3	28.2	2.9	2.9	.0	.0	1.6	121.9	168.9
1985- 1	-10.2	86.9	.0	.0	.0	.0	.0	.0	88.5	121.9	255.8
1985- 2	-6.9	104.6	70.4	39.2	1.3	1.3	.0	98.1	83.5	132.0	360.4
1985- 3	-1.3	33.8	15.2	102.1	5.3	5.3	.0	112.0	.0	132.0	394.2
1985- 4	6.6	35.8	35.0	.8	38.6	38.6	.0	.0	.0	129.2	430.0
1985- 5	12.8	108.6	108.6	.0	79.1	79.1	.0	26.6	.0	132.0	538.6
1985- 6	14.9	19.4	19.4	.0	94.2	94.2	.0	.0	.0	57.2	558.0
1985- 7	18.9	51.6	51.6	.0	122.3	102.8	-19.5	.0	.0	6.0	609.6
1985- 8	18.4	88.8	88.8	.0	110.1	90.4	-19.7	.0	.0	4.4	698.4
1985- 9	16.3	90.8	90.8	.0	84.6	84.6	.0	.0	.0	10.6	789.2
1985-10	9.1	84.2	84.2	.0	42.0	42.0	.0	.0	.0	52.8	84.2
1985-11	1.6	175.8	162.2	13.3	10.8	10.8	.0	85.5	.3	132.0	260.0
1985-12	-6.6	72.2	13.0	.3	.4	.4	.0	12.9	59.2	132.0	332.2
1986- 1	-7.9	25.0	13.2	27.5	.8	.8	.0	39.9	43.5	132.0	357.2
1986- 2	-7.8	33.4	3.4	6.3	.1	.1	.0	9.6	67.3	132.0	390.6
1986- 3	-.8	73.5	53.5	87.3	11.6	11.6	.0	129.1	.0	132.0	464.1
1986- 4	7.7	47.4	45.6	1.8	42.4	42.4	.0	5.0	.0	132.0	511.5
1986- 5	14.2	66.8	66.8	.0	88.5	88.5	.0	.0	.0	110.3	578.3
1986- 6	16.0	119.0	119.0	.0	101.2	101.2	.0	.0	.0	128.2	697.3
1986- 7	19.9	76.6	76.6	.0	128.9	128.9	.0	.0	.0	75.9	773.9
1986- 8	17.8	186.8	186.8	.0	106.5	106.5	.0	24.2	.0	132.0	960.7
1986- 9	13.9	198.8	198.8	.0	71.1	71.1	.0	127.7	.0	132.0	1159.5
1986-10	8.2	58.8	58.8	.0	37.3	37.3	.0	21.5	.0	132.0	58.8
1986-11	.5	34.4	28.8	5.6	7.9	7.9	.0	26.5	.0	132.0	93.2
1986-12	-3.0	54.4	20.2	19.3	.4	.4	.0	39.1	14.9	132.0	147.6
1987- 1	-6.4	46.8	.0	.0	.0	.0	.0	.0	61.7	132.0	194.4
1987- 2	-7.3	22.0	16.0	4.7	.1	.1	.0	20.6	63.0	132.0	216.4
1987- 3	.5	46.8	39.4	64.0	14.0	14.0	.0	89.4	6.4	132.0	263.2
1987- 4	9.0	49.6	46.0	10.0	49.8	49.8	.0	6.2	.0	132.0	312.8
1987- 5	14.0	44.8	44.8	.0	87.5	87.5	.0	.0	.0	89.3	357.6

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YR MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1987- 6	18.2	101.0	101.0	.0	115.7	115.7	.0	.0	.0	74.7	458.6
1987- 7	21.2	78.2	78.2	.0	137.9	134.6	-3.3	.0	.0	18.2	536.8
1987- 8	18.6	135.6	135.6	.0	111.3	111.3	.0	.0	.0	42.5	672.4
1987- 9	14.9	89.4	89.4	.0	76.8	76.8	.0	.0	.0	55.1	761.8
1987-10	6.5	82.6	82.6	.0	29.3	29.3	.0	.0	.0	108.4	82.6
1987-11	1.9	91.4	81.8	9.6	12.3	12.3	.0	55.5	.0	132.0	174.0
1987-12	-2.1	29.0	21.8	7.2	2.5	2.5	.0	26.5	.0	132.0	203.0
1988- 1	-6.6	60.4	27.2	33.2	2.0	2.0	.0	58.4	.0	132.0	263.4
1988- 2	-7.8	69.2	6.0	.0	.0	.0	.0	6.0	63.2	132.0	332.6
1988- 3	-2.4	35.0	29.4	68.8	6.6	6.6	.0	91.6	.0	132.0	367.6
1988- 4	5.5	88.2	88.2	.0	29.1	29.1	.0	59.1	.0	132.0	455.8
1988- 5	13.8	54.0	54.0	.0	85.7	85.7	.0	.0	.0	100.3	509.8
1988- 6	17.0	81.6	81.6	.0	108.0	108.0	.0	.0	.0	73.8	591.4
1988- 7	22.4	137.8	137.8	.0	145.5	145.0	-.5	.0	.0	66.6	729.2
1988- 8	20.3	76.8	76.8	.0	122.5	115.3	-7.2	.0	.0	28.1	806.0
1988- 9	14.2	113.8	113.8	.0	72.7	72.7	.0	.0	.0	69.2	919.8
1988-10	6.1	114.4	114.4	.0	28.0	28.0	.0	23.6	.0	132.0	114.4
1988-11	3.3	90.4	90.4	.0	12.8	12.8	.0	77.6	.0	132.0	204.8
1988-12	-5.1	64.0	9.8	6.4	1.0	1.0	.0	15.2	47.8	132.0	268.8
1989- 1	-4.5	31.5	11.9	25.3	.8	.8	.0	36.4	42.2	132.0	300.3
1989- 2	-7.8	23.0	2.2	10.6	.2	.2	.0	12.5	52.4	132.0	323.3
1989- 3	-3.6	42.4	21.4	73.4	6.3	6.3	.0	88.5	.0	132.0	365.7
1989- 4	3.9	50.6	50.2	.4	22.3	22.3	.0	28.3	.0	132.0	416.3
1989- 5	12.6	104.2	104.2	.0	78.7	78.7	.0	25.5	.0	132.0	520.5
1989- 6	17.7	95.8	95.8	.0	112.7	112.7	.0	.0	.0	115.1	616.3
1989- 7	21.1	6.0	6.0	.0	136.7	121.1	-15.7	.0	.0	.0	622.3
1989- 8	19.1	94.2	94.2	.0	114.9	94.2	-20.7	.0	.0	.0	716.5
1989- 9	14.7	118.2	118.2	.0	76.0	76.0	.0	.0	.0	42.2	834.7
1989-10	9.4	112.0	112.0	.0	42.6	42.6	.0	.0	.0	111.6	112.0
1989-11	.6	144.6	126.6	12.8	9.9	9.9	.0	109.1	5.2	132.0	256.6
1989-12	-11.8	42.8	.0	.0	.0	.0	.0	.0	48.0	132.0	299.4
1990- 1	-2.8	57.1	24.5	26.9	.6	.6	.0	50.8	53.7	132.0	356.5
1990- 2	-5.7	62.8	27.8	33.8	1.0	1.0	.0	60.6	54.9	132.0	419.3
1990- 3	-.3	46.6	42.8	58.7	12.9	12.9	.0	88.7	.0	132.0	465.9
1990- 4	7.1	81.2	81.2	.0	41.1	41.1	.0	40.1	.0	132.0	547.1
1990- 5	11.0	102.6	102.6	.0	68.0	68.0	.0	34.6	.0	132.0	649.7
1990- 6	17.2	101.2	101.2	.0	109.0	109.0	.0	.0	.0	124.2	750.9
1990- 7	19.8	80.4	80.4	.0	128.3	128.3	.0	.0	.0	76.3	831.3
1990- 8	19.5	61.4	61.4	.0	116.6	114.8	-1.9	.0	.0	23.0	892.7
1990- 9	14.1	89.8	89.8	.0	72.4	72.4	.0	.0	.0	40.4	982.5
1990-10	8.1	117.8	117.8	.0	37.4	37.4	.0	.0	.0	120.8	117.8
1990-11	2.9	58.2	54.0	4.2	13.4	13.4	.0	33.6	.0	132.0	176.0
1990-12	-2.6	109.8	80.0	24.5	2.4	2.4	.0	102.1	5.3	132.0	285.8
1991- 1	-7.7	43.6	3.4	7.0	.1	.1	.0	10.2	38.5	132.0	329.4
1991- 2	-4.2	26.8	6.8	51.9	2.4	2.4	.0	56.4	6.6	132.0	356.2
1991- 3	-.5	117.9	94.3	30.2	7.5	7.5	.0	117.0	.0	132.0	474.1
1991- 4	7.2	144.6	144.6	.0	39.5	39.5	.0	105.1	.0	132.0	618.7
1991- 5	15.3	71.6	71.6	.0	96.4	96.4	.0	.0	.0	107.2	690.3
1991- 6	19.0	45.6	45.6	.0	121.3	121.3	.0	.0	.0	31.5	735.9
1991- 7	20.6	63.8	63.8	.0	133.6	91.6	-41.9	.0	.0	3.7	799.7
1991- 8	20.3	111.8	111.8	.0	121.7	112.3	-9.5	.0	.0	3.2	911.5
1991- 9	14.2	94.0	94.0	.0	73.8	73.8	.0	.0	.0	23.4	1005.5
1991-10	9.5	71.3	71.3	.0	43.6	43.6	.0	.0	.0	51.1	71.3
1991-11	1.5	62.4	59.4	3.0	9.2	9.2	.0	.0	.0	104.3	133.7
1991-12	-4.0	70.4	24.8	22.7	2.7	2.7	.0	17.1	22.9	132.0	204.1
1992- 1	-6.3	67.3	22.6	28.5	1.0	1.0	.0	50.1	39.1	132.0	271.4

Burketon McLaughlin WATER BUDGET VALUES FOR THE PERIOD 1969-2001
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YR MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1992- 2	-5.3	49.6	15.0	11.6	.3	.3	.0	26.3	62.1	132.0	321.0
1992- 3	-3.5	49.6	32.8	78.9	5.3	5.3	.0	106.4	.0	132.0	370.6
1992- 4	4.6	111.0	96.8	14.2	26.6	26.6	.0	84.4	.0	132.0	481.6
1992- 5	12.3	67.0	67.0	.0	76.0	76.0	.0	.0	.0	123.0	548.6
1992- 6	15.7	30.6	30.6	.0	99.2	99.2	.0	.0	.0	54.5	579.2
1992- 7	17.0	110.9	110.9	.0	109.1	109.1	.0	.0	.0	56.2	690.1
1992- 8	17.2	147.2	147.2	.0	102.4	102.4	.0	.0	.0	101.0	837.3
1992- 9	13.8	126.2	126.2	.0	71.0	71.0	.0	24.2	.0	132.0	963.5
1992-10	6.2	72.0	71.2	.8	29.5	29.5	.0	42.5	.0	132.0	72.0
1992-11	1.6	122.6	115.2	7.4	8.7	8.7	.0	113.9	.0	132.0	194.6
1992-12	-3.6	71.2	17.2	30.5	.9	.9	.0	46.7	23.5	132.0	265.8
1993- 1	-6.0	120.9	51.6	31.5	.8	.8	.0	82.3	61.3	132.0	386.7
1993- 2	-10.6	47.5	.0	.0	.0	.0	.0	.0	108.8	132.0	434.2
1993- 3	-3.3	32.6	.0	141.4	6.3	6.3	.0	135.1	.0	132.0	466.8
1993- 4	5.6	83.4	83.4	.0	30.4	30.4	.0	53.0	.0	132.0	550.2
1993- 5	11.6	58.6	58.6	.0	71.5	71.5	.0	.0	.0	119.1	608.8
1993- 6	16.1	139.1	139.1	.0	101.7	101.7	.0	24.5	.0	132.0	747.9
1993- 7	19.9	83.6	83.6	.0	128.9	128.9	.0	.0	.0	86.7	831.5
1993- 8	19.7	73.4	73.4	.0	118.0	118.0	.0	.0	.0	42.1	904.9
1993- 9	12.5	67.0	67.0	.0	64.2	64.2	.0	.0	.0	44.9	971.9
1993-10	6.5	107.8	107.8	.0	29.9	29.9	.0	.0	.0	122.8	107.8
1993-11	.7	89.2	86.0	3.2	6.6	6.6	.0	73.5	.0	132.0	197.0
1993-12	-4.7	41.0	32.6	.4	1.8	1.8	.0	31.2	8.0	132.0	238.0
1994- 1	-14.9	52.4	.0	.0	.0	.0	.0	.0	60.4	132.0	290.4
1994- 2	-10.1	19.5	.0	39.1	1.5	1.5	.0	37.6	40.8	132.0	309.9
1994- 3	-2.1	45.2	10.8	75.2	4.0	4.0	.0	82.0	.0	132.0	355.1
1994- 4	5.6	81.8	76.6	5.2	31.1	31.1	.0	50.7	.0	132.0	436.9
1994- 5	10.7	99.2	99.2	.0	66.2	66.2	.0	33.0	.0	132.0	536.1
1994- 6	18.1	46.0	46.0	.0	115.6	115.6	.0	.0	.0	62.4	582.1
1994- 7	20.4	74.0	74.0	.0	132.4	120.1	-12.3	.0	.0	16.3	656.1
1994- 8	17.7	76.4	76.4	.0	105.5	82.4	-23.1	.0	.0	10.3	732.5
1994- 9	15.4	88.4	88.4	.0	79.1	79.1	.0	.0	.0	19.6	820.9
1994-10	9.4	37.4	37.4	.0	42.8	38.7	-4.0	.0	.0	18.3	37.4
1994-11	4.3	97.3	80.3	17.0	18.7	18.7	.0	.0	.0	96.8	134.7
1994-12	-1.6	63.4	29.4	24.6	4.7	4.7	.0	14.1	9.4	132.0	198.1
1995- 1	-4.3	124.0	77.2	51.4	3.5	3.5	.0	125.1	4.8	132.0	322.1
1995- 2	-9.0	34.4	4.0	7.4	.2	.2	.0	11.1	27.8	132.0	356.5
1995- 3	1.5	51.1	48.6	30.3	17.0	17.0	.0	62.0	.0	132.0	407.6
1995- 4	3.1	87.6	73.4	14.2	20.2	20.2	.0	67.4	.0	132.0	495.2
1995- 5	12.3	104.8	104.8	.0	75.7	75.7	.0	29.1	.0	132.0	600.0
1995- 6	19.2	34.2	34.2	.0	122.4	122.4	.0	.0	.0	43.8	634.2
1995- 7	20.2	119.8	119.8	.0	130.5	125.7	-4.8	.0	.0	37.9	754.0
1995- 8	20.6	76.0	76.0	.0	123.8	98.9	-24.9	.0	.0	15.0	830.0
1995- 9	12.8	32.7	32.7	.0	65.7	39.0	-26.8	.0	.0	8.7	862.7
1995-10	9.6	155.8	155.8	.0	44.5	44.5	.0	.0	.0	120.0	155.8
1995-11	-.8	155.1	122.3	9.2	4.7	4.7	.0	114.8	23.6	132.0	310.9
1995-12	-7.4	34.2	1.0	2.3	.0	.0	.0	3.3	54.6	132.0	345.1
1996- 1	-8.7	76.0	33.2	27.6	.8	.8	.0	60.0	69.8	132.0	421.1
1996- 2	-7.4	5.0	.0	39.4	1.2	1.2	.0	38.2	35.4	132.0	426.1
1996- 3	-3.6	30.9	11.4	54.9	4.7	4.7	.0	61.6	.0	132.0	457.0
1996- 4	3.8	96.6	94.4	2.2	21.7	21.7	.0	74.9	.0	132.0	553.6
1996- 5	11.1	87.8	87.8	.0	68.5	68.5	.0	19.3	.0	132.0	641.4
1996- 6	17.8	152.2	152.2	.0	113.2	113.2	.0	39.0	.0	132.0	793.6
1996- 7	18.8	104.3	104.3	.0	121.6	121.6	.0	.0	.0	114.7	897.9
1996- 8	19.8	135.8	135.8	.0	118.8	118.8	.0	.0	.0	131.7	1033.7
1996- 9	15.8	177.2	177.2	.0	81.7	81.7	.0	95.1	.0	132.0	1210.9
1996-10	8.6	64.4	64.4	.0	39.1	39.1	.0	25.3	.0	132.0	64.4

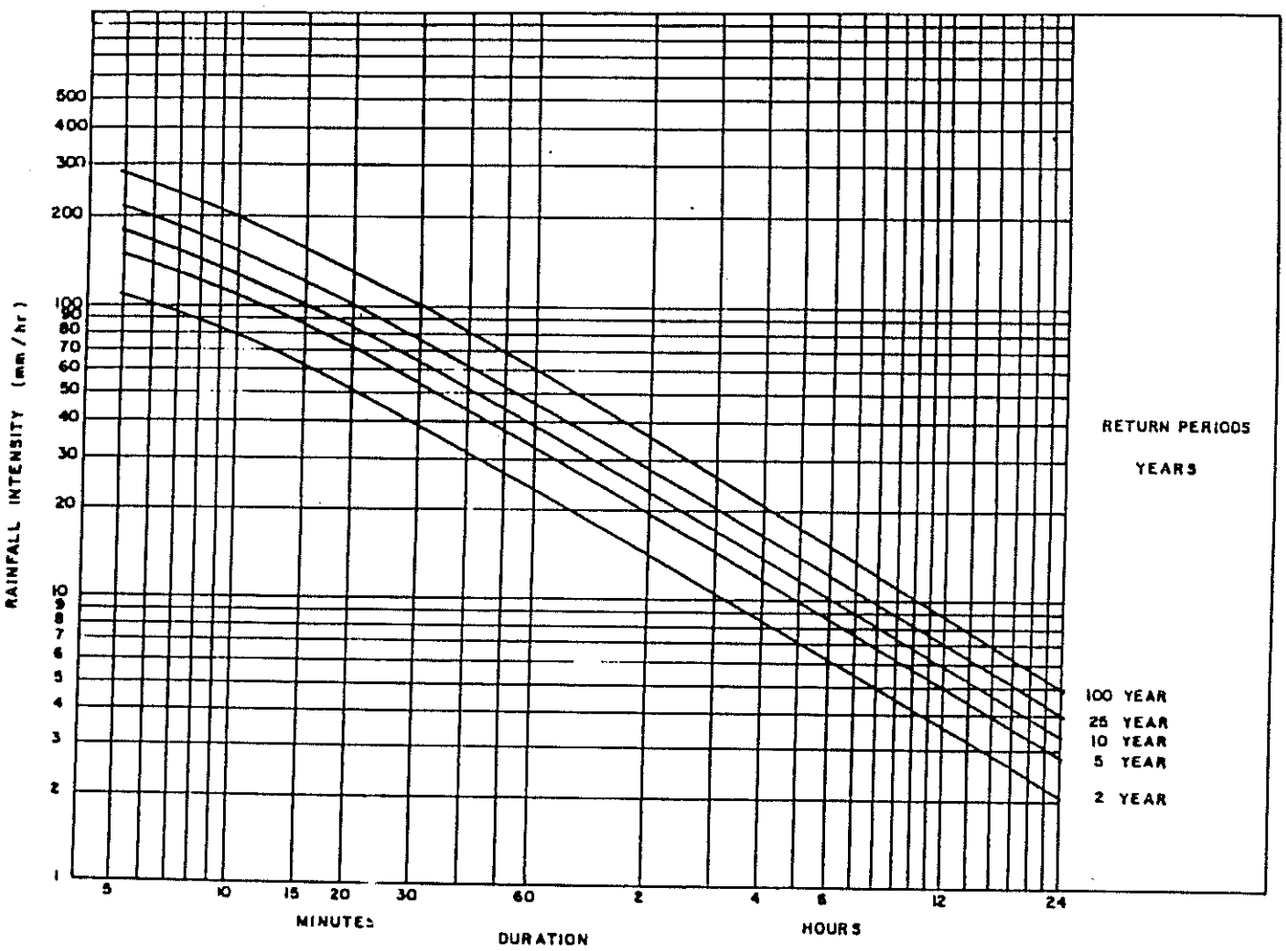
Burketon McLaughlin WATER BUDGET VALUES FOR THE PERIOD 1969-2001
DC20492

YR MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
1996-11	-.4	59.4	42.6	4.6	6.6	6.6	.0	40.6	12.2	132.0	123.8
1996-12	-1.4	99.8	75.0	20.0	3.2	3.2	.0	91.8	17.0	132.0	223.6
1997- 1	-8.0	102.9	14.6	23.8	1.4	1.4	.0	37.0	81.5	132.0	326.5
1997- 2	-4.9	90.8	37.3	28.4	1.0	1.0	.0	64.8	106.6	132.0	417.3
1997- 3	-2.7	50.2	25.7	93.6	4.0	4.0	.0	115.4	37.4	132.0	467.5
1997- 4	4.8	37.0	37.0	37.4	28.3	28.3	.0	46.2	.0	132.0	504.5
1997- 5	8.9	81.0	81.0	.0	54.2	54.2	.0	26.8	.0	132.0	585.5
1997- 6	19.1	125.0	125.0	.0	121.6	121.6	.0	3.4	.0	132.0	710.5
1997- 7	19.6	44.4	44.4	.0	126.9	126.9	.0	.0	.0	49.5	754.9
1997- 8	18.2	82.6	82.6	.0	108.6	98.9	-9.7	.0	.0	33.2	837.5
1997- 9	14.6	103.0	103.0	.0	75.3	75.3	.0	.0	.0	60.9	940.5
1997-10	8.4	40.9	39.9	1.0	39.5	39.5	.0	.0	.0	62.3	40.9
1997-11	1.0	75.2	57.8	17.4	8.6	8.6	.0	.0	.0	129.0	116.1
1997-12	-2.8	29.0	2.0	20.3	.7	.7	.0	18.5	6.7	132.0	145.1
1998- 1	-4.0	98.8	51.6	10.1	2.6	2.6	.0	59.1	43.8	132.0	243.9
1998- 2	-1.7	23.5	23.5	43.8	2.8	2.8	.0	64.5	.0	132.0	267.4
1998- 3	.2	99.5	58.5	41.0	14.6	14.6	.0	84.9	.0	132.0	366.9
1998- 4	8.1	70.9	70.9	.0	43.4	43.4	.0	27.5	.0	132.0	437.8
1998- 5	16.5	50.0	50.0	.0	103.6	103.6	.0	.0	.0	78.4	487.8
1998- 6	17.8	198.0	198.0	.0	113.3	113.3	.0	31.2	.0	132.0	685.8
1998- 7	19.8	93.2	93.2	.0	128.2	128.2	.0	.0	.0	97.0	779.0
1998- 8	20.4	98.3	98.3	.0	122.6	122.6	.0	.0	.0	72.7	877.3
1998- 9	16.7	57.0	57.0	.0	86.5	84.2	-2.4	.0	.0	45.5	934.3
1998-10	9.7	37.6	37.6	.0	44.3	41.5	-2.9	.0	.0	41.6	37.6
1998-11	3.0	57.0	57.0	.0	11.3	11.3	.0	.0	.0	87.4	94.6
1998-12	-.9	73.6	59.2	.0	7.3	7.3	.0	7.3	14.4	132.0	168.2
1999- 1	-7.7	73.6	.0	27.8	.8	.8	.0	27.0	60.2	132.0	241.8
1999- 2	-3.3	39.9	18.4	46.5	1.4	1.4	.0	63.4	35.3	132.0	281.7
1999- 3	-1.0	39.2	10.2	64.3	8.9	8.9	.0	65.6	.0	132.0	320.9
1999- 4	7.5	23.0	23.0	.0	40.0	40.0	.0	.0	.0	115.0	343.9
1999- 5	15.0	84.0	84.0	.0	93.2	93.2	.0	.0	.0	105.8	427.9
1999- 6	19.0	68.4	68.4	.0	120.9	120.9	.0	.0	.0	53.4	496.3
1999- 7	22.3	55.2	55.2	.0	145.0	108.6	-36.4	.0	.0	.0	551.5
1999- 8	19.1	60.4	60.4	.0	114.1	60.4	-53.7	.0	.0	.0	611.9
1999- 9	16.9	120.0	120.0	.0	87.9	87.9	.0	.0	.0	32.1	731.9
1999-10	8.2	106.4	106.4	.0	37.0	37.0	.0	.0	.0	101.5	106.4
1999-11	4.1	103.8	103.8	.0	17.3	17.3	.0	56.0	.0	132.0	210.2
1999-12	-2.5	83.2	68.6	3.0	4.2	4.2	.0	67.4	11.6	132.0	293.4
2000- 1	-7.7	49.2	29.0	11.6	1.7	1.7	.0	38.9	20.2	132.0	342.6
2000- 2	-4.6	49.8	9.4	60.6	4.2	4.2	.0	65.8	.0	132.0	392.4
2000- 3	2.7	32.2	25.4	6.8	18.3	18.3	.0	13.9	.0	132.0	424.6
2000- 4	5.3	62.2	56.4	5.8	29.8	29.8	.0	32.4	.0	132.0	486.8
2000- 5	12.9	146.6	146.6	.0	79.9	79.9	.0	66.7	.0	132.0	633.4
2000- 6	16.4	211.8	211.8	.0	104.0	104.0	.0	107.8	.0	132.0	845.2
2000- 7	18.5	67.4	67.4	.0	119.1	119.1	.0	.0	.0	80.3	912.6
2000- 8	18.8	70.2	70.2	.0	112.6	112.6	.0	.0	.0	37.9	982.8
2000- 9	14.1	74.8	74.8	.0	72.8	72.8	.0	.0	.0	39.8	1057.6
2000-10	9.5	35.4	35.4	.0	43.5	39.5	-4.0	.0	.0	35.8	35.4
2000-11	2.0	55.2	43.8	11.4	12.4	12.4	.0	.0	.0	78.6	90.6
2000-12	-8.7	105.4	20.8	17.4	.5	.5	.0	.0	67.2	116.4	196.0
2001- 1	-6.1	42.2	2.8	.0	.0	.0	.0	.0	106.6	119.2	238.2
2001- 2	-6.0	81.8	24.0	16.0	.4	.4	.0	26.8	148.3	132.0	320.0
2001- 3	-2.3	41.6	.0	51.8	1.9	1.9	.0	49.9	138.1	132.0	361.6
2001- 4	6.8	33.4	33.4	138.1	36.4	36.4	.0	135.1	.0	132.0	395.0
2001- 5	13.7	122.8	122.8	.0	84.7	84.7	.0	38.1	.0	132.0	517.8
2001- 6	18.4	54.8	54.8	.0	117.1	117.1	.0	.0	.0	69.7	572.6

Burketon McLaughlin WATER BUDGET VALUES FOR THE PERIOD 1969-2001
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YR	MO	TEMP(C)	PCPN	RAIN	MELT	PE	AE	DEF	SURP	SNOW	SOIL	ACC P
2001-	7	19.0	37.6	37.6	.0	122.8	107.3	-15.5	.0	.0	.0	610.2
2001-	8	21.4	86.4	86.4	.0	129.2	86.4	-42.8	.0	.0	.0	696.6
2001-	9	14.9	144.0	144.0	.0	77.0	77.0	.0	.0	.0	67.0	840.6
2001-	10	8.7	147.4	147.4	.0	40.3	40.3	.0	42.1	.0	132.0	147.4
2001-	11	5.6	118.0	118.0	.0	21.2	21.2	.0	96.8	.0	132.0	265.4
2001-	12	-.3	54.8	41.0	4.8	6.4	6.4	.0	39.4	9.0	132.0	320.2

APPENDIX VII –IDF DATA



1. EQUATIONS FOR TYPICAL INTENSITY - DURATION - FREQUENCY CURVES T - TIME (MINUTES)
I - INTENSITY (mm/hr)

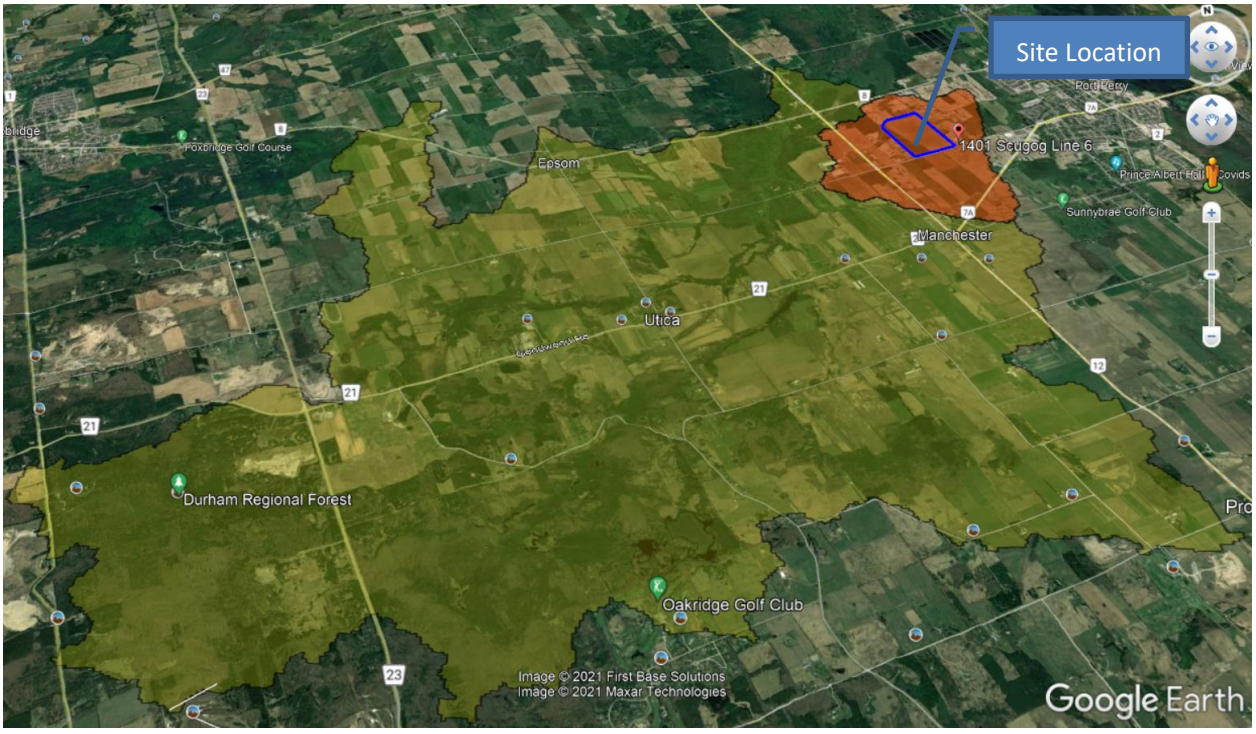
$$I_2 = \frac{645}{(T+5)^{0.786}} \quad , \quad I_5 = \frac{\quad}{(T+\quad)^{0.788}} \quad , \quad I_{10} = \frac{\quad}{(T+5)^{0.788}} \quad , \quad I_{25} = \frac{1243}{(T+4)^{0.787}} \quad , \quad I_{100} = \frac{1799}{(T+5)^{0.810}}$$

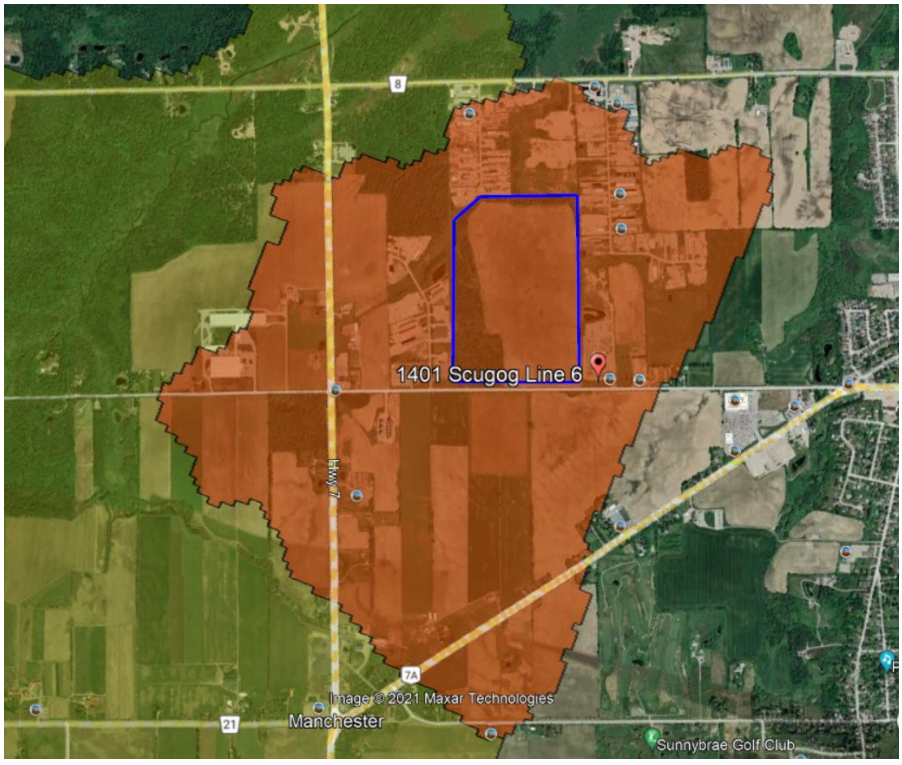
2. THE ABOVE EQUATIONS ARE ONLY VALID FOR T = 10 MINUTES TO 1440 MINUTES.

REVISION DATE OF REVISION	TOWNSHIP OF SCUGOG RAINFALL INTENSITY DURATION CURVES	DATE OF ISSUE APRIL 1990 DRAWING No. SS-600

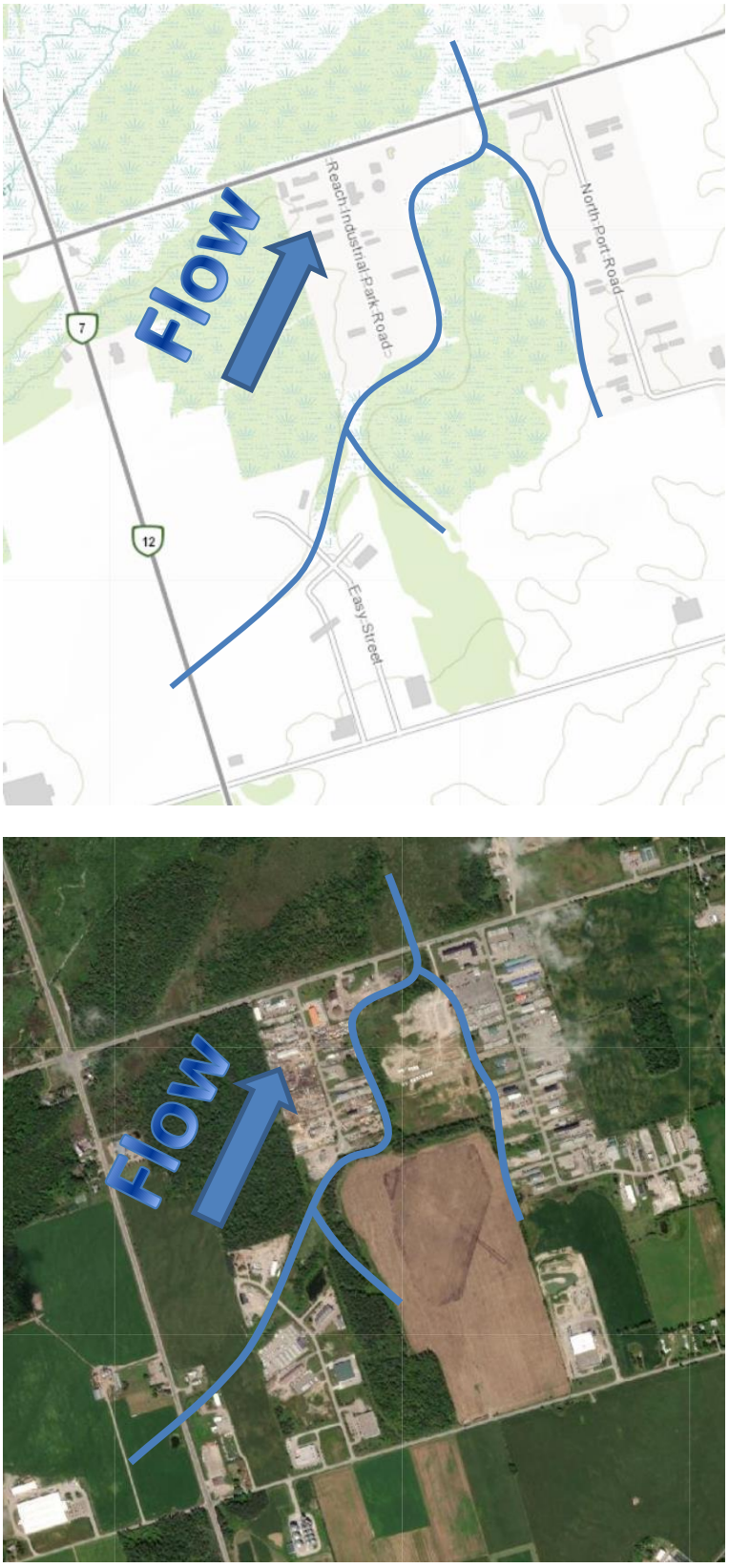
APPENDIX VIII –SUBWATERSHEDS WHITHIN THE SITE

Watershed Calculations





Overview of Watersheds and Property Boundary



Identified Water courses in the site

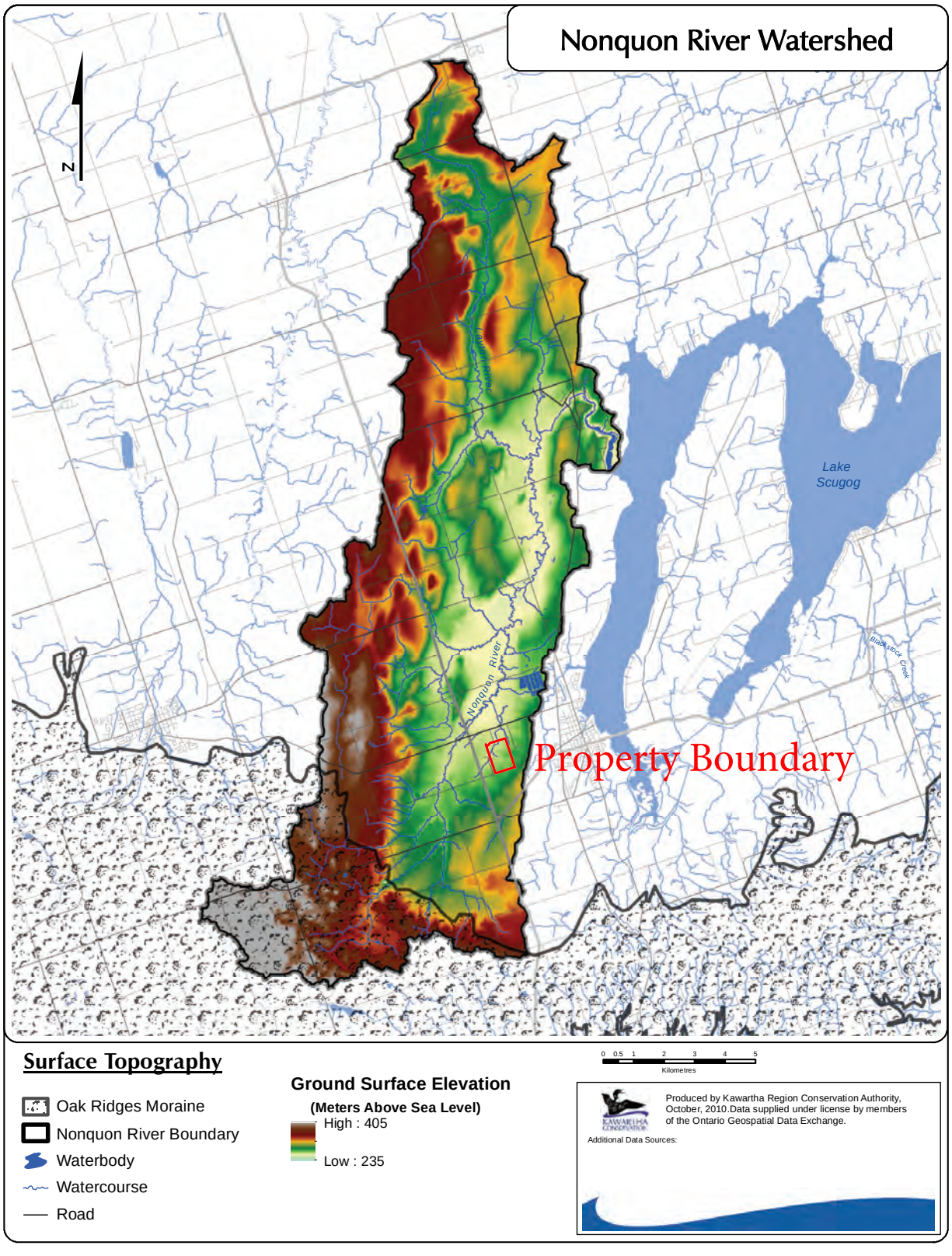
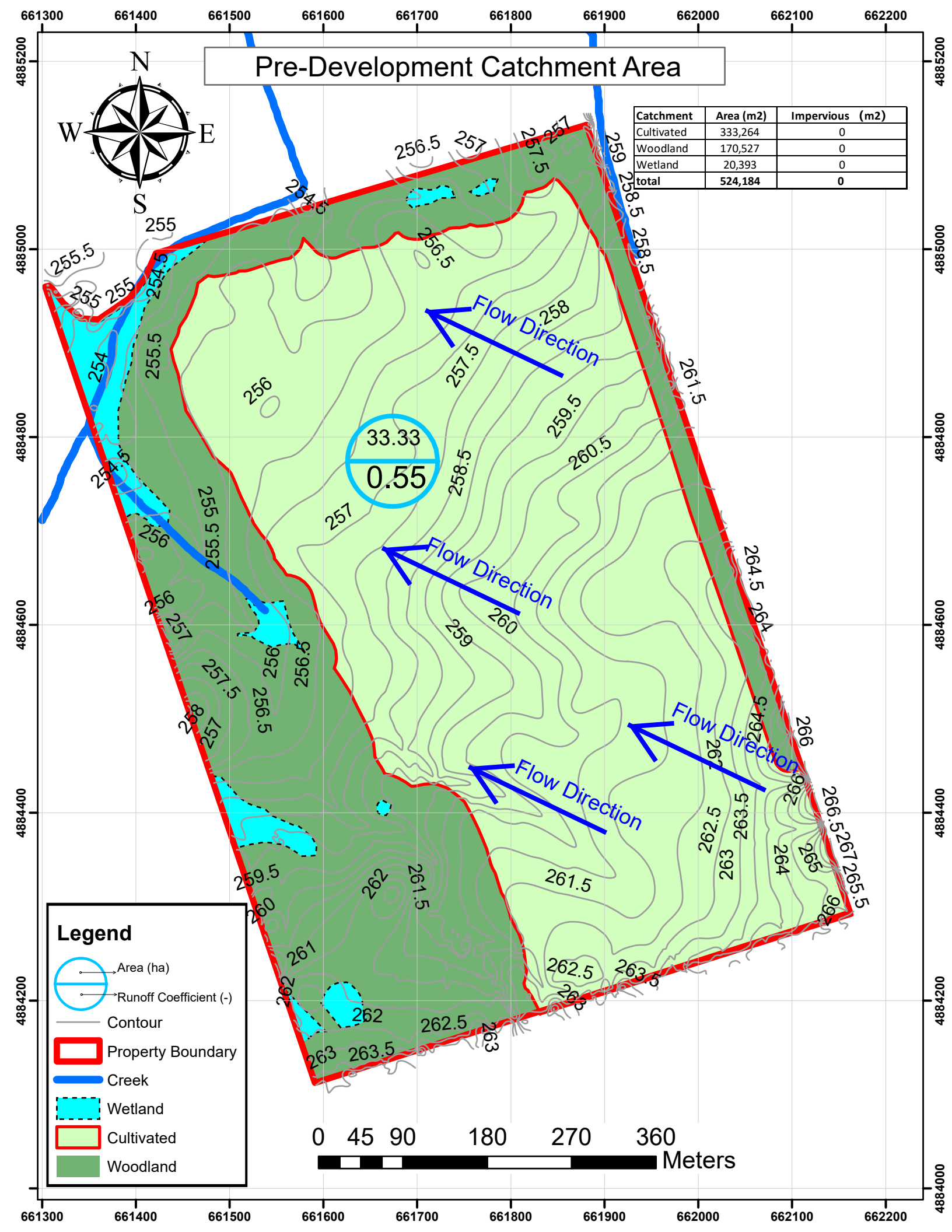
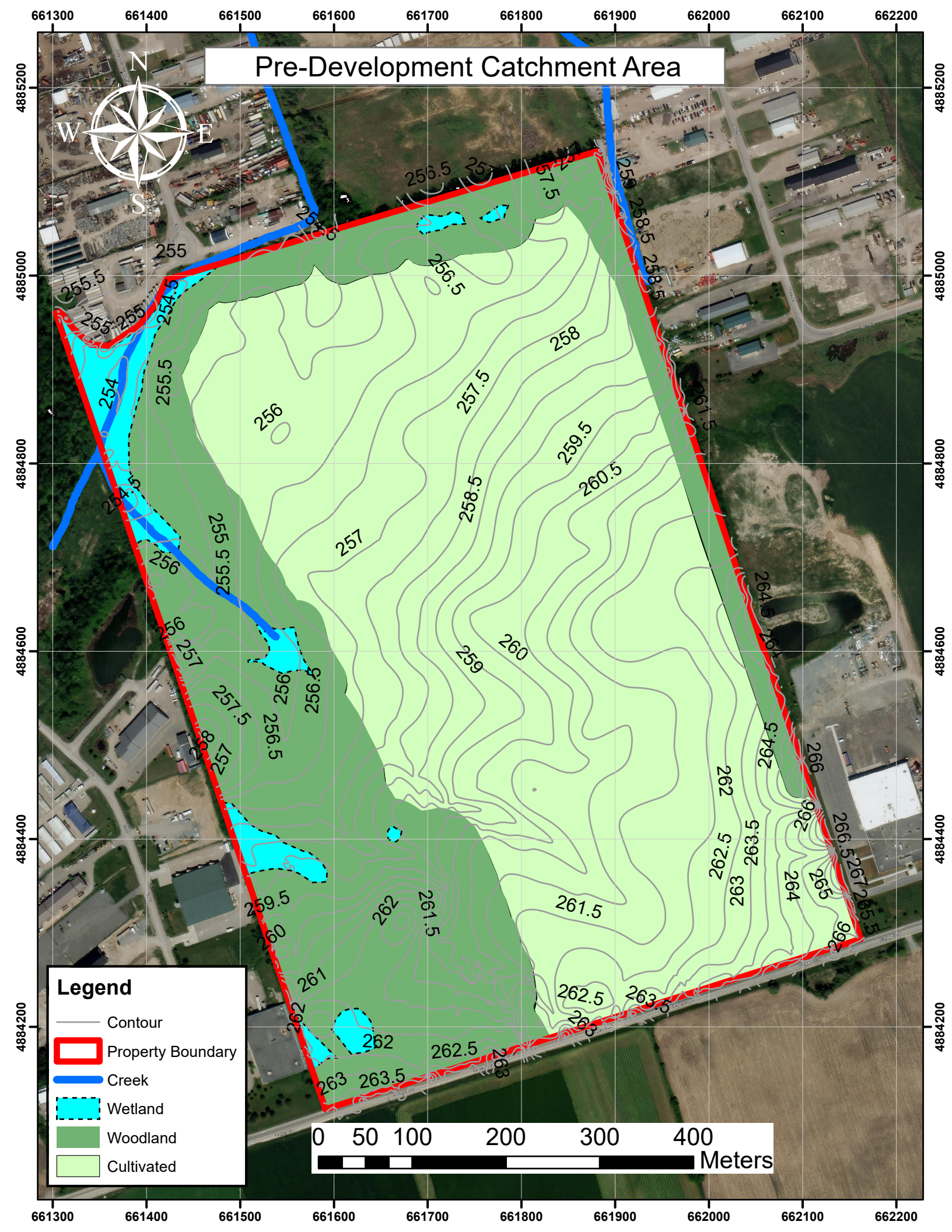


Figure 2.6: Surface topography.

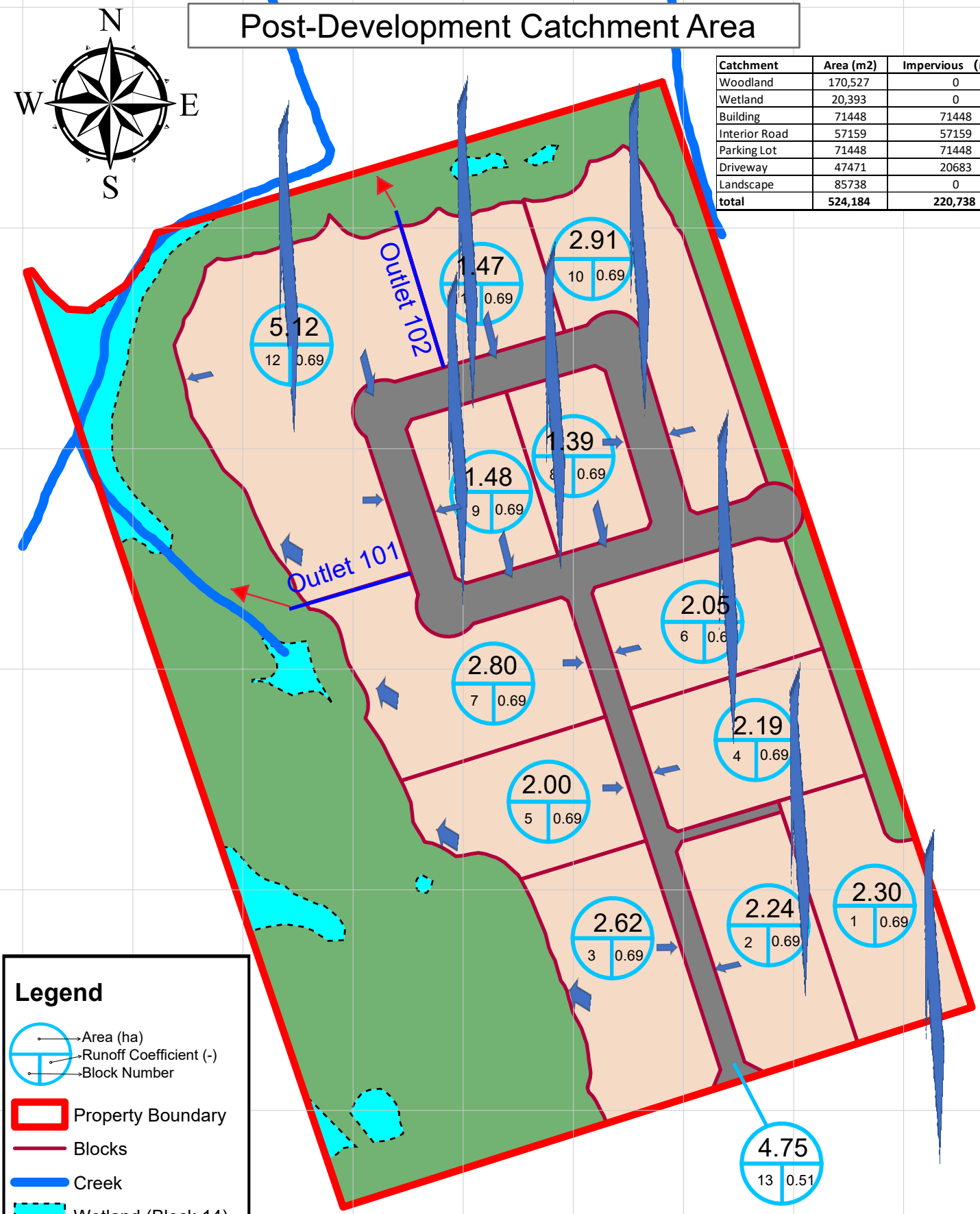
APPENDIX IX –PRE- AND POST-DEVELOPMENT CATCHEMENT AREA


Pre-Development Catchment Area
N
S
E
W
Catchment | Area (m2) | Impervious (m2)
Cultivated | 333,264 | 0
Woodland | 170,527 | 0
Wetland | 20,393 | 0
total | 524,184 | 0
33.33
0.55
Flow Direction
Flow Direction
Flow Direction
Flow Direction
Legend
Area (ha)
Runoff Coefficient (-)
Contour
Property Boundary
Creek
Wetland
Cultivated
Woodland
0 45 90 180 270 360
Meters
661300 661400 661500 661600 661700 661800 661900 662000 662100 662200
4885200 4885000 4884800 4884600 4884400 4884200 4884000


Pre-Development Catchment Area
N
S
E
W
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
Legend
Contour
Property Boundary
Creek
Wetland
Woodland
Cultivated
0 50 100 200 300 400
Meters

Post-Development Catchment Area

Catchment	Area (m2)	Impervious (m2)
Woodland	170,527	0
Wetland	20,393	0
Building	71448	71448
Interior Road	57159	57159
Parking Lot	71448	71448
Driveway	47471	20683
Landscape	85738	0
total	**524,184**	**220,738**



Legend

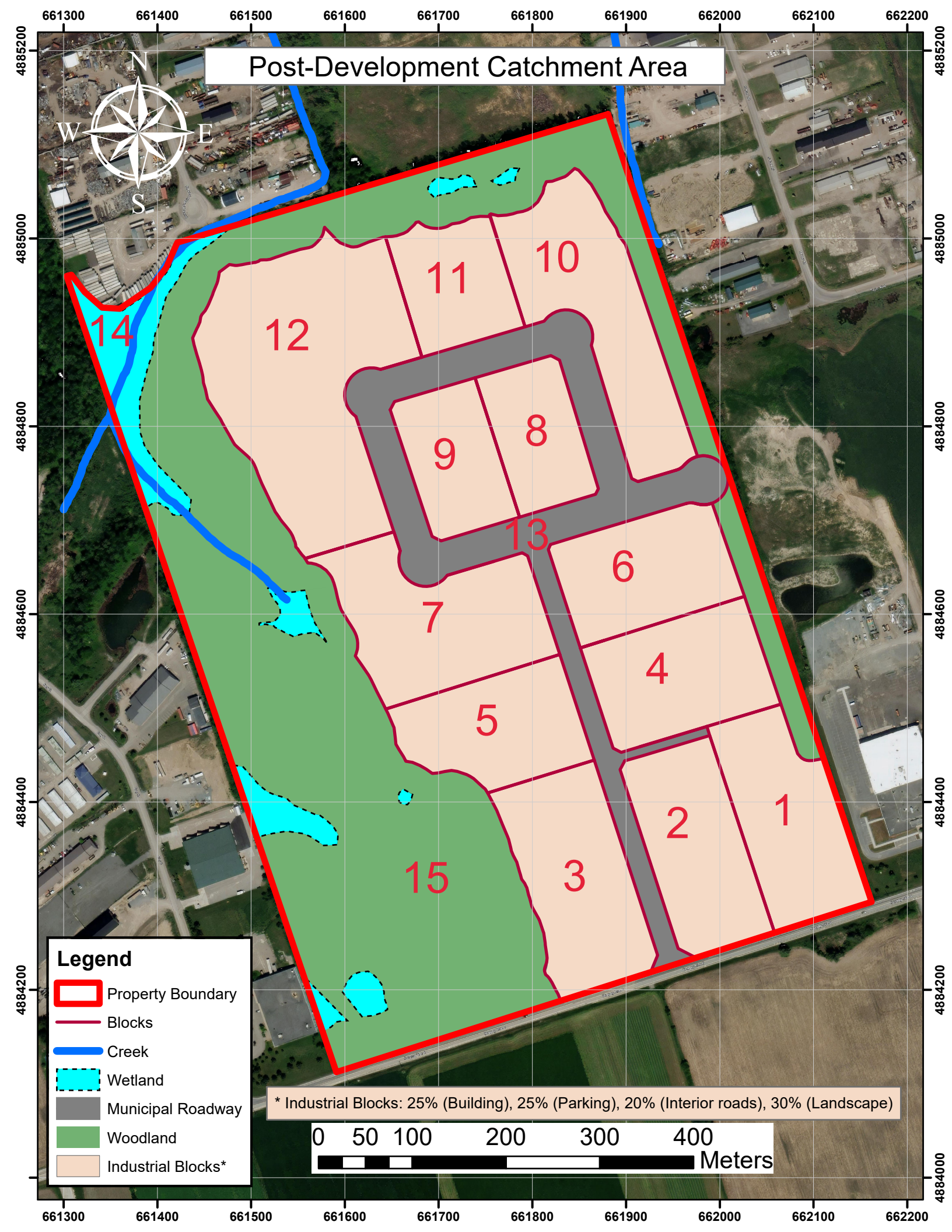
- Area (ha)
- Runoff Coefficient (-)
- Block Number
- Property Boundary
- Blocks
- Creek
- Wetland (Block 14)
- Municipal Roadway
- Woodland (Block 15)
- Industrial Blocks*

* Industrial Blocks: 25% (Building), 25% (Parking), 20% (Interior roads), 30% (Landscape)

0 50 100 200 300 400 Meters

661300 661400 661500 661600 661700 661800 661900 662000 662100 662200

4885200 4885000 4884800 4884600 4884400 4884200 4884000


Post-Development Catchment Area
N
S
E
W
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
4884000
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
Legend
Property Boundary
Blocks
Creek
Wetland
Municipal Roadway
Woodland
Industrial Blocks*
* Industrial Blocks: 25% (Building), 25% (Parking), 20% (Interior roads), 30% (Landscape)
0 50 100 200 300 400
Meters

APPENDIX X –WATER BALANCE CALCULATIONS

WATER BUDGET- PRE-DEVELOPMENT

WATER BALANCE/WATER BUDGET ASSESSMENT

Catchment Designation	Site			
	Cultivated	Woodland	Wetland	Total
Area (m^2)	333,264	170,527	20,393	524,184
Pervious Area (m^2)	333,264	170,527	20,393	524,184
Impervious Area (m^2)	0	0	0	0
Infiltration Factors				
Topography Infiltration Factor	0.2	0.2	0.2	
Soil Infiltratin Factor	0.2	0.2	0.2	
Land Cover Infiltration Factor	0.1	0.2	0.2	
MOE Infiltration Factor	0.5	0.6	0.6	
Actual Infiltratin Factor	0.5	0.6	0.6	
Run-Off Coefficient	0.5	0.4	0.4	
Runoff From Impervious Surfaces *	0	0	1	
Inputs (per unit area)				
Precipitation (mm/yr)	898	898	898	898
Run-On (mm/yr)	0	0	0	0
Other Inputs (mm/yr)	0	0	0	0
Total Inputs (mm/yr)	**898**	**898**	**898**	**898**
Outputs (per unit area)				
Precipitation Surplus (mm/yr)	331	331	331	331
Net Surplus (mm/yr)	331	331	331	331
Evapotranspiratin (mm/yr)	567	567	567	567
Infiltration (mm/yr)	165.5	198.6	198.6	178
Rooftop Infiltration (mm/yr)	0	0	0	0
Total Infiltration (mm/yr)	165.5	198.6	198.6	178
Runoff Pervious Area	165.5	132.4	132.4	153
Runoff Impervious Area	0	0	0	0
Total Runoff (mm/yr)	165.5	132.4	132.4	153
Total Outputs (mm/yr)	898	898	898	898
Difference (Inputs-Outputs)	0	0	0	0
Inputs (Volumes)				
Precipitaiton (m^3/yr)	299,271	153,133	18,313	470,717
Run-On (m^3/yr)	0	0	0	0
Other Inputs (m^3/yr)	0	0	0	0
Total Inputs (m^3/yr)	299,271	153,133	18,313	470,717
Outputs (Volumes)				
Precipitation Surplus (m^3/yr)	110,310	56,444	6,750	173,505
Net Surplus (m^3/yr)	110,310	56,444	6,750	173,505
Evapotranspiratin (m^3/yr)	188,961	96,689	11,563	297,212
Infiltration (m^3/yr)	55,155	33,867	4,050	93,072
Rooftop Infiltration (m^3/yr)	0	0	0	0
Total Infiltration (m^3/yr)	55,155	33,867	4,050	93,072
Runoff Pervious Area (m^3/yr)	55,155	22,578	2,700	80,433
Runoff Impervious Area (m^3/yr)	0	0	0	0
Total Runoff (m^3/yr)	55,155	22,578	2,700	80,433
Total Outputs (m^3/yr)	299,271	153,133	18,313	470,717
Difference (Inputs-Outputs)	0	0	0	0

WATER BUDGET- POST-DEVELOPMENT

WATER BALANCE/WATER BUDGET ASSESSMENT

Catchment Designation	Site							
	Building	Roadway	Interior Road	Parking	Landscape	Woodland	Wetland	Total
Area (m^2)	71,448	47,471	57,159	71,448	85,738	170,527	20,393	524,184
Pervious Area (m^2)	0	26,788	0	0	85,738	170,527	20,393	303,446
Impervious Area (m^2)	71,448	20,683	57,159	71,448	0	0	0	220,738
Infiltration Factors								
Topography Infiltration Factor	0	0.2	0	0	0.3	0.2	0.2	
Soil Infiltratin Factor	0	0.2	0	0	0.2	0.2	0.2	
Land Cover Infiltration Factor	0	0.1	0	0	0.15	0.2	0.2	
MOE Infiltration Factor	0	0.5	0	0	0.65	0.6	0.6	
Actual Infiltratin Factor	0	0.5	0	0	0.65	0.6	0.6	
Run-Off Coefficient	1	0.5	1	1	0.35	0.4	0.4	
Runoff From Impervious Surfaces *	0.95	0.95	0.95	0.95	0	0	0	
Inputs (per unit area)								
Precipitation (mm/yr)	898	898	898	898	898	898	898	898
Run-On (mm/yr)	0	0	0	0	0	0	0	0
Other Inputs (mm/yr)	0	0	0	0	0	0	0	0
Total Inputs (mm/yr)	**898**	**898**	**898**	**898**	**898**	**898**	**898**	**898**
Outputs (per unit area)								
Precipitation Surplus (mm/yr)	853.1	558.48	853.1	853.1	331	331	331	551
Net Surplus (mm/yr)	853.1	558.48	853.1	853.1	331	331	331	551
Evapotranspiratin (mm/yr)	44.9	339.52	44.9	44.9	567	567	567	347
Infiltration (mm/yr)	0	93.39	0	0	215.15	198.6	198.6	116
Rooftop Infiltration (mm/yr)	0	0	0	0	0	0	0	0
Total Infiltration (mm/yr)	0	93.39	0	0	215.15	198.6	198.6	116
Runoff Pervious Area	0	93.39	0	0	115.85	132.4	132.4	124
Runoff Impervious Area	853.1	371.69	853.1	853.1	0	0	0	808
Total Runoff (mm/yr)	853.1	465.09	853.1	853.1	115.85	132.4	132.4	435
Total Outputs (mm/yr)	898	898.00	898	898	898	898	898	898
Difference (Inputs-Outputs)	0	0	0	0	0	0	0	0
Inputs (Volumes)								
Precipitaiton (m^3/yr)	64,160	42,629	51,329	64,160	76,993	153,133	18,313	470,717
Run-On (m^3/yr)	0	0	0	0	0	0	0	0
Other Inputs (m^3/yr)	0	0	0	0	0	0	0	0
Total Inputs (m^3/yr)	64,160	42,629	51,329	64,160	76,993	153,133	18,313	470,717
Outputs (Volumes)								
Precipitation Surplus (m^3/yr)	60,952	26,511	48,762	60,952	28,379	56,444	6,750	288,752
Net Surplus (m^3/yr)	60,952	26,511	48,762	60,952	28,379	56,444	6,750	288,752
Evapotranspiratin (m^3/yr)	3,208	16,117	2,566	3,208	48,613	96,689	11,563	181,965
Infiltration (m^3/yr)	0	4,433	0	0	18,447	33,867	4,050	60,797
Rooftop Infiltration (m^3/yr)	0	0	0	0	0	0	0	0
Total Infiltration (m^3/yr)	0	4,433	0	0	18,447	33,867	4,050	60,797
Runoff Pervious Area (m^3/yr)	0	2,502	0	0	9,933	22,578	2,700	37,712
Runoff Impervious Area (m^3/yr)	60,952	7,688	48,762	60,952	0	0	0	178,355
Total Runoff (m^3/yr)	60,952	10,190	48,762	60,952	9,933	22,578	2,700	216,067
Total Outputs (m^3/yr)	64,160	42,629	51,329	64,160	76,993	153,133	18,313	470,717
Difference (Inputs-Outputs)	0	0	0	0	0	0	0	0

* Based on the Design Chart 1.07 (MTO, 1997), the runoff coefficients for rooftop and pavement are 0.7 - 0.95 and 0.8 - 0.95, respectively. We used the highest ratio (95%), as the grades of the pavement is generally 1-2% and would run into the side swales, and we have steep rooftops.

WATER BUDGET- POST-DEVELOPMENT WITH MITIGATION

WATER BALANCE/WATER BUDGET ASSESSMENT

Catchment Designation	Site							
	Building	Roadway	Interior Road	Parking	Landscape	Woodland	Wetland	Total
Area (m^2)	71,448	47,471	57,159	71,448	85,738	170,527	20,393	524,184
Pervious Area (m^2)	0	26,788	0	0	85,738	170,527	20,393	303,446
Impervious Area (m^2)	71,448	20,683	57,159	71,448	0	0	0	220,738
Infiltration Factors								
Topography Infiltration Factor	0	0.2	0	0	0.3	0.2	0.2	
Soil Infiltratin Factor	0	0.2	0	0	0.2	0.2	0.2	
Land Cover Infiltration Factor	0	0.1	0	0	0.15	0.2	0.2	
MOE Infiltration Factor	0	0.5	0	0	0.65	0.6	0.6	
Actual Infiltratin Factor	0	0.5	0	0	0.65	0.6	0.6	
Run-Off Coefficient	1	0.5	1	1	0.35	0.4	0.4	
Runoff From Impervious Surfaces *	0.95	0.95	0.95	0.95	0	0	0	
Inputs (per unit area)								
Precipitation (mm/yr)	898	898	898	898	898	898	898	898
Run-On (mm/yr)	0	0	0	0	0	0	0	0
Other Inputs (mm/yr)	0	0	0	0	0	0	0	0
Total Inputs (mm/yr)	**898**	**898**	**898**	**898**	**898**	**898**	**898**	**898**
Outputs (per unit area)								
Precipitation Surplus (mm/yr)	853.1	558.48	853.1	853.1	331	331	331	551
Net Surplus (mm/yr)	853.1	558.48	853.1	853.1	331	331	331	551
Evapotranspiratin (mm/yr)	44.9	339.52	44.9	44.9	567	567	567	347
Infiltration (mm/yr)	0	93.39	725.135	725.135	215.15	198.6	198.6	294
Rooftop Infiltration (mm/yr)	682.48	0	0	0	0	0	0	93
Total Infiltration (mm/yr)	682.48	93.39	725.135	725.135	215.15	198.6	198.6	387
Runoff Pervious Area	0	93.39	0	0	115.85	132.4	132.4	124
Runoff Impervious Area	170.62	371.69	127.965	127.965	0	0	0	165
Total Runoff (mm/yr)	170.62	465.09	127.965	127.965	115.85	132.4	132.4	164
Total Outputs (mm/yr)	898	898	898	898	898	898	898	898
Difference (Inputs-Outputs)	0	0	0	0	0	0	0	0
Inputs (Volumes)								
Precipitaiton (m^3/yr)	64,160	42,629	51,329	64,160	76,993	153,133	18,313	470,717
Run-On (m^3/yr)	0	0	0	0	0	0	0	0
Other Inputs (m^3/yr)	0	0	0	0	0	0	0	0
Total Inputs (m^3/yr)	64,160	42,629	51,329	64,160	76,993	153,133	18,313	470,717
Outputs (Volumes)								
Precipitation Surplus (m^3/yr)	60,952	26,511	48,762	60,952	28,379	56,444	6,750	288,752
Net Surplus (m^3/yr)	60,952	26,511	48,762	60,952	28,379	56,444	6,750	288,752
Evapotranspiratin (m^3/yr)	3,208	16,117	2,566	3,208	48,613	96,689	11,563	181,965
Infiltration (m^3/yr)	0	4,433	41,448	51,809	18,447	33,867	4,050	154,054
Rooftop Infiltration (m^3/yr)	48,762	0	0	0	0	0	0	48,762
Total Infiltration (m^3/yr)	48,762	4,433	41,448	51,809	18,447	33,867	4,050	202,816
Runoff Pervious Area (m^3/yr)	0	2,502	0	0	9,933	22,578	2,700	37,712
Runoff Impervious Area (m^3/yr)	12,190	7,688	7,314	9,143	0	0	0	36,335
Total Runoff (m^3/yr)	12,190	10,190	7,314	9,143	9,933	22,578	2,700	74,048
Total Outputs (m^3/yr)	64,160	42,629	51,329	64,160	76,993	153,133	18,313	470,717
Difference (Inputs-Outputs)	0	0	0	0	0	0	0	0

* Based on the Design Chart 1.07 (MTO, 1997), the runoff coefficients for rooftop and pavement are 0.7 - 0.95 and 0.8 - 0.95, respectively. We used the highest ratio (95%), as the grades of the pavement is generally 1-2% and would run into the side swales, and we have steep rooftops.

** Considering that 75% of the captured runoff by the Bioretension LIDs in the western and northern blocks enters the wetland as a sheet flow, it was assumed that only 80% of the net surplus from parking lots and interior roads infiltrates into the ground by Bioretention LID. Similarly, it can be assumed that only 80% of the runoff from the rooftop gets a chance to infiltrate into the ground due to a low infiltration potential in the lower layers and the rest is discharged to the outlet due to the overflow to the roadside Bioswales.

WATER BUDGET SUMMARY

WATER BALANCE/WATER BUDGET ASSESSMENT

Characterstic	Site				
	Pre-Development	Post-Development	Change (Pre- to Post-)	Post-Development with Mitigation	Change (Pre- to Post- with Mitigation)
Inputs (Volumes)					
Precipitaiton (m^3/yr)	470,717	470,717	0.0%	470,717	0.0%
Run-On (m^3/yr)	0	0	0.0%	0	0.0%
Other Inputs (m^3/yr)	0	0	0.0%	0	0.0%
Total Inputs (m^3/yr)	**470,717**	470,717	0.0%	470,717	0.0%
Outputs (Volumes)					
Precipitation Surplus (m^3/yr)	173,505	288,752	66.4%	288,752	66.4%
Net Surplus (m^3/yr)	173,505	288,752	66.4%	288,752	66.4%
Evapotranspiratin (m^3/yr)	297,212	181,965	-38.8%	181,965	-38.8%
Infiltration (m^3/yr)	93,072	60,797	-34.7%	154,054	65.5%
Rooftop Infiltration (m^3/yr)	0	0	0.0%	48,762	0.0%
Total Infiltration (m^3/yr)	93,072	60,797	-34.7%	202,816	117.9%
Runoff Pervious Area (m^3/yr)	80,433	37,712	-53.1%	37,712	-53.1%
Runoff Impervious Area (m^3/yr)	0	178,355	0.0%	36,335	0.0%
Total Runoff (m^3/yr)	80,433	216,067	168.6%	74,048	-7.9%
Total Outputs (m^3/yr)	470,717	470,717	0.0%	470,717	0.0%

APPENDIX XI –PLDT (NVCA P tool) OUTPUT RESULTS

Development Export Summary

Development :1401 Scugog Line 6

Pre-Development Phosphorus Export

DEVELOPMENT : 1401 Scugog Line 6

Landuse		Area (ha)	P coeff (kg/ha)	Pload (kg/yr)
Agricultural				
Cropland	Block	33.32		37.63
	Agricultural Land use Class Total :	**33.32**		**37.63**
Natural Heritage				
Forest		17.05	0.06	1.02
Wetland		2.04	0.05	0.10
	Natural Heritage Land use Class Total :	**19.09**		**1.12**
	Development Total :	**52.41**		**38.75**

Cropland Site Sediment & Phosphorus Pre-Development Export

DEVELOPMENT : 1401 Scugog Line 6

COLOUR KEY : Site Specific Input | Constant / Lookup | Calculation

SubArea : Block

Slope Area (ha)	33.32	R (rainfall / runoff for Lake Simcoe)	90.00
Surface Slope Gradient (%)	1.25	K (soil errodability factor)	0.54
Length of Slope (m)	1,005.00	NN (determined by slope)	0.30
Cropt Type Factor)	0.50	LS (slope length gradient factor)	0.42
Tillage Type Factor	0.60	C (crop management factor)	0.30
		P (prevention + capture)	1.00
		Soil Loss (kg/year)	6.06
		Phosphorus export (kg/ha/yr)	1.13
		Phosphorus load (kg/yr)	37.63

PRE Developed Area (ha) : **33.32**

Phosphorus export (kg/ha/yr) : **1.13**

Phosphorus load (kg/yr) : **37.63**

Updated : Sept 2014

Post-Development Phosphorus Export

DEVELOPMENT : 1401 Scugog Line 6

Landuse	Area (ha)	P coeff (kg/ha)	Pload (kg/yr)
Natural Heritage			
Forest	17.05	0.06	1.02
Wetland	2.04	0.05	0.10
Natural Heritage Land use Class Total :	**19.09**		**1.12**
Urban			
Industrial	28.58	0.41	53.26
Transportation	4.74	0.50	6.29
Urban Land use Class Total :	**33.32**		**59.55**
Development Total :	**52.41**		**60.67**

1/8/2024 Page 1 of 1

Updated : Sept 2014

Cropland Site Sediment & Phosphorus Post-Development Export

DEVELOPMENT : 1401 Scugog Line 6

COLOUR KEY : Site Specific Input | Constant / Lookup | Calculation

SubArea :

Slope Area (ha)
Surface Slope Gradient (%)
Length of Slope (m)

Cropt Type Factor)
Tillage Type Factor

R (rainfall / runoff for Lake Simcoe)
K (soil errodability factor)
NN (determined by slope)

LS (slope length gradient factor)
C (crop management factor)
P (prevention + capture)
Soil Loss (kg/year)
Phosphorus export (kg/ha/yr)
Phosphorus load (kg/yr)

PRE Developed Area (ha) :

Phosphorus export (kg/ha/yr) :

Phosphorus load (kg/yr) :

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Updated : Sept 2014

Post Dev BMP

Area (ha)	Treated Area %	P coefficient	P coefficient	P Load Reduction (kg/yr)	Rationale
Best Management Practices (BMP) Applied (and Rationale)					
Transportation					
33.30	100	0.50	100 %	16.65	Enhanced Grass swale with Check dams, enough capacity and overflow
Enhanced Grass/Water Quality Swales					
Industrial					
28.60	100	0.41	100 %	11.73	Bioretention LID with enough capacity and overflow
Bioretention Systems					
Transportation					
33.30	100	0.50	65 %	10.82	Existing wetland with vegetated filter strips
Vegetated Filter Strips/Stream Buffers					

Updated : Sept 2014

Development Area P and BMP Summary

Total PreDevelopment Area (ha):	52.41
PreDevelopment Area excluding Wetlands (ha):	50.37
Total PostDevelopment Area (ha):	52.41
Total Area treated by BMP's (ha):	95.20
Treated Area total:	95.20
Total PreDevelopment Load (kg/yr):	38.75
Total PostDevelopment Load (kg/yr):	60.67
Total P Load Reduction with BMP's (kg/yr):	39.20
Minimum P Load Reduction Required:	21.92
Total PostDevelopment Load with BMP's (kg/yr)	21.47

Conclusion : No Net Increase in P Load.

Updated : Sept 2014

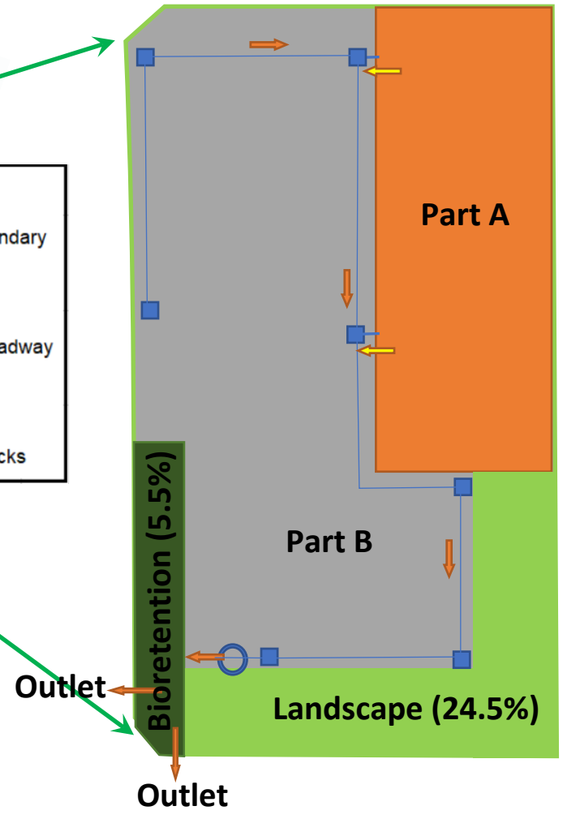
Post Dev Construction

Sediment Control Measures - Perimeter Controls	
Sediment/Silt Fence	To be used surrounding the construction area
Vehicle Tracking Control/Mud Mat	To be installed at the entrance of the site
Silt Soxx	to be used behind silt fence to trap sediment

APPENDIX XII –SWM FLOWCHART

Stormwater Management Flowchart/Diagram





Site Impervious Areas Layout

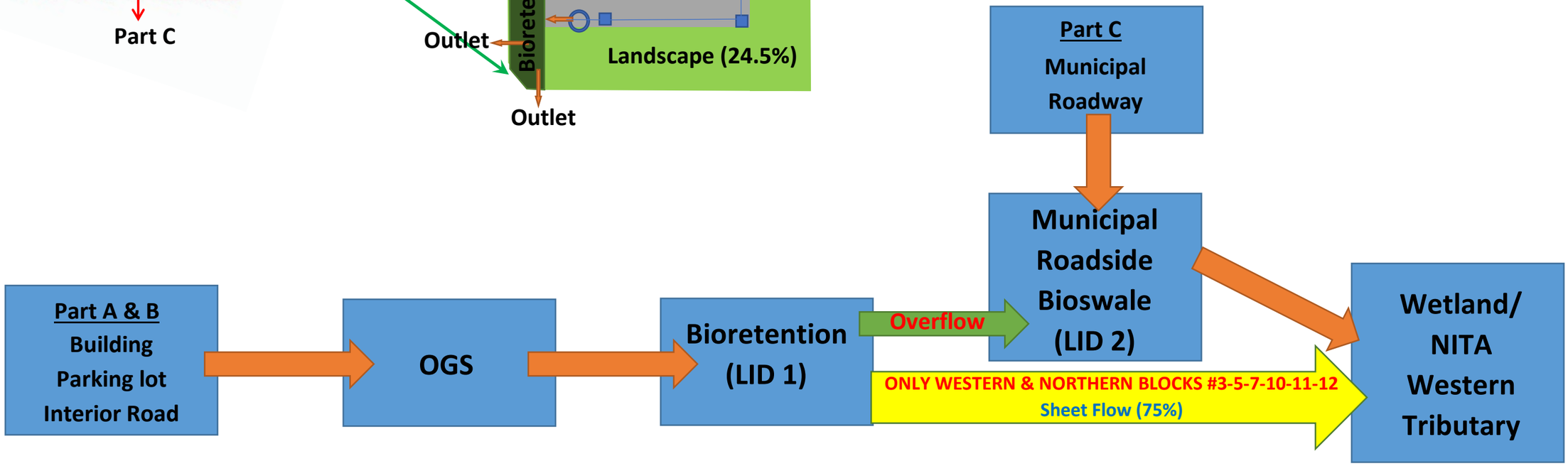
Part A: Building (25%)

Part B: Parking Lot (25%) & Interior Roads (20%)

Part C: Municipal Roadway

Catchbasin (CB)

Oil Grit Separator (OGS)



APPENDIX XIII –STORMWATER MANAGEMENT CALCULATIONS

STORMWATER MANAGEMENT CALCULATIONS

Pre-Development Peak Flows – Blocks 1 - 13

Township of Scugog — Modified Rational Method

Storm (yrs)	Coeff A	Coeff B	Coeff C
2	645	5	0.786
5	904	5	0.788
10	1065	5	0.788
25	1243	4	0.787
100	1540	4.5	0.8

Q= CaCIA/360

Where:
Q- Flow Rate (m3/s)
Ca- Peaking Coefficient
C- Rational Method Runoff Coefficient
I- Storm Intensity (mm/hr)
A- Area (ha)

Block #	1	2	3	4	5	6
Area	2.30 ha	2.24 ha	2.62 ha	2.19 ha	2.00 ha	2.05 ha
Runoff Coefficient	0.55	0.55	0.55	0.55	0.55	0.55
Time of Concentration	12.0 min	12.8 min	12.7 min	10.6 min	11.2 min	10.7 min
Return Rate	2 year	2 year	2 year	2 year	2 year	2 year
Peaking Coefficient	1.0	1.0	1.0	1.0	1.0	1.0
Rainfall Intensity	69.64 mm/hr	67.16 mm/hr	67.33 mm/hr	74.43 mm/hr	72.33 mm/hr	73.88 mm/hr
Pre-Development Peak Flow	245.2 l/s	230.2 l/s	269.2 l/s	249.2 l/s	221.4 l/s	230.9 l/s
Return Rate	5 year	5 year	5 year	5 year	5 year	5 year
Peaking Coefficient	1.0	1.0	1.0	1.0	1.0	1.0
Rainfall Intensity	97.06 mm/hr	93.59 mm/hr	93.82 mm/hr	103.74 mm/hr	100.81 mm/hr	102.97 mm/hr
Pre-Development Peak Flow	341.7 l/s	320.7 l/s	375.2 l/s	347.4 l/s	308.6 l/s	321.9 l/s
Return Rate	10 year	10 year	10 year	10 year	10 year	10 year
Peaking Coefficient	1.0	1.0	1.0	1.0	1.0	1.0
Rainfall Intensity	114.34 mm/hr	110.26 mm/hr	110.53 mm/hr	122.22 mm/hr	118.77 mm/hr	121.31 mm/hr
Pre-Development Peak Flow	402.6 l/s	377.9 l/s	442.0 l/s	409.3 l/s	363.6 l/s	379.2 l/s
Return Rate	25 year	25 year	25 year	25 year	25 year	25 year
Peaking Coefficient	1.1	1.1	1.1	1.1	1.1	1.1
Rainfall Intensity	140.38 mm/hr	135.07 mm/hr	135.42 mm/hr	150.69 mm/hr	146.16 mm/hr	149.50 mm/hr
Pre-Development Peak Flow	543.7 l/s	509.2 l/s	595.7 l/s	555.1 l/s	492.2 l/s	514.1 l/s
Return Rate	100 year	100 year	100 year	100 year	100 year	100 year
Peaking Coefficient	1.25	1.25	1.25	1.25	1.25	1.25
Rainfall Intensity	163.69 mm/hr	157.58 mm/hr	157.98 mm/hr	175.51 mm/hr	170.32 mm/hr	174.15 mm/hr
Pre-Development Peak Flow	720.4 l/s	675.0 l/s	789.7 l/s	734.7 l/s	651.7 l/s	680.5 l/s

7	8	9	10	11	12	13
2.80 ha	1.39 ha	1.48 ha	2.92 ha	1.47 ha	5.12 ha	4.75 ha
0.55	0.55	0.55	0.55	0.55	0.55	0.55
14.6 min	10.0 min	10.0 min	15.6 min	10.0 min	17.6 min	40.6 min
2 year	2 year	2 year	2 year	2 year	2 year	2 year
1.0	1.0	1.0	1.0	1.0	1.0	1.0
62.25 mm/hr	76.76 mm/hr	76.76 mm/hr	59.88 mm/hr	76.76 mm/hr	55.68 mm/hr	32.03 mm/hr
266.2 l/s	163.2 l/s	173.3 l/s	267.0 l/s	172.0 l/s	435.9 l/s	232.3 l/s
5 year	5 year	5 year	5 year	5 year	5 year	5 year
1.0	1.0	1.0	1.0	1.0	1.0	1.0
86.73 mm/hr	107.01 mm/hr	107.01 mm/hr	83.42 mm/hr	107.01 mm/hr	77.55 mm/hr	44.54 mm/hr
370.8 l/s	227.5 l/s	241.6 l/s	371.9 l/s	239.8 l/s	607.1 l/s	323.1 l/s
10 year	10 year	10 year	10 year	10 year	10 year	10 year
1.0	1.0	1.0	1.0	1.0	1.0	1.0
102.18 mm/hr	126.06 mm/hr	126.06 mm/hr	98.27 mm/hr	126.06 mm/hr	91.36 mm/hr	52.48 mm/hr
436.9 l/s	268.0 l/s	284.6 l/s	438.1 l/s	282.5 cms	715.2 l/s	380.7 l/s
25 year	25 year	25 year	25 year	25 year	25 year	25 year
1.1	1.1	1.1	1.1	1.1	1.1	1.1
124.65 mm/hr	155.76 mm/hr	155.76 mm/hr	119.64 mm/hr	155.76 mm/hr	110.85 mm/hr	62.56 mm/hr
586.3 l/s	364.3 l/s	386.8 l/s	586.8 l/s	383.9 l/s	954.5 l/s	499.3 l/s
100 year	100 year	100 year	100 year	100 year	100 year	100 year
1.25	1.25	1.25	1.25	1.25	1.25	1.25
145.56 mm/hr	181.31 mm/hr	181.31 mm/hr	139.76 mm/hr	181.31 mm/hr	129.56 mm/hr	73.12 mm/hr
777.9 l/s	481.9 l/s	511.7 l/s	778.9 l/s	507.8 l/s	1267.8 l/s	663.1 l/s

Post-Development Peak Flows – Blocks 1 - 13

Storm (yrs)	Township of Scugog Coeff A	Coeff B	Coeff C
2	645	5	0.786
5	904	5	0.788
10	1065	5	0.788
25	1243	4	0.787
100	1799	5	0.81

Modified Rational Method
Q= CaCIA/360

Where:
Q- Flow Rate (m3/s)
Ca- Peaking Coefficient
C- Rational Method Runoff Coefficient
I- Storm Intensity (mm/hr)
A- Area (ha)

Block #	1	2	3	4	5	6
Area	2.30 ha	2.24 ha	2.62 ha	2.19 ha	2.00 ha	2.05 ha
Runoff Coefficient	0.69	0.69	0.69	0.69	0.69	0.69
Time of Concentration	13.1 min	14.4 min	12.7 min	10.6 min	10.5 min	10.6 min
Return Rate	2 year	2 year	2 year	2 year	2 year	2 year
Peaking Coefficient	1.0	1.0	1.0	1.0	1.0	1.0
Rainfall Intensity	66.32 mm/hr	62.78 mm/hr	67.33 mm/hr	74.43 mm/hr	74.99 mm/hr	74.46 mm/hr
Post-Development Peak Flow	292.9 l/s	269.9 l/s	337.8 l/s	312.7 l/s	288.0 l/s	292.0 l/s
Return Rate	5 year	5 year	5 year	5 year	5 year	5 year
Peaking Coefficient	1.0	1.0	1.0	1.0	1.0	1.0
Rainfall Intensity	92.42 mm/hr	87.47 mm/hr	93.82 mm/hr	103.74 mm/hr	104.53 mm/hr	103.79 mm/hr
Post-Development Peak Flow	408.2 l/s	376.1 l/s	470.7 l/s	435.8 l/s	401.4 l/s	407.0 l/s
Return Rate	10 year	10 year	10 year	10 year	10 year	10 year
Peaking Coefficient	1.0	1.0	1.0	1.0	1.0	1.0
Rainfall Intensity	108.87 mm/hr	103.04 mm/hr	110.53 mm/hr	122.22 mm/hr	123.15 mm/hr	122.28 mm/hr
Post-Development Peak Flow	480.9 l/s	443.0 l/s	554.5 l/s	513.4 l/s	472.9 l/s	479.5 l/s
Return Rate	25 year	25 year	25 year	25 year	25 year	25 year
Peaking Coefficient	1.1	1.1	1.1	1.1	1.1	1.1
Rainfall Intensity	133.28 mm/hr	125.76 mm/hr	135.42 mm/hr	150.69 mm/hr	151.92 mm/hr	150.77 mm/hr
Post-Development Peak Flow	647.6 l/s	594.8 l/s	747.4 l/s	696.4 l/s	641.8 l/s	650.4 l/s
Return Rate	100 year	100 year	100 year	100 year	100 year	100 year
Peaking Coefficient	1.25	1.25	1.25	1.25	1.25	1.25
Rainfall Intensity	172.57 mm/hr	163.07 mm/hr	175.26 mm/hr	194.34 mm/hr	195.87 mm/hr	194.44 mm/hr
Post-Development Peak Flow	952.8 l/s	876.4 l/s	1099.1 l/s	1020.6 l/s	940.2 l/s	953.2 l/s

7	8	9	10		11	12	13	
2.80 ha	1.39 ha	1.48 ha	1.25	1.66 ha	1.47 ha	5.12 ha	2.98	1.77 ha
0.69	0.69	0.69	0.69	0.69	0.69	0.69	0.51	0.51
			1	2			101	102
13.5 min	10.0 min	15.4 min	10.0	17.6 min	10.0 min	16.1 min	36.8	27.9 min
2 year	2 year	2 year	2	2 year	2 year	2 year	2	2 year
1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
65.02 mm/hr	76.76 mm/hr	60.33 mm/hr	76.76	55.71 mm/hr	76.76 mm/hr	58.66 mm/hr	34.29	41.40 mm/hr
348.7 l/s	204.8 l/s	170.9 l/s	184.5	177.2 l/s	215.8 l/s	576.1 l/s	144.8	103.6 l/s
5 year	5 year	5 year	5	5 year	5 year	5 year	5	5 year
1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
90.60 mm/hr	107.01 mm/hr	84.05 mm/hr	107.01	77.59 mm/hr	107.01 mm/hr	81.72 mm/hr	47.71	57.62 mm/hr
485.9 l/s	285.4 l/s	238.1 l/s	257.2	246.9 l/s	300.8 l/s	802.5 l/s	201.4	144.2 l/s
10 year	10 year	10 year	10	10 year	10 year	10 year	10	10 year
1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
106.73 mm/hr	126.06 mm/hr	99.02 mm/hr	126.06	91.41 mm/hr	126.06 mm/hr	96.27 mm/hr	56.20	67.88 mm/hr
572.5 l/s	336.3 l/s	280.5 l/s	303.0	290.8 l/s	354.4 l/s	945.5 l/s	237.3	169.9 l/s
25 year	25 year	25 year	25	25 year	25 year	25 year	25	25 year
1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1	1.1
130.51 mm/hr	155.76 mm/hr	120.61 mm/hr	155.76	110.91 mm/hr	155.76 mm/hr	117.09 mm/hr	67.11	81.45 mm/hr
770.0 l/s	457.0 l/s	375.8 l/s	411.8	388.2 l/s	481.6 l/s	1264.9 l/s	311.6	224.3 l/s
100 year	100 year	100 year	100	100 year	100 year	100 year	100	100 year
1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25	1.25
169.07 mm/hr	200.63 mm/hr	156.54 mm/hr	200.63	144.18 mm/hr	200.63 mm/hr	152.07 mm/hr	87.45	106.18 mm/hr
1133.6 l/s	669.0 l/s	554.2 l/s	602.8	573.4 l/s	705.0 l/s	1866.8 l/s	461.5	332.3 l/s

Allowable Release Rates

$$Q = P_{pre} - Q_U$$

Where:

Q = Allowable Post-Development Release Rate
Q_{PRE} = Pre-Development Flow
Q_U = Post-Development Uncontrolled Flow

Block 1

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.3	0.0	0.3
10	0.4	0.0	0.4
25	0.5	0.0	0.5
100	0.7	0.0	0.7

Block 2

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.3	0.0	0.3
10	0.4	0.0	0.4
25	0.5	0.0	0.5
100	0.7	0.0	0.7

Block 3

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.3	0.0	0.3
5	0.4	0.0	0.4
10	0.4	0.0	0.4
25	0.6	0.0	0.6
100	0.8	0.0	0.8

Block 4

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.3	0.0	0.3
10	0.4	0.0	0.4
25	0.6	0.0	0.6
100	0.7	0.0	0.7

Block 5

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.3	0.0	0.3
10	0.4	0.0	0.4
25	0.5	0.0	0.5
100	0.7	0.0	0.7

Block 6

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.3	0.0	0.3
10	0.4	0.0	0.4
25	0.5	0.0	0.5
100	0.7	0.0	0.7

Block 7

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.3	0.0	0.3
5	0.4	0.0	0.4
10	0.4	0.0	0.4
25	0.6	0.0	0.6
100	0.8	0.0	0.8

Block 8

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.2	0.0	0.2
10	0.3	0.0	0.3
25	0.4	0.0	0.4
100	0.5	0.0	0.5

Block 9

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.2	0.0	0.2
10	0.3	0.0	0.3
25	0.4	0.0	0.4
100	0.5	0.0	0.5

Block 10

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.3	0.0	0.3
5	0.4	0.0	0.4
10	0.4	0.0	0.4
25	0.6	0.0	0.6
100	0.8	0.0	0.8

Block 11

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.2	0.0	0.2
10	0.3	0.0	0.3
25	0.4	0.0	0.4
100	0.5	0.0	0.5

Block 12

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.4	0.0	0.4
5	0.6	0.0	0.6
10	0.7	0.0	0.7
25	1.0	0.0	1.0
100	1.3	0.0	1.3

Block 13 (Roadway)

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Qpre (cms)	Q_U (cms)	Q (cms)
2	0.2	0.0	0.2
5	0.3	0.0	0.3
10	0.4	0.0	0.4
25	0.5	0.0	0.5
100	0.7	0.0	0.7

Quantity Control Volume Calculations

Block 1

Drainage Area (ha)	**2.30**
Runoff Coeff. (C)	**0.69**
Time of Concentration	13.1 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.7
Max. Storage Required	**259.3**

Results		
Storm Event	**Storage**	**Time**
(yr)	**(m3)**	**(min)**
2	82.5	6
5	96	6
10	113.1	6
25	162.2	6
100	259.3	7

Required Storage Volumes (2yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.702	42.1	14.7	27.4
2	139.74	0.622	74.7	29.4	45.2
3	125.81	0.560	100.8	44.1	56.7
4	114.69	0.511	122.6	58.8	63.7
5	105.57	0.470	141.0	73.6	67.5
6	106.56	0.474	170.8	88.3	82.5
7	98.88	0.440	184.9	103.0	81.9
8	92.36	0.411	197.4	117.7	79.7
9	86.74	0.386	208.6	132.4	76.1
10	81.85	0.364	218.6	147.1	71.5

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.981	58.8	20.5	38.3
2	195.09	0.869	104.2	41.0	63.2
3	175.60	0.782	140.7	61.5	79.2
4	160.04	0.713	171.0	82.0	89.0
5	147.29	0.656	196.7	102.5	94.2
6	136.63	0.608	219.0	123.0	96.0
7	127.58	0.568	238.6	143.5	95.0
8	119.78	0.533	256.0	164.0	92.0
9	112.98	0.503	271.6	184.5	87.1
10	107.01	0.476	285.9	205.0	80.8

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.155	69.3	24.2	45.2
2	229.83	1.023	122.8	48.3	74.5
3	206.88	0.921	165.8	72.5	93.3
4	188.54	0.839	201.5	96.6	104.8
5	173.52	0.773	231.8	120.8	111.0
6	160.97	0.717	258.0	144.9	113.1
7	150.30	0.669	281.1	169.1	112.0
8	141.11	0.628	301.6	193.2	108.3
9	133.11	0.593	320.0	217.4	102.6
10	126.06	0.561	336.8	241.5	95.2

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.715	102.9	32.6	70.3
2	303.44	1.486	178.3	65.2	113.1
3	268.77	1.316	236.9	97.9	139.1
4	241.96	1.185	284.4	130.5	153.9
5	220.54	1.080	324.0	163.1	160.9
6	202.99	0.994	357.9	195.7	162.2
7	188.32	0.922	387.4	228.3	159.0
8	175.86	0.861	413.4	261.0	152.4
9	165.12	0.809	436.7	293.6	143.1
10	155.76	0.763	457.7	326.2	131.5

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.345	140.7	43.2	97.5
2	371.96	2.070	248.4	86.4	162.0
3	333.83	1.858	334.4	129.7	204.8
4	303.46	1.689	405.3	172.9	232.4
5	278.63	1.551	465.2	216.1	249.1
6	257.93	1.435	516.8	259.3	257.4
7	240.38	1.338	561.9	302.6	259.3
8	225.29	1.254	601.8	345.8	256.0
9	212.16	1.181	637.6	389.0	248.6
10	200.63	1.117	669.9	432.2	237.7

:Maximum Storage Volume

Block 2

Drainage Area (ha)	**2.24**
Runoff Coeff. (C)	**0.69**
Time of Concentration	14.4 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.7
Max. Storage Required	**244.4**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	43.2	6
5	90.1	6
10	106.2	6
25	152.6	6
100	244.4	7

Required Storage Volumes (2yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	1.069	64.2	13.9	50.2
2	139.74	0.947	113.7	27.9	85.8
3	125.81	0.853	153.5	41.8	111.7
4	114.69	0.777	186.6	55.8	130.8
5	105.57	0.716	214.7	69.7	145.0
6	106.56	0.722	260.0	83.6	176.4
7	98.88	0.670	281.5	97.6	184.0
8	92.36	0.626	300.5	111.5	189.0
9	86.74	0.588	317.5	125.5	192.1
10	81.85	0.555	332.9	139.4	193.5
11	77.54	0.526	346.9	153.3	193.6
12	73.71	0.500	359.8	167.3	192.5
13	70.29	0.476	371.7	181.2	190.5
14	67.21	0.456	382.7	195.2	187.6

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	1.493	89.6	19.4	70.2
2	195.09	1.322	158.7	38.8	119.9
3	175.60	1.190	214.3	58.2	156.1
4	160.04	1.085	260.4	77.6	182.8
5	147.29	0.998	299.5	96.9	202.6
6	136.63	0.926	333.4	116.3	217.1
7	127.58	0.865	363.2	135.7	227.5
8	119.78	0.812	389.7	155.1	234.6
9	112.98	0.766	413.6	174.5	239.1
10	107.01	0.725	435.2	193.9	241.3
11	101.70	0.689	455.0	213.3	241.7
12	96.96	0.657	473.2	232.7	240.6
13	92.69	0.628	490.1	252.1	238.0
14	88.82	0.602	505.8	271.4	234.3

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.759	105.6	22.8	82.7
2	229.83	1.558	187.0	45.7	141.3
3	206.88	1.402	252.4	68.5	183.9
4	188.54	1.278	306.7	91.4	215.4
5	173.52	1.176	352.9	114.2	238.7
6	160.97	1.091	392.8	137.0	255.8
7	150.30	1.019	427.9	159.9	268.0
8	141.11	0.957	459.2	182.7	276.4
9	133.11	0.902	487.3	205.6	281.7
10	126.06	0.855	512.7	228.4	284.3
11	119.81	0.812	536.1	251.3	284.8
12	114.22	0.774	557.5	274.1	283.4
13	109.19	0.740	577.4	296.9	280.4
14	104.64	0.709	595.8	319.8	276.1

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	2.612	156.7	30.0	126.8
2	303.44	2.263	271.5	59.9	211.6
3	268.77	2.004	360.7	89.9	270.9
4	241.96	1.804	433.0	119.8	313.2
5	220.54	1.645	493.4	149.8	343.6
6	202.99	1.514	544.9	179.7	365.2
7	188.32	1.404	589.8	209.7	380.1
8	175.86	1.311	629.4	239.6	389.8
9	165.12	1.231	664.9	269.6	395.3
10	155.76	1.162	696.9	299.6	397.3
11	147.53	1.100	726.1	329.5	396.6
12	140.23	1.046	752.9	359.5	393.4
13	133.69	0.997	777.6	389.4	388.2
14	127.81	0.953	800.6	419.4	381.2

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	3.571	214.3	39.8	174.5
2	371.96	3.152	378.2	79.6	298.7
3	333.83	2.829	509.2	119.4	389.8
4	303.46	2.571	617.1	159.1	458.0
5	278.63	2.361	708.3	198.9	509.4
6	257.93	2.186	786.8	238.7	548.1
7	240.38	2.037	855.5	278.5	577.0
8	225.29	1.909	916.3	318.3	598.0
9	212.16	1.798	970.8	358.1	612.7
10	200.63	1.700	1020.0	397.9	622.2
11	190.41	1.613	1064.9	437.6	627.3
12	181.29	1.536	1106.0	477.4	628.6
13	173.09	1.467	1144.0	517.2	626.8
14	165.67	1.404	1179.2	557.0	622.2

:Maximum Storage Volume

Block 3

Drainage Area (ha)	**2.62**
Runoff Coeff. (C)	**0.69**
Time of Concentration	12.7 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.3 - 0.8
Max. Storage Required	**306.5**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	97.1	6
5	113.7	6
10	133.9	6
25	192.1	6
100	306.5	7

Required Storage Volumes (2yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	1.562	93.7	26.2	67.5
2	139.74	1.383	166.0	52.3	113.7
3	125.81	1.245	224.2	78.5	145.7
4	114.69	1.135	272.5	104.6	167.9
5	105.57	1.045	313.5	130.8	182.8
6	106.56	1.055	379.7	156.9	222.8
7	98.88	0.979	411.1	183.1	228.1
8	92.36	0.914	438.9	209.2	229.7
9	86.74	0.859	463.7	235.4	228.3
10	81.85	0.810	486.2	261.5	224.6
11	77.54	0.768	506.6	287.7	218.9
12	73.71	0.730	525.4	313.8	211.6

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	2.181	130.8	36.4	94.4
2	195.09	1.931	231.8	72.8	158.9
3	175.60	1.738	312.9	109.3	203.6
4	160.04	1.584	380.2	145.7	234.5
5	147.29	1.458	437.4	182.1	255.3
6	136.63	1.353	486.9	218.5	268.4
7	127.58	1.263	530.4	255.0	275.5
8	119.78	1.186	569.2	291.4	277.8
9	112.98	1.118	604.0	327.8	276.2
10	107.01	1.059	635.6	364.2	271.3
11	101.70	1.007	664.5	400.7	263.8
12	96.96	0.960	691.1	437.1	254.0

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	2.569	154.1	42.9	111.2
2	229.83	2.275	273.0	85.8	187.2
3	206.88	2.048	368.6	128.7	239.9
4	188.54	1.866	447.9	171.6	276.3
5	173.52	1.718	515.3	214.6	300.8
6	160.97	1.593	573.6	257.5	316.2
7	150.30	1.488	624.9	300.4	324.5
8	141.11	1.397	670.5	343.3	327.2
9	133.11	1.318	711.5	386.2	325.3
10	126.06	1.248	748.8	429.1	319.7
11	119.81	1.186	782.8	472.0	310.8
12	114.22	1.131	814.1	514.9	299.2

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	3.814	228.8	57.3	171.6
2	303.44	3.304	396.5	114.5	282.0
3	268.77	2.927	526.8	171.8	355.0
4	241.96	2.635	632.3	229.1	403.3
5	220.54	2.402	720.5	286.4	434.1
6	202.99	2.210	795.8	343.6	452.1
7	188.32	2.051	861.3	400.9	460.4
8	175.86	1.915	919.2	458.2	461.0
9	165.12	1.798	970.9	515.4	455.5
10	155.76	1.696	1017.7	572.7	445.0
11	147.53	1.607	1060.3	630.0	430.3
12	140.23	1.527	1099.4	687.3	412.2

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	5.215	312.9	76.1	236.8
2	371.96	4.603	552.3	152.1	400.2
3	333.83	4.131	743.6	228.2	515.4
4	303.46	3.755	901.2	304.3	596.9
5	278.63	3.448	1034.4	380.3	654.0
6	257.93	3.192	1149.0	456.4	692.6
7	240.38	2.975	1249.3	532.5	716.8
8	225.29	2.788	1338.1	608.5	729.6
9	212.16	2.625	1417.7	684.6	733.1
10	200.63	2.483	1489.6	760.7	728.9
11	190.41	2.356	1555.1	836.7	718.4
12	181.29	2.243	1615.2	912.8	702.4

:Maximum Storage Volume

Block 4

Drainage Area (ha)	**2.19**
Runoff Coeff. (C)	**0.69**
Time of Concentration	10.6 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.7
Max. Storage Required	**227**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	72.7	6
5	83.2	6
10	98.1	6
25	141.7	5
100	227	6

Required Storage Volumes (2yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.447	26.8	10.3	16.5
2	139.74	0.396	47.5	20.6	26.9
3	125.81	0.356	64.2	31.0	33.2
4	114.69	0.325	78.0	41.3	36.7
5	105.57	0.299	89.7	51.6	38.1
6	106.56	0.302	108.7	61.9	46.8
7	98.88	0.280	117.7	72.2	45.4
8	92.36	0.262	125.6	82.6	43.1
9	86.74	0.246	132.7	92.9	39.8
10	81.85	0.232	139.1	103.2	36.0

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.624	37.5	14.4	23.1
2	195.09	0.553	66.3	28.8	37.6
3	175.60	0.498	89.6	43.2	46.4
4	160.04	0.453	108.8	57.5	51.3
5	147.29	0.417	125.2	71.9	53.3
6	136.63	0.387	139.4	86.3	53.1
7	127.58	0.361	151.8	100.7	51.1
8	119.78	0.339	162.9	115.1	47.8
9	112.98	0.320	172.9	129.5	43.4
10	107.01	0.303	181.9	143.9	38.1

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	0.735	44.1	16.9	27.2
2	229.83	0.651	78.1	33.9	44.3
3	206.88	0.586	105.5	50.8	54.7
4	188.54	0.534	128.2	67.8	60.4
5	173.52	0.492	147.5	84.7	62.8
6	160.97	0.456	164.2	101.7	62.5
7	150.30	0.426	178.9	118.6	60.2
8	141.11	0.400	191.9	135.6	56.3
9	133.11	0.377	203.7	152.5	51.1
10	126.06	0.357	214.3	169.5	44.8

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.092	65.5	23.0	42.5
2	303.44	0.946	113.5	46.1	67.4
3	268.77	0.838	150.8	69.1	81.7
4	241.96	0.754	181.0	92.1	88.9
5	220.54	0.687	206.2	115.2	91.0
6	202.99	0.633	227.8	138.2	89.6
7	188.32	0.587	246.5	161.2	85.3
8	175.86	0.548	263.1	184.3	78.8
9	165.12	0.515	277.9	207.3	70.6
10	155.76	0.485	291.3	230.3	60.9

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	1.493	89.6	30.5	59.1
2	371.96	1.317	158.1	60.9	97.2
3	333.83	1.182	212.8	91.4	121.4
4	303.46	1.075	257.9	121.9	136.1
5	278.63	0.987	296.1	152.3	143.7
6	257.93	0.914	328.9	182.8	146.1
7	240.38	0.851	357.6	213.3	144.3
8	225.29	0.798	383.0	243.8	139.3
9	212.16	0.751	405.8	274.2	131.6
10	200.63	0.711	426.4	304.7	121.7

:Maximum Storage Volume

Block 5

Drainage Area (ha)	**2.00**
Runoff Coeff. (C)	**0.69**
Time of Concentration	10.5 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.7
Max. Storage Required	**214.8**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	68.8	6
5	79.3	6
10	93.4	6
25	134.1	5
100	214.8	7

Required Storage Volumes (2yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.888	53.3	16.0	37.3
2	139.74	0.787	94.4	32.0	62.4
3	125.81	0.708	127.5	48.1	79.4
4	114.69	0.646	155.0	64.1	90.9
5	105.57	0.594	178.3	80.1	98.2
6	106.56	0.600	216.0	96.1	119.9
7	98.88	0.557	233.8	112.1	121.7
8	92.36	0.520	249.6	128.1	121.5
9	86.74	0.488	263.7	144.2	119.6
10	81.85	0.461	276.5	160.2	116.3

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	1.240	74.4	22.3	52.1
2	195.09	1.098	131.8	44.6	87.2
3	175.60	0.989	178.0	66.9	111.0
4	160.04	0.901	216.2	89.3	127.0
5	147.29	0.829	248.8	111.6	137.2
6	136.63	0.769	276.9	133.9	143.0
7	127.58	0.718	301.7	156.2	145.5
8	119.78	0.674	323.7	178.5	145.2
9	112.98	0.636	343.5	200.8	142.7
10	107.01	0.602	361.5	223.1	138.3

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.461	87.7	26.3	61.4
2	229.83	1.294	155.3	52.6	102.7
3	206.88	1.165	209.7	78.9	130.8
4	188.54	1.061	254.8	105.2	149.6
5	173.52	0.977	293.1	131.4	161.6
6	160.97	0.906	326.2	157.7	168.5
7	150.30	0.846	355.4	184.0	171.4
8	141.11	0.794	381.3	210.3	171.0
9	133.11	0.749	404.7	236.6	168.1
10	126.06	0.710	425.8	262.9	163.0

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	2.169	130.1	35.2	94.9
2	303.44	1.879	225.5	70.4	155.1
3	268.77	1.664	299.6	105.6	194.0
4	241.96	1.498	359.6	140.8	218.8
5	220.54	1.366	409.7	176.0	233.7
6	202.99	1.257	452.6	211.2	241.3
7	188.32	1.166	489.8	246.4	243.4
8	175.86	1.089	522.8	281.7	241.1
9	165.12	1.023	552.2	316.9	235.3
10	155.76	0.965	578.8	352.1	226.7

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.966	178.0	46.7	131.2
2	371.96	2.618	314.1	93.5	220.7
3	333.83	2.349	422.9	140.2	282.7
4	303.46	2.136	512.5	186.9	325.6
5	278.63	1.961	588.3	233.7	354.6
6	257.93	1.815	653.5	280.4	373.1
7	240.38	1.692	710.5	327.1	383.4
8	225.29	1.585	761.0	373.9	387.1
9	212.16	1.493	806.3	420.6	385.7
10	200.63	1.412	847.2	467.3	379.8

:Maximum Storage Volume

Block 6

Drainage Area (ha)	**2.05**
Runoff Coeff. (C)	**0.69**
Time of Concentration	10.6 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.7
Max. Storage Required	**213.9**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	68.5	6
5	78.6	6
10	92.5	6
25	133.5	5
100	213.9	6

Required Storage Volumes (2yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.450	27.0	10.4	16.6
2	139.74	0.399	47.9	20.8	27.1
3	125.81	0.359	64.7	31.2	33.5
4	114.69	0.327	78.6	41.6	37.0
5	105.57	0.301	90.4	52.0	38.4
6	106.56	0.304	109.5	62.4	47.1
7	98.88	0.282	118.6	72.8	45.8
8	92.36	0.264	126.6	83.2	43.4
9	86.74	0.248	133.7	93.6	40.1
10	81.85	0.234	140.2	104.0	36.2

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.629	37.7	14.5	23.2
2	195.09	0.557	66.8	29.0	37.8
3	175.60	0.501	90.2	43.5	46.8
4	160.04	0.457	109.7	58.0	51.7
5	147.29	0.421	126.2	72.5	53.7
6	136.63	0.390	140.4	87.0	53.5
7	127.58	0.364	153.0	101.5	51.5
8	119.78	0.342	164.1	116.0	48.2
9	112.98	0.323	174.2	130.5	43.7
10	107.01	0.306	183.3	144.9	38.4

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	0.741	44.5	17.1	27.4
2	229.83	0.656	78.7	34.2	44.6
3	206.88	0.591	106.3	51.2	55.1
4	188.54	0.538	129.2	68.3	60.9
5	173.52	0.495	148.6	85.4	63.2
6	160.97	0.460	165.4	102.5	63.0
7	150.30	0.429	180.2	119.5	60.7
8	141.11	0.403	193.4	136.6	56.8
9	133.11	0.380	205.2	153.7	51.5
10	126.06	0.360	215.9	170.8	45.2

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.100	66.0	23.2	42.8
2	303.44	0.953	114.4	46.4	67.9
3	268.77	0.844	151.9	69.6	82.3
4	241.96	0.760	182.4	92.8	89.5
5	220.54	0.693	207.8	116.0	91.7
6	202.99	0.637	229.5	139.3	90.2
7	188.32	0.591	248.4	162.5	85.9
8	175.86	0.552	265.1	185.7	79.4
9	165.12	0.519	280.0	208.9	71.1
10	155.76	0.489	293.5	232.1	61.4

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	1.504	90.2	30.7	59.5
2	371.96	1.327	159.3	61.4	97.9
3	333.83	1.191	214.4	92.1	122.3
4	303.46	1.083	259.9	122.8	137.1
5	278.63	0.994	298.3	153.5	144.8
6	257.93	0.920	331.4	184.2	147.2
7	240.38	0.858	360.3	214.9	145.4
8	225.29	0.804	385.9	245.6	140.3
9	212.16	0.757	408.9	276.3	132.6
10	200.63	0.716	429.6	307.0	122.6

:Maximum Storage Volume

Block 7

Drainage Area (ha)	**2.80**
Runoff Coeff. (C)	**0.69**
Time of Concentration	13.5 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.3 - 0.8
Max. Storage Required	**357.4**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	112.8	7
5	134	7
10	157.8	7
25	224.2	7
100	357.4	6

Required Storage Volumes (2yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.424	25.4	9.8	15.7
2	139.74	0.376	45.1	19.6	25.5
3	125.81	0.338	60.9	29.4	31.5
4	114.69	0.308	74.0	39.2	34.8
5	105.57	0.284	85.2	49.0	36.2
6	106.56	0.286	103.1	58.8	44.4
7	98.88	0.266	111.7	68.5	43.1
8	92.36	0.248	119.2	78.3	40.9
9	86.74	0.233	125.9	88.1	37.8
10	81.85	0.220	132.0	97.9	34.1

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.592	35.5	13.7	21.9
2	195.09	0.525	62.9	27.3	35.6
3	175.60	0.472	85.0	41.0	44.0
4	160.04	0.430	103.3	54.6	48.7
5	147.29	0.396	118.8	68.3	50.6
6	136.63	0.367	132.3	81.9	50.3
7	127.58	0.343	144.1	95.6	48.5
8	119.78	0.322	154.6	109.2	45.4
9	112.98	0.304	164.0	122.9	41.2
10	107.01	0.288	172.6	136.5	36.1

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	0.698	41.9	16.1	25.8
2	229.83	0.618	74.2	32.2	42.0
3	206.88	0.556	100.1	48.2	51.9
4	188.54	0.507	121.7	64.3	57.3
5	173.52	0.467	140.0	80.4	59.6
6	160.97	0.433	155.8	96.5	59.3
7	150.30	0.404	169.7	112.6	57.2
8	141.11	0.379	182.1	128.7	53.5
9	133.11	0.358	193.3	144.7	48.5
10	126.06	0.339	203.4	160.8	42.6

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.036	62.2	21.9	40.3
2	303.44	0.897	107.7	43.7	64.0
3	268.77	0.795	143.1	65.6	77.5
4	241.96	0.716	171.7	87.4	84.3
5	220.54	0.652	195.7	109.3	86.4
6	202.99	0.600	216.1	131.1	85.0
7	188.32	0.557	233.9	153.0	80.9
8	175.86	0.520	249.7	174.9	74.8
9	165.12	0.488	263.7	196.7	67.0
10	155.76	0.461	276.4	218.6	57.8

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	1.416	85.0	28.9	56.1
2	371.96	1.250	150.0	57.8	92.2
3	333.83	1.122	202.0	86.7	115.2
4	303.46	1.020	244.8	115.7	129.1
5	278.63	0.936	280.9	144.6	136.4
6	257.93	0.867	312.1	173.5	138.6
7	240.38	0.808	339.3	202.4	136.9
8	225.29	0.757	363.4	231.3	132.1
9	212.16	0.713	385.1	260.2	124.8
10	200.63	0.674	404.6	289.1	115.5

:Maximum Storage Volume

Block 8

Drainage Area (ha)	**1.39**
Runoff Coeff. (C)	**0.69**
Time of Concentration	10 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.5
Max. Storage Required	**138.6**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	44.4	6
5	50.6	5
10	59.6	5
25	86.4	5
100	138.6	6

Required Storage Volumes (2yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.853	51.2	16.0	35.2
2	139.74	0.756	90.7	31.9	58.7
3	125.81	0.680	122.4	47.9	74.5
4	114.69	0.620	148.8	63.9	84.9
5	105.57	0.571	171.2	79.9	91.4
6	106.56	0.576	207.4	95.8	111.6
7	98.88	0.535	224.5	111.8	112.8
8	92.36	0.499	239.7	127.8	111.9
9	86.74	0.469	253.3	143.7	109.5
10	81.85	0.443	265.5	159.7	105.8

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	1.191	71.5	22.3	49.2
2	195.09	1.055	126.6	44.5	82.1
3	175.60	0.949	170.9	66.8	104.1
4	160.04	0.865	207.7	89.0	118.7
5	147.29	0.796	238.9	111.3	127.7
6	136.63	0.739	265.9	133.5	132.4
7	127.58	0.690	289.7	155.8	134.0
8	119.78	0.648	310.9	178.0	132.9
9	112.98	0.611	329.9	200.3	129.6
10	107.01	0.579	347.1	222.5	124.6

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*V_R = Q_R.T.60*	*V_C = Q_C.T.60*	*V= V_R-V_C*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.403	84.2	26.2	58.0
2	229.83	1.243	149.1	52.4	96.7
3	206.88	1.119	201.3	78.6	122.7
4	188.54	1.019	244.7	104.9	139.8
5	173.52	0.938	281.5	131.1	150.4
6	160.97	0.870	313.3	157.3	156.0
7	150.30	0.813	341.3	183.5	157.8
8	141.11	0.763	366.2	209.7	156.5
9	133.11	0.720	388.6	235.9	152.7
10	126.06	0.682	409.0	262.1	146.8

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*V_R = Q_R.T.60*	*V_C = Q_C.T.60*	*V= V_R-V_C*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	2.083	125.0	35.2	89.8
2	303.44	1.805	216.6	70.4	146.2
3	268.77	1.598	287.7	105.5	182.2
4	241.96	1.439	345.4	140.7	204.7
5	220.54	1.312	393.5	175.9	217.6
6	202.99	1.207	434.6	211.1	223.6
7	188.32	1.120	470.4	246.2	224.2
8	175.86	1.046	502.0	281.4	220.6
9	165.12	0.982	530.3	316.6	213.7
10	155.76	0.926	555.8	351.8	204.1

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*V_R = Q_R.T.60*	*V_C = Q_C.T.60*	*V= V_R-V_C*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.848	170.9	46.7	124.2
2	371.96	2.514	301.7	93.3	208.3
3	333.83	2.256	406.1	140.0	266.1
4	303.46	2.051	492.2	186.7	305.5
5	278.63	1.883	564.9	233.4	331.6
6	257.93	1.743	627.6	280.0	347.5
7	240.38	1.625	682.3	326.7	355.6
8	225.29	1.523	730.8	373.4	357.4
9	212.16	1.434	774.3	420.1	354.2
10	200.63	1.356	813.6	466.7	346.8

:Maximum Storage Volume

Block 9

Drainage Area (ha)	**1.48**
Runoff Coeff. (C)	**0.69**
Time of Concentration	15.4 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.5
Max. Storage Required	**147.2**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	47.1	6
5	53.7	5
10	63.2	5
25	91.7	5
100	147.2	6

Required Storage Volumes (2yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.624	37.4	13.9	23.6
2	139.74	0.552	66.3	27.7	38.6
3	125.81	0.497	89.5	41.6	48.0
4	114.69	0.453	108.8	55.4	53.4
5	105.57	0.417	125.2	69.3	55.9
6	106.56	0.421	151.6	83.1	68.5
7	98.88	0.391	164.2	97.0	67.2
8	92.36	0.365	175.3	110.8	64.4
9	86.74	0.343	185.2	124.7	60.5
10	81.85	0.324	194.1	138.6	55.6

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.871	52.2	19.3	32.9
2	195.09	0.771	92.5	38.6	53.9
3	175.60	0.694	124.9	57.9	67.0
4	160.04	0.633	151.8	77.3	74.6
5	147.29	0.582	174.7	96.6	78.1
6	136.63	0.540	194.4	115.9	78.6
7	127.58	0.504	211.8	135.2	76.6
8	119.78	0.473	227.3	154.5	72.8
9	112.98	0.447	241.2	173.8	67.4
10	107.01	0.423	253.8	193.1	60.7

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.C_a.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.026	61.6	22.8	38.8
2	229.83	0.909	109.0	45.5	63.5
3	206.88	0.818	147.2	68.3	78.9
4	188.54	0.745	178.9	91.0	87.9
5	173.52	0.686	205.8	113.8	92.0
6	160.97	0.636	229.1	136.5	92.5
7	150.30	0.594	249.5	159.3	90.3
8	141.11	0.558	267.7	182.0	85.7
9	133.11	0.526	284.1	204.8	79.4
10	126.06	0.498	299.0	227.5	71.5

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.C_a.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.523	91.4	30.8	60.5
2	303.44	1.319	158.3	61.7	96.6
3	268.77	1.169	210.4	92.5	117.8
4	241.96	1.052	252.5	123.4	129.1
5	220.54	0.959	287.7	154.2	133.5
6	202.99	0.883	317.8	185.1	132.7
7	188.32	0.819	343.9	215.9	128.0
8	175.86	0.765	367.0	246.7	120.3
9	165.12	0.718	387.7	277.6	110.1
10	155.76	0.677	406.4	308.4	97.9

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.C_a.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.082	124.9	40.8	84.1
2	371.96	1.838	220.6	81.7	138.9
3	333.83	1.650	296.9	122.5	174.4
4	303.46	1.499	359.9	163.3	196.6
5	278.63	1.377	413.0	204.1	208.9
6	257.93	1.274	458.8	245.0	213.9
7	240.38	1.188	498.9	285.8	213.1
8	225.29	1.113	534.3	326.6	207.7
9	212.16	1.048	566.1	367.4	198.7
10	200.63	0.991	594.8	408.3	186.5

:Maximum Storage Volume

Block 10

Drainage Area (ha)	**2.91**
Runoff Coeff. (C)	**0.69**
Time of Concentration	10 - 17.6 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.3 - 0.8
Max. Storage Required	**387.1**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	121.7	7
5	145.5	7
10	171.4	5
25	243.4	5
100	387.1	6

Required Storage Volumes (2yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.611	36.6	13.3	23.4
2	139.74	0.541	64.9	26.6	38.3
3	125.81	0.487	87.7	39.9	47.8
4	114.69	0.444	106.6	53.1	53.4
5	105.57	0.409	122.6	66.4	56.2
6	106.56	0.412	148.5	79.7	68.8
7	98.88	0.383	160.8	93.0	67.8
8	92.36	0.358	171.6	106.3	65.3
9	86.74	0.336	181.3	119.6	61.8
10	81.85	0.317	190.1	132.8	57.3

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.853	51.2	18.5	32.6
2	195.09	0.755	90.6	37.0	53.6
3	175.60	0.680	122.4	55.5	66.8
4	160.04	0.620	148.7	74.1	74.6
5	147.29	0.570	171.0	92.6	78.5
6	136.63	0.529	190.4	111.1	79.3
7	127.58	0.494	207.4	129.6	77.8
8	119.78	0.464	222.6	148.1	74.4
9	112.98	0.437	236.2	166.6	69.5
10	107.01	0.414	248.5	185.2	63.4

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.005	60.3	21.8	38.5
2	229.83	0.890	106.8	43.6	63.1
3	206.88	0.801	144.1	65.4	78.7
4	188.54	0.730	175.2	87.3	87.9
5	173.52	0.672	201.5	109.1	92.4
6	160.97	0.623	224.3	130.9	93.4
7	150.30	0.582	244.4	152.7	91.7
8	141.11	0.546	262.2	174.5	87.7
9	133.11	0.515	278.2	196.3	81.9
10	126.06	0.488	292.8	218.1	74.7

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.491	89.5	29.5	60.0
2	303.44	1.292	155.0	59.1	96.0
3	268.77	1.144	206.0	88.6	117.4
4	241.96	1.030	247.3	118.1	129.2
5	220.54	0.939	281.7	147.6	134.1
6	202.99	0.864	311.2	177.2	134.0
7	188.32	0.802	336.8	206.7	130.1
8	175.86	0.749	359.4	236.2	123.2
9	165.12	0.703	379.7	265.8	113.9
10	155.76	0.663	398.0	295.3	102.7

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.039	122.4	39.1	83.3
2	371.96	1.800	216.0	78.2	137.8
3	333.83	1.615	290.8	117.3	173.5
4	303.46	1.468	352.4	156.4	196.0
5	278.63	1.348	404.5	195.5	209.0
6	257.93	1.248	449.3	234.6	214.7
7	240.38	1.163	488.5	273.7	214.8
8	225.29	1.090	523.3	312.8	210.4
9	212.16	1.027	554.4	351.9	202.4
10	200.63	0.971	582.5	391.0	191.5

:Maximum Storage Volume

Block 11

Drainage Area (ha)	**1.47**
Runoff Coeff. (C)	**0.69**
Time of Concentration	10 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.5
Max. Storage Required	**146.1**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	46.8	6
5	53.3	5
10	62.8	5
25	91	5
100	146.1	6

Required Storage Volumes (2yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.668	40.1	15.0	25.1
2	139.74	0.592	71.0	29.9	41.1
3	125.81	0.533	95.9	44.9	51.0
4	114.69	0.486	116.6	59.8	56.7
5	105.57	0.447	134.1	74.8	59.4
6	106.56	0.451	162.4	89.7	72.7
7	98.88	0.419	175.9	104.7	71.2
8	92.36	0.391	187.7	119.6	68.1
9	86.74	0.367	198.4	134.6	63.8
10	81.85	0.347	208.0	149.5	58.4

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.933	56.0	20.8	35.1
2	195.09	0.826	99.1	41.7	57.4
3	175.60	0.744	133.9	62.5	71.3
4	160.04	0.678	162.7	83.4	79.3
5	147.29	0.624	187.1	104.2	82.9
6	136.63	0.579	208.3	125.1	83.2
7	127.58	0.540	226.9	145.9	81.0
8	119.78	0.507	243.5	166.8	76.7
9	112.98	0.478	258.4	187.6	70.8
10	107.01	0.453	271.9	208.4	63.4

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.099	65.9	24.6	41.4
2	229.83	0.973	116.8	49.1	67.7
3	206.88	0.876	157.7	73.7	84.0
4	188.54	0.798	191.6	98.2	93.4
5	173.52	0.735	220.4	122.8	97.7
6	160.97	0.682	245.4	147.3	98.1
7	150.30	0.636	267.3	171.9	95.4
8	141.11	0.598	286.8	196.4	90.4
9	133.11	0.564	304.4	221.0	83.4
10	126.06	0.534	320.3	245.6	74.7

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.632	97.9	33.3	64.6
2	303.44	1.413	169.6	66.6	103.0
3	268.77	1.252	225.4	99.9	125.4
4	241.96	1.127	270.5	133.2	137.3
5	220.54	1.027	308.2	166.5	141.7
6	202.99	0.946	340.4	199.8	140.6
7	188.32	0.877	368.4	233.1	135.3
8	175.86	0.819	393.2	266.4	126.8
9	165.12	0.769	415.3	299.7	115.6
10	155.76	0.726	435.3	333.0	102.3

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.Ca.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.231	133.8	44.1	89.8
2	371.96	1.969	236.3	88.2	148.1
3	333.83	1.767	318.1	132.2	185.8
4	303.46	1.606	385.5	176.3	209.2
5	278.63	1.475	442.5	220.4	222.1
6	257.93	1.365	491.5	264.5	227.0
7	240.38	1.272	534.4	308.6	225.8
8	225.29	1.193	572.4	352.6	219.8
9	212.16	1.123	606.4	396.7	209.7
10	200.63	1.062	637.2	440.8	196.4

:Maximum Storage Volume

Block 12

Drainage Area (ha)	**5.12**
Runoff Coeff. (C)	**0.69**
Time of Concentration	16.1 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.4 - 1.3
Max. Storage Required	**733.1**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	229.7	8
5	277.8	8
10	327.2	8
25	461	8
100	733.1	9

Required Storage Volumes (2yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.798	47.9	16.2	31.7
2	139.74	0.707	84.8	32.3	52.5
3	125.81	0.636	114.5	48.5	66.1
4	114.69	0.580	139.2	64.6	74.6
5	105.57	0.534	160.2	80.8	79.4
6	106.56	0.539	194.0	96.9	97.1
7	98.88	0.500	210.0	113.1	96.9
8	92.36	0.467	224.2	129.2	95.0
9	86.74	0.439	236.9	145.4	91.5
10	81.85	0.414	248.4	161.5	86.8

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Ca.I.A)*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	1.114	66.8	22.5	44.3
2	195.09	0.987	118.4	45.0	73.4
3	175.60	0.888	159.9	67.5	92.3
4	160.04	0.809	194.2	90.0	104.2
5	147.29	0.745	223.5	112.6	110.9
6	136.63	0.691	248.8	135.1	113.7
7	127.58	0.645	271.0	157.6	113.4
8	119.78	0.606	290.8	180.1	110.7
9	112.98	0.571	308.6	202.6	105.9
10	107.01	0.541	324.7	225.1	99.6

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.C\alpha.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.312	78.7	26.5	52.2
2	229.83	1.162	139.5	53.0	86.4
3	206.88	1.046	188.3	79.6	108.8
4	188.54	0.954	228.8	106.1	122.8
5	173.52	0.878	263.3	132.6	130.7
6	160.97	0.814	293.1	159.1	133.9
7	150.30	0.760	319.2	185.7	133.6
8	141.11	0.714	342.5	212.2	130.4
9	133.11	0.673	363.5	238.7	124.8
10	126.06	0.638	382.5	265.2	117.3

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.C\alpha.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.948	116.9	35.7	81.2
2	303.44	1.688	202.6	71.5	131.1
3	268.77	1.495	269.1	107.2	161.9
4	241.96	1.346	323.0	143.0	180.1
5	220.54	1.227	368.1	178.7	189.3
6	202.99	1.129	406.5	214.5	192.1
7	188.32	1.048	440.0	250.2	189.8
8	175.86	0.978	469.6	286.0	183.6
9	165.12	0.919	496.0	321.7	174.3
10	155.76	0.867	519.9	357.4	162.5

:Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A*(t+B)^{-C}$	$Q_R = 0.0028(C.C\alpha.I.A)$	$V_R = Q_R.T.60$	$V_C = Q_C.T.60$	$V = V_R - V_C$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.664	159.8	47.4	112.5
2	371.96	2.351	282.2	94.8	187.4
3	333.83	2.110	379.9	142.2	237.7
4	303.46	1.918	460.4	189.5	270.9
5	278.63	1.761	528.4	236.9	291.5
6	257.93	1.631	587.0	284.3	302.7
7	240.38	1.520	638.2	331.7	306.5
8	225.29	1.424	683.6	379.1	304.5
9	212.16	1.341	724.2	426.5	297.8
10	200.63	1.268	761.0	473.8	287.1

:Maximum Storage Volume

Block 13

Drainage Area (ha)	**4.75**
Runoff Coeff. (C)	**0.51**
Time of Concentration	27.9 - 36.8 min
Time Step	1 min
Controlled Release Rate (Q_C)	0.2 - 0.7
Max. Storage Required	**628.6**

Results

Storm Event	Storage	Time
(yr)	(m3)	(min)
2	193.6	11
5	241.7	11
10	284.8	11
25	397.3	10
100	628.6	12

Required Storage Volumes (2yr)

T	*$I = A*(t+B)^{-C}$*	*$Q_R = 0.0028(C.Ca.I.A)$*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	157.74	0.684	41.0	13.8	27.2
2	139.74	0.606	72.7	27.6	45.1
3	125.81	0.545	98.1	41.4	56.7
4	114.69	0.497	119.3	55.2	64.0
5	105.57	0.458	137.3	69.0	68.2
6	106.56	0.462	166.2	82.9	83.4
7	98.88	0.429	180.0	96.7	83.3
8	92.36	0.400	192.1	110.5	81.7
9	86.74	0.376	203.0	124.3	78.7
10	81.85	0.355	212.8	138.1	74.7

:Maximum Storage Volume

Required Storage Volumes (5yr)

T	*$I = A*(t+B)^{-C}$*	*$Q_R = 0.0028(C.Ca.I.A)$*	*$V_R = Q_R.T.60$*	*$V_C = Q_C.T.60$*	*$V = V_R - V_C$*
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	220.29	0.955	57.3	19.2	38.0
2	195.09	0.845	101.5	38.5	63.0
3	175.60	0.761	137.0	57.7	79.3
4	160.04	0.694	166.5	77.0	89.5
5	147.29	0.638	191.5	96.2	95.3
6	136.63	0.592	213.2	115.5	97.7
7	127.58	0.553	232.2	134.7	97.5
8	119.78	0.519	249.2	154.0	95.2
9	112.98	0.490	264.4	173.2	91.2
10	107.01	0.464	278.2	192.4	85.8

:Maximum Storage Volume

Required Storage Volumes (10yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Cα.I.A)*	*V_R = Q_R.T.60*	*V_C = Q_C.T.60*	*V= V_R-V_C*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	259.52	1.125	67.5	22.7	44.8
2	229.83	0.996	119.5	45.3	74.2
3	206.88	0.897	161.4	68.0	93.4
4	188.54	0.817	196.1	90.7	105.4
5	173.52	0.752	225.6	113.4	112.2
6	160.97	0.698	251.1	136.0	115.1
7	150.30	0.651	273.6	158.7	114.9
8	141.11	0.612	293.5	181.4	112.2
9	133.11	0.577	311.5	204.0	107.5
10	126.06	0.546	327.8	226.7	101.1

:Maximum Storage Volume

Required Storage Volumes (25yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Cα.I.A)*	*V_R = Q_R.T.60*	*V_C = Q_C.T.60*	*V= V_R-V_C*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	350.25	1.670	100.2	30.6	69.6
2	303.44	1.447	173.6	61.1	112.5
3	268.77	1.281	230.6	91.7	139.0
4	241.96	1.153	276.8	122.2	154.6
5	220.54	1.051	315.4	152.8	162.6
6	202.99	0.968	348.4	183.3	165.1
7	188.32	0.898	377.1	213.9	163.2
8	175.86	0.838	402.4	244.4	158.0
9	165.12	0.787	425.1	275.0	150.1
10	155.76	0.743	445.5	305.5	140.0

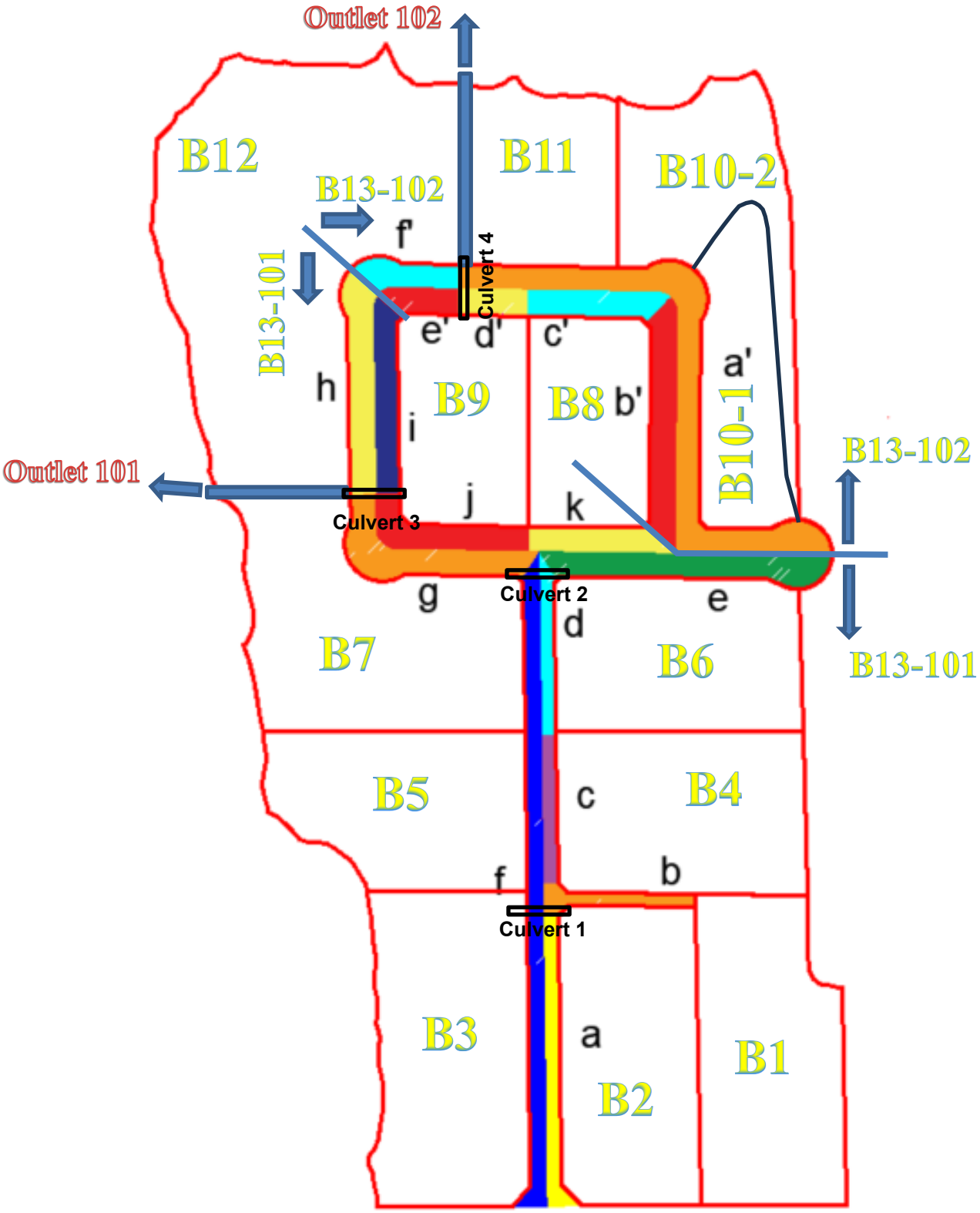
:Maximum Storage Volume

Required Storage Volumes (100yr)

T	*I = A*(t+B)^-C*	*Q_R = 0.0028(C.Cα.I.A)*	*V_R = Q_R.T.60*	*V_C = Q_C.T.60*	*V= V_R-V_C*
Time	**Rainfall Intensity**	**Runoff**	**Runoff Vol.**	**Controlled Release Vol.**	**Storage Vol.**
(min)	(mm/hr)	(cms)	(m3)	(m3)	(m3)
1	421.43	2.283	137.0	40.5	96.5
2	371.96	2.015	241.8	81.0	160.8
3	333.83	1.808	325.5	121.5	204.0
4	303.46	1.644	394.5	162.0	232.5
5	278.63	1.509	452.8	202.5	250.3
6	257.93	1.397	503.0	243.0	260.0
7	240.38	1.302	546.9	283.5	263.4
8	225.29	1.220	585.8	324.0	261.8
9	212.16	1.149	620.6	364.5	256.1
10	200.63	1.087	652.1	405.0	247.1

:Maximum Storage Volume

Swale Size Calculation



Catchment	Ditch	Slope (%)	Q Components*	Q100 (Major) (cms)
101	a	0.6	B2	0.917
	b	0.6	B1	0.974
	c	0.6	B4	1.040
	d	0.6	c+B6 (50%)	1.539
	e	0.5	B6 (50%)	0.539
	f	0.6	a+b+B3 (25%)+B5 (25%)+ B7 (10%)	2.599
	g	0.85	d+f+e+ B7 (15%)	4.902
	h	1.8	B12 (15%)	0.327
	i	1.4	B9 (35%)	0.234
	j	0.5	k + B9 (35%)	0.462
	k	0.5	B8 (30%)	0.229
	Culvert 1	1	a+b	1.890
	Culvert 2	1	e+d	2.079
	Culvert 3	2.0	i+j	0.696
	Outlet 101	1.5	g+h+i+j	5.925
102	a'	0.5	B10-1 + B10-2 (25%) + B11 (25%)	1.064
	b'	0.5	B8 (35%)	0.284
	c'	0.5	b'+ B8 (35%)	0.546
	d'	0.5	c'+ B9 (20%)	0.755
	e'	1	B9 (10%)	0.071
	f'	1	B12 (10%)	0.211
	Culvert 4	1	e'+d'	0.826
	Outlet 102	3.5	a'+d'+e'+f'	2.101

*** B#: Block number**
Note: No static ponding above the crown of the roads.

Ditch	Section	Bottom Width (m)	Depth (m)	Side Slope (H/V)
Outlet 101	Trapezoidal Channel	1.6	0.9	3
Others	Trapezoidal Channel	1.0	0.9	3

Culvert	Type - Material	Width (m)	Height (m)	Length (m)
1	Box - Concrete	1.4	0.45	16.5
2	Box - Concrete	1.45	0.45	31.0
3	Box - Concrete	0.65	0.45	27.0
4	Box - Concrete	0.7	0.45	27.0

S = 0.6% (Ditch # a,b,c,d,f)

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

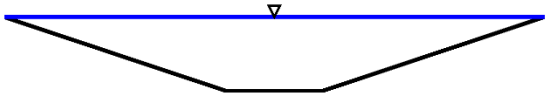
Ditch Flow Estimation

Inputs

Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	0.6	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results

Flow area, a	2.4375	m^2
Wetted perimeter, P_w	5.7434	m
Hydraulic radius, R_h	0.4244	m
Velocity, v	1.4582	m/s
Flow, Q	3.5543	m^3/s
Velocity head, h_v	0.1084	m
Top width, T	5.5000	m
Froude number, F	0.70	
Average shear stress (tractive force), tau	24.9699	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0605	
n for design rock size per Bathurst	0.0430	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.0807	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.0851	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.0851	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.0755	m
Required angular rock size, D50 (Searcy 1967)	0.0468	m



S = 0.5% (Ditch # e,j,k,a',b',c',d')

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

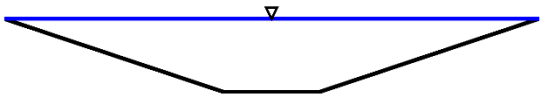
Ditch Flow Estimation

Inputs

Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	0.5	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results

Flow area, a	2.4375	m^2
Wetted perimeter, P_w	5.7434	m
Hydraulic radius, R_h	0.4244	m
Velocity, v	1.3311	m/s
Flow, Q	3.2446	m^3/s
Velocity head, h_v	0.0903	m
Top width, T	5.5000	m
Froude number, F	0.64	
Average shear stress (tractive force), tau	20.8083	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0605	
n for design rock size per Bathurst	0.0437	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.0673	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.0709	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.0709	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.0601	m
Required angular rock size, D50 (Searcy 1967)	0.0390	m



S = 0.85% (Ditch # g)

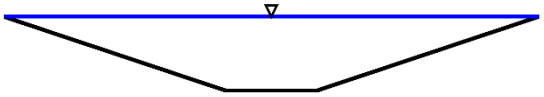
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Ditch Flow Estimation

Inputs		
Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	0.85	% rise/run
Flow depth, y	0.8	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	2.7200	m^2
Wetted perimeter, P_w	6.0596	m
Hydraulic radius, R_h	0.4489	m
Velocity, v	1.8016	m/s
Flow, Q	4.9005	m^3/s
Velocity head, h_v	0.1655	m
Top width, T	5.8000	m
Froude number, F	0.84	
Average shear stress (tractive force), tau	37.4139	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0594	
n for design rock size per Bathurst	0.0409	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.1232	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.1299	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.1299	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.1260	m
Required angular rock size, D50 (Searcy 1967)	0.0714	m



S = 1.8% (Ditch # h)

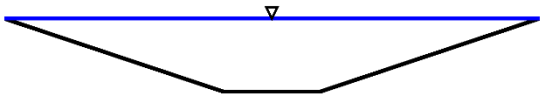
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Ditch Flow Estimation

Inputs		
Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	1.8	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	2.4375	m^2
Wetted perimeter, P_w	5.7434	m
Hydraulic radius, R_h	0.4244	m
Velocity, v	2.5256	m/s
Flow, Q	6.1561	m^3/s
Velocity head, h_v	0.3252	m
Top width, T	5.5000	m
Froude number, F	1.21	
Average shear stress (tractive force), tau	74.9098	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0605	
n for design rock size per Bathurst	0.0396	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.2421	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.2552	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.2552	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.2979	m
Required angular rock size, D50 (Searcy 1967)	0.1403	m



S = 1.4% (Ditch # i)

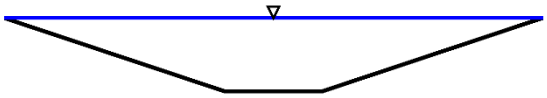
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Ditch Flow Estimation

Inputs		
Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	1.4	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	2.4375	m^2
Wetted perimeter, P_w	5.7434	m
Hydraulic radius, R_h	0.4244	m
Velocity, v	2.2274	m/s
Flow, Q	5.4292	m^3/s
Velocity head, h_v	0.2530	m
Top width, T	5.5000	m
Froude number, F	1.07	
Average shear stress (tractive force), tau	58.2632	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0605	
n for design rock size per Bathurst	0.0403	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.1883	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.1985	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.1985	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.2176	m
Required angular rock size, D50 (Searcy 1967)	0.1091	m



S = 1.5% (Ditch # Outlet 101)

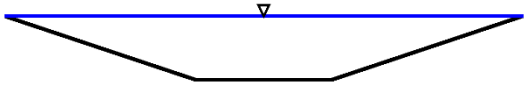
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Ditch Flow Estimation

Inputs		
Bottom width, b	1.6	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	1.5	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	2.8875	m^2
Wetted perimeter, P_w	6.3434	m
Hydraulic radius, R_h	0.4552	m
Velocity, v	2.4158	m/s
Flow, Q	6.9756	m^3/s
Velocity head, h_v	0.2976	m
Top width, T	6.1000	m
Froude number, F	1.12	
Average shear stress (tractive force), tau	66.9548	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0593	
n for design rock size per Bathurst	0.0396	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.2215	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.2335	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.2335	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.2666	m
Required angular rock size, D50 (Searcy 1967)	0.1284	m



S = 1.0% (Ditch # e',f')

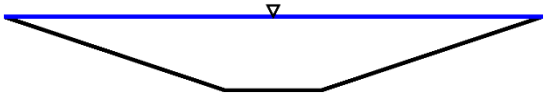
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Ditch Flow Estimation

Inputs		
Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	1.0	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	2.4375	m^2
Wetted perimeter, P_w	5.7434	m
Hydraulic radius, R_h	0.4244	m
Velocity, v	1.8825	m/s
Flow, Q	4.5885	m^3/s
Velocity head, h_v	0.1807	m
Top width, T	5.5000	m
Froude number, F	0.90	
Average shear stress (tractive force), tau	41.6166	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0605	
n for design rock size per Bathurst	0.0414	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.1345	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.1418	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.1418	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.1429	m
Required angular rock size, D50 (Searcy 1967)	0.0780	m



S = 3.5% (Ditch # Outlet 102)

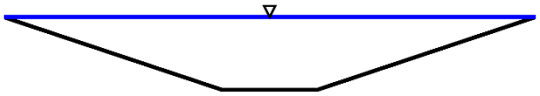
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Ditch Flow Estimation

Inputs		
Bottom width, b	1.0	m
Side slope 1 (horiz./vert.)	3	
Side slope 2 (horiz./vert.)	3	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.03	
Channel slope, S	3.5	% rise/run
Flow depth, y	0.75	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	2.4375	m^2
Wetted perimeter, P_w	5.7434	m
Hydraulic radius, R_h	0.4244	m
Velocity, v	3.5218	m/s
Flow, Q	8.5843	m^3/s
Velocity head, h_v	0.6324	m
Top width, T	5.5000	m
Froude number, F	1.69	
Average shear stress (tractive force), tau	145.6580	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0605	
n for design rock size per Bathurst	0.0376	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.4708	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.4963	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.4963	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.6841	m
Required angular rock size, D50 (Searcy 1967)	0.2729	m



Culvert Size Calculation (Culvert 1)

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

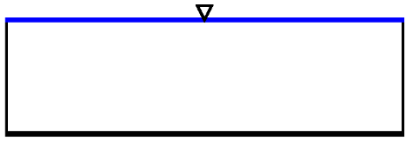
Culvert Flow Estimation

Inputs

Bottom width, b	1.4 m
Side slope 1 (horiz./vert.)	0
Side slope 2 (horiz./vert.)	0
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.011
Channel slope, S	1 % rise/run
Flow depth, y	0.4 m
Bend Angle ? (for riprap sizing)	1
Rock specific gravity (2.65)	2.65
Design rock size, D50 ◉Isbash ○Maynord ○Searcy * 1.25 (See notes)	10331559375494216.0000 m

Results

Flow area, a	0.5600	m^2
Wetted perimeter, P_w	2.2000	m
Hydraulic radius, R_h	0.2545	m
Velocity, v	3.6513	m/s
Flow, Q	2.0447	m^3/s
Velocity head, h_v	0.6798	m
Top width, T	1.4000	m
Froude number, F	1.84	
Average shear stress (tractive force), tau	24.9607	N/m^2
n for design rock size per Strickler	21.4605	
n for design rock size per Blodgett	-0.0033	
n for design rock size per Bathurst	0.0000	
Blodgett vs. Bathurst	----	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.5061	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	8265247500395373.0000	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	8265247500395373.0000	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.8762	m
Required angular rock size, D50 (Searcy 1967)	0.2933	m



Culvert Size Calculation (Culvert 2)

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

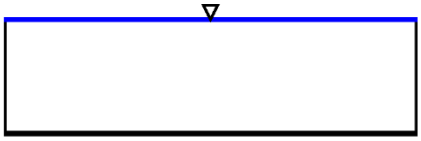
Culvert Flow Estimation

Inputs

Bottom width, b	1.45 m
Side slope 1 (horiz./vert.)	0
Side slope 2 (horiz./vert.)	0
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.011
Channel slope, S	1 % rise/run
Flow depth, y	0.4 m
Bend Angle ? (for riprap sizing)	1
Rock specific gravity (2.65)	2.65
Design rock size, D50 ◉Isbash ○Maynord ○Searcy * 1.25 (See notes)	10506854422802526.0000 m

Results

Flow area, a	0.5800	m^2
Wetted perimeter, P_w	2.2500	m
Hydraulic radius, R_h	0.2578	m
Velocity, v	3.6822	m/s
Flow, Q	2.1357	m^3/s
Velocity head, h_v	0.6913	m
Top width, T	1.4500	m
Froude number, F	1.86	
Average shear stress (tractive force), tau	25.2777	N/m^2
n for design rock size per Strickler	22.1180	
n for design rock size per Blodgett	-0.0033	
n for design rock size per Bathurst	0.0000	
Blodgett vs. Bathurst	----	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.5147	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	8405483538242021.0000	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	8405483538242021.0000	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.8948	m
Required angular rock size, D50 (Searcy 1967)	0.2983	m



Culvert Size Calculation (Culvert 3)

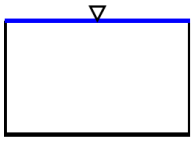
Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

Culvert Flow Estimation

Inputs		
Bottom width, b	0.65	m
Side slope 1 (horiz./vert.)	0	
Side slope 2 (horiz./vert.)	0	
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.011	
Channel slope, S	2	% rise/run
Flow depth, y	0.4	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	
Design rock size, D50 ○Isbash ○Maynord ○ Searcy * 1.25 (See notes)	0.1573	m

Results		
Flow area, a	0.2600	m^2
Wetted perimeter, P_w	1.4500	m
Hydraulic radius, R_h	0.1793	m
Velocity, v	4.0881	m/s
Flow, Q	1.0629	m^3/s
Velocity head, h_v	0.8522	m
Top width, T	0.6500	m
Froude number, F	2.06	
Average shear stress (tractive force), tau	35.1663	N/m^2
n for design rock size per Strickler	0.0348	
n for design rock size per Blodgett	0.0627	
n for design rock size per Bathurst	0.0117	
Blodgett vs. Bathurst	Blodgett	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.6344	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	10361099825336410.0000	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	10361099825336410.0000	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	1.1622	m
Required angular rock size, D50 (Searcy 1967)	0.3677	m



Culvert Size Calculation (Culvert 4)

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0 Scugog Line 6

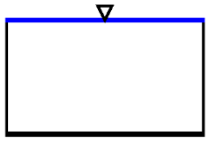
Culvert Flow Estimation

Inputs

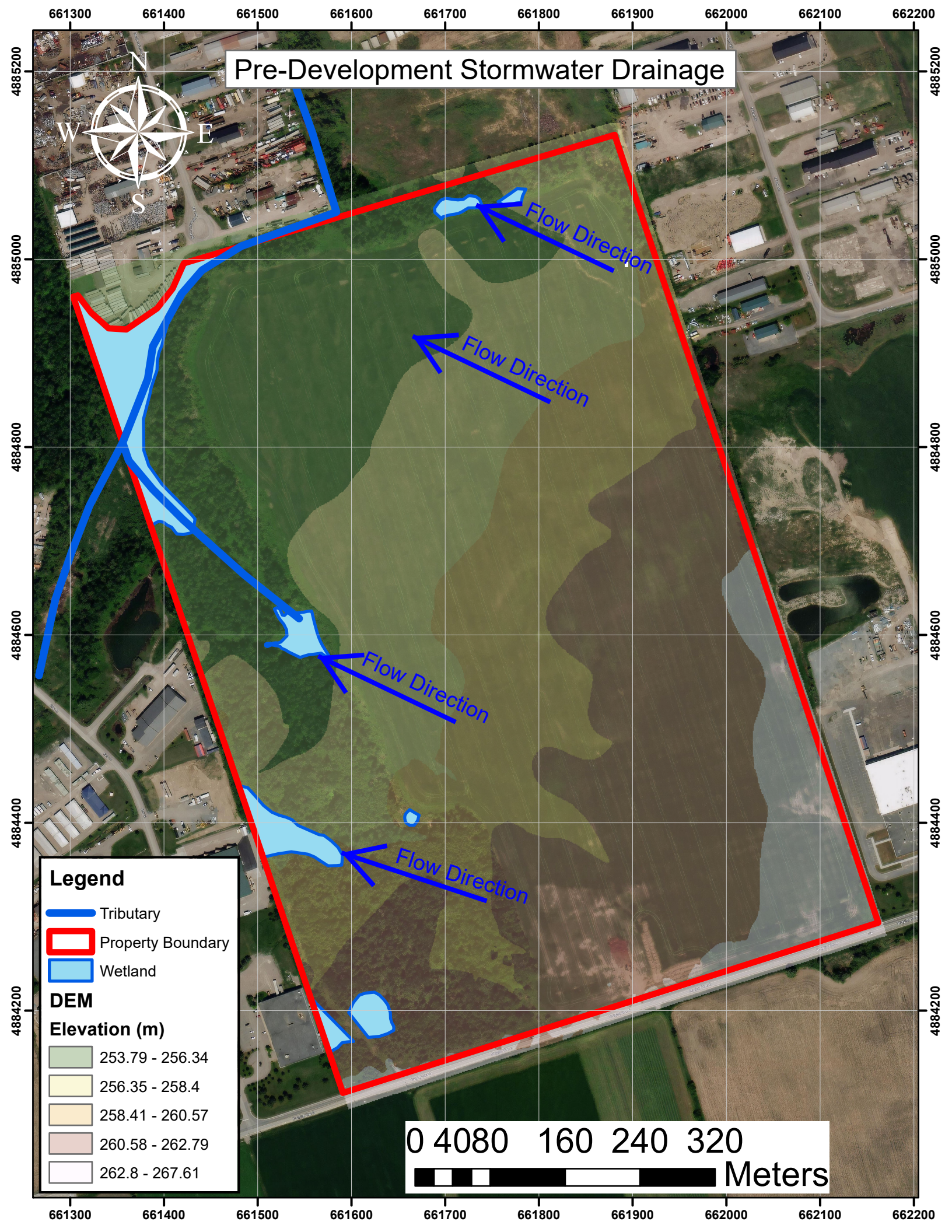
Bottom width, b	0.7 m
Side slope 1 (horiz./vert.)	0
Side slope 2 (horiz./vert.)	0
Manning roughness, n ? ○Strickler ○B/B (See notes)	0.011
Channel slope, S	1 % rise/run
Flow depth, y	0.4 m
Bend Angle ? (for riprap sizing)	1
Rock specific gravity (2.65)	2.65
Design rock size, D50 ◉Isbash ○Maynord ○Searcy * 1.25 (See notes)	6832313369497750.0000 m

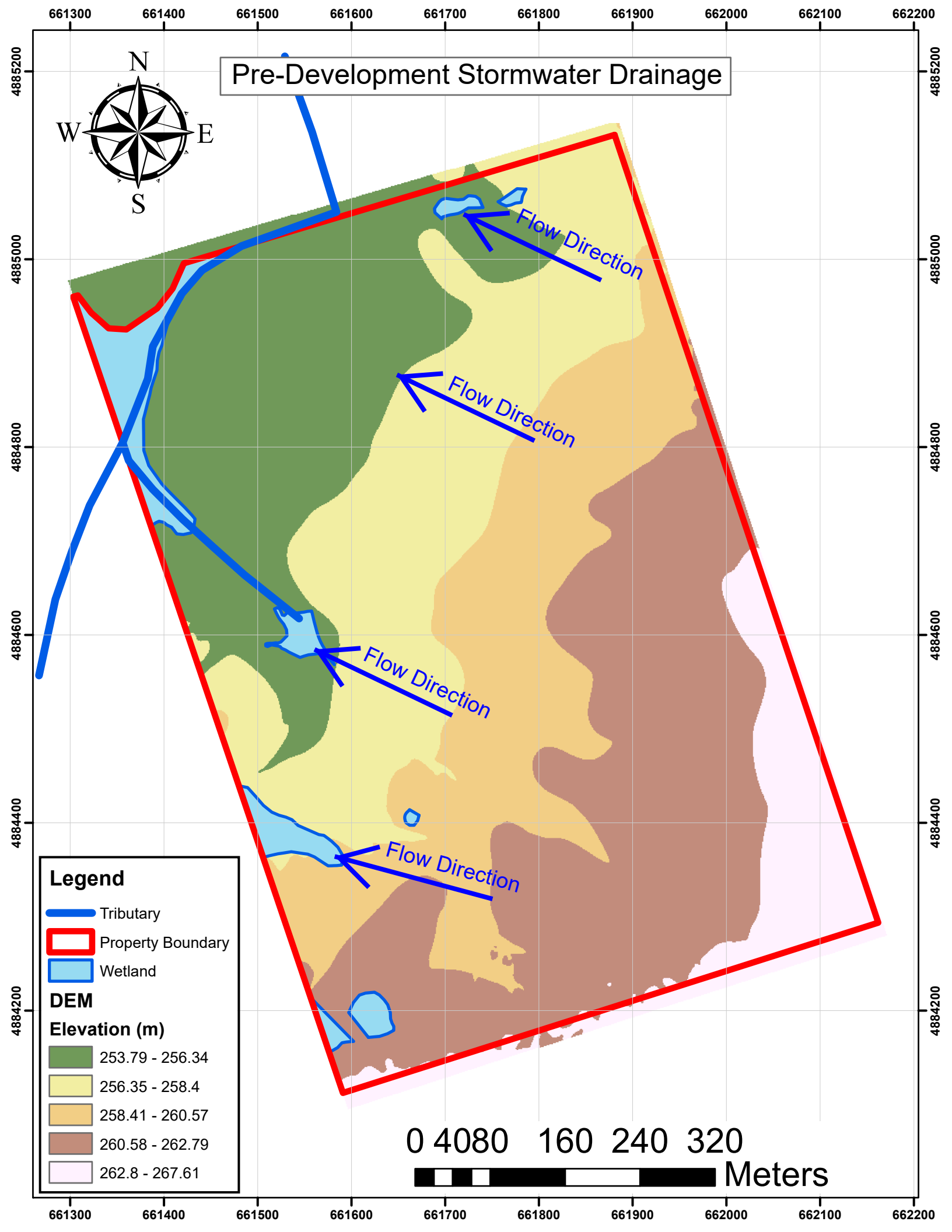
Results

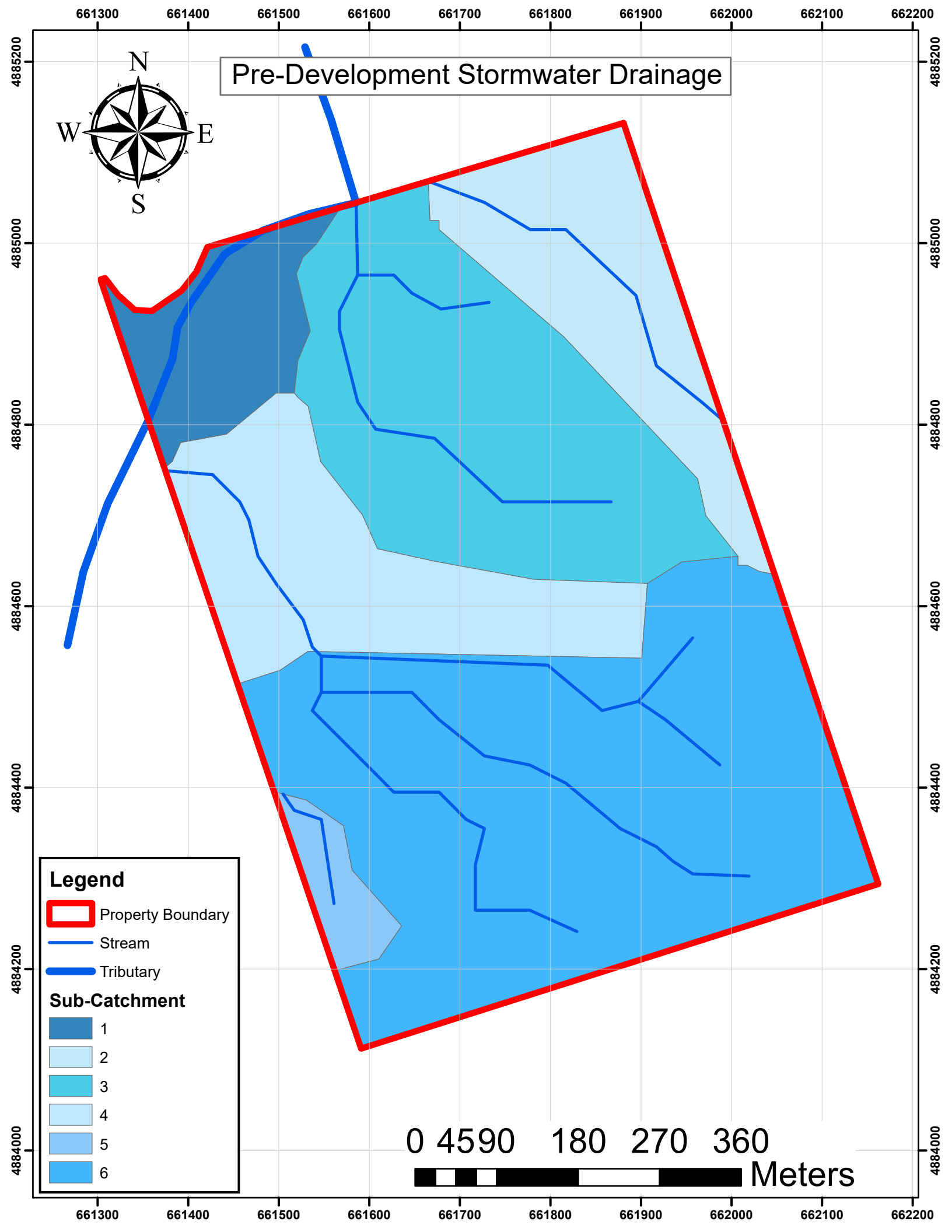
Flow area, a	0.2800	m^2
Wetted perimeter, P_w	1.5000	m
Hydraulic radius, R_h	0.1867	m
Velocity, v	2.9693	m/s
Flow, Q	0.8314	m^3/s
Velocity head, h_v	0.4496	m
Top width, T	0.7000	m
Froude number, F	1.50	
Average shear stress (tractive force), tau	18.3045	N/m^2
n for design rock size per Strickler	20.4612	
n for design rock size per Blodgett	-0.0033	
n for design rock size per Bathurst	0.0000	
Blodgett vs. Bathurst	----	
Required bottom angular rock size, D50 (Isbash & MC) ?	0.3347	m
Required side slope 1 angular rock size, D50 (Isbash & MC) ?	5465850695598200.0000	m
Required side slope 2 angular rock size, D50 (Isbash & MC) ?	5465850695598200.0000	m
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	0.5225	m
Required angular rock size, D50 (Searcy 1967)	0.1940	m

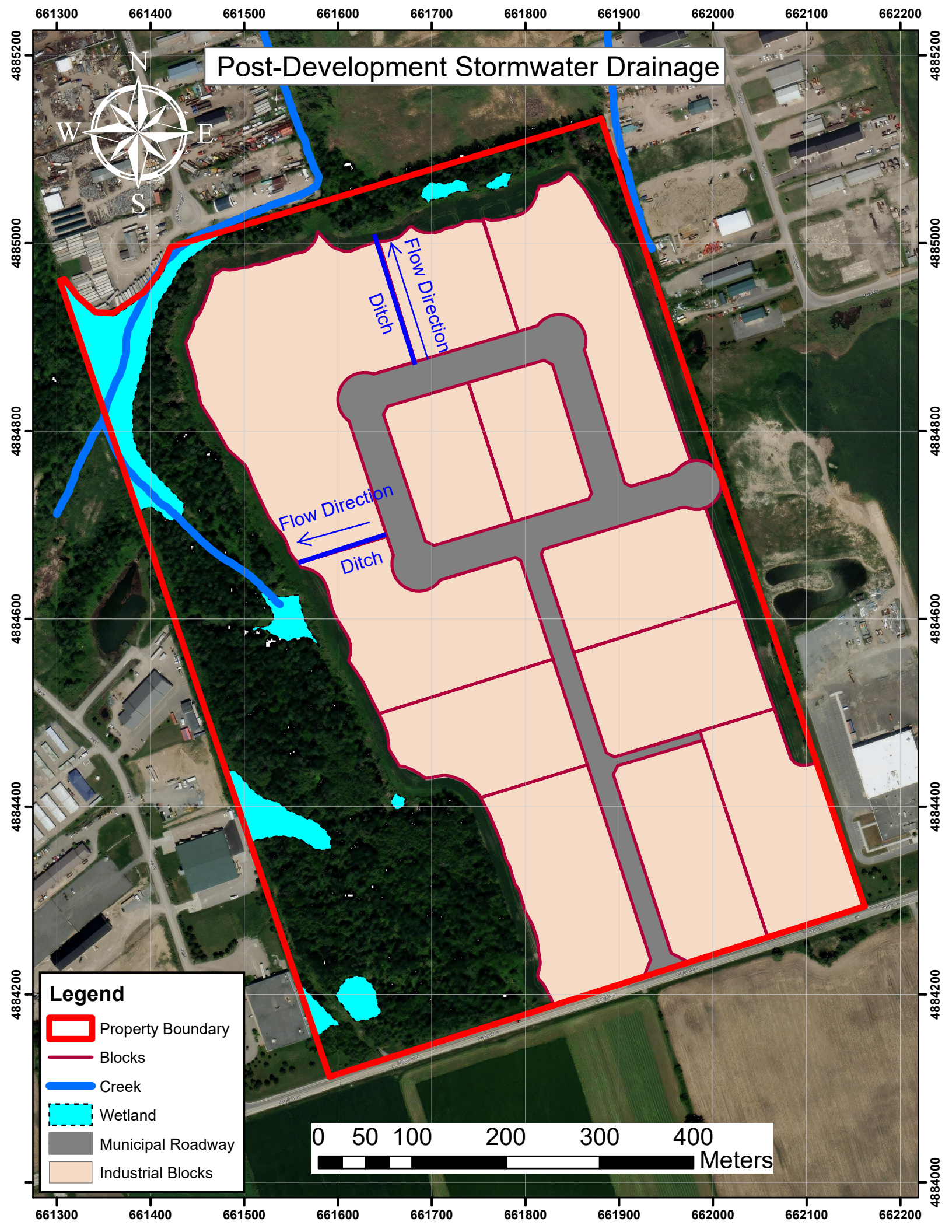


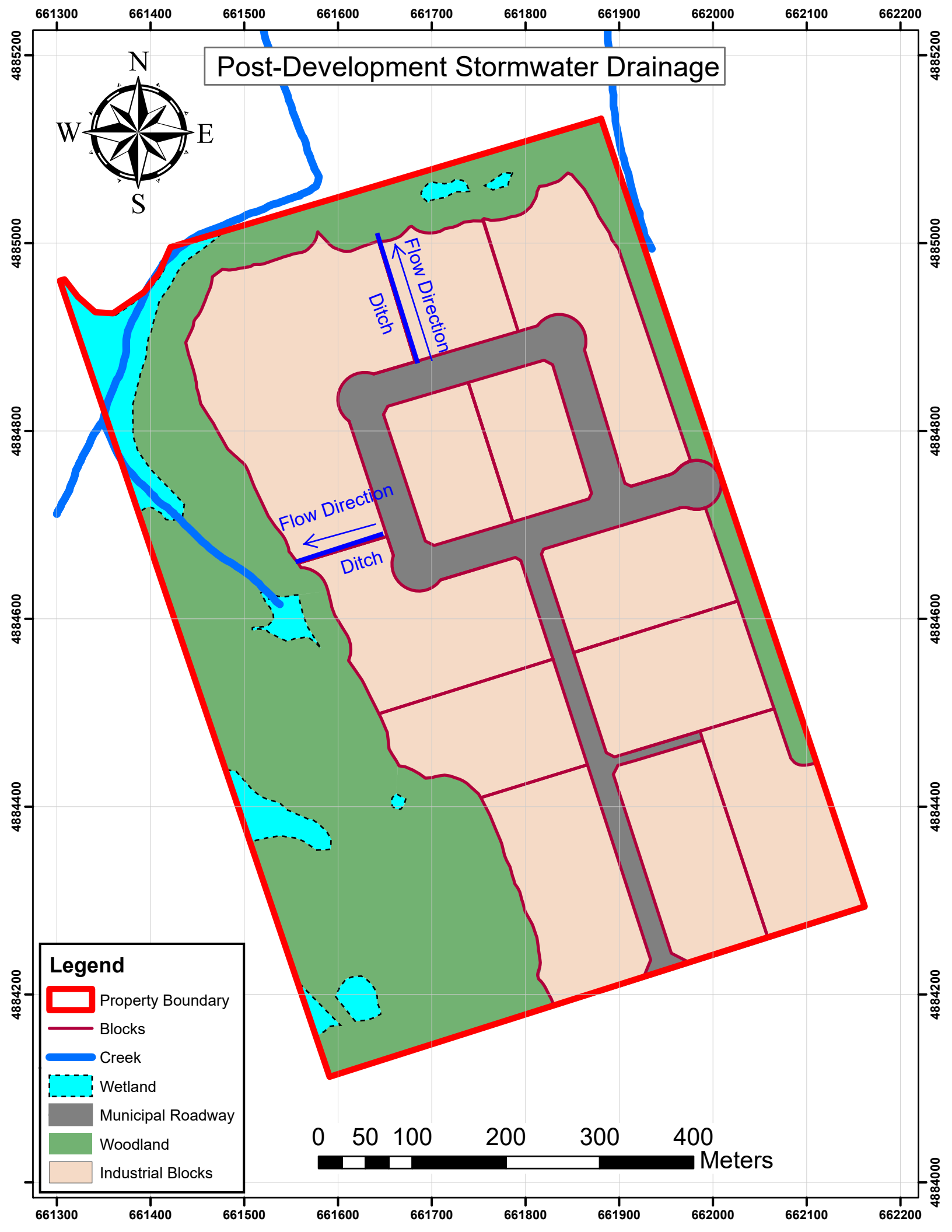
APPENDIX XIV– PRE- AND POST- DEVELOPMENT STORMWATER DRAINAGE NETWORK


Pre-Development Stormwater Drainage
N
W
E
S
Flow Direction
Flow Direction
Flow Direction
Flow Direction
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
Legend
Tributary
Property Boundary
Wetland
DEM
Elevation (m)
253.79 - 256.34
256.35 - 258.4
258.41 - 260.57
260.58 - 262.79
262.8 - 267.61
0 40 80 160 240 320
Meters

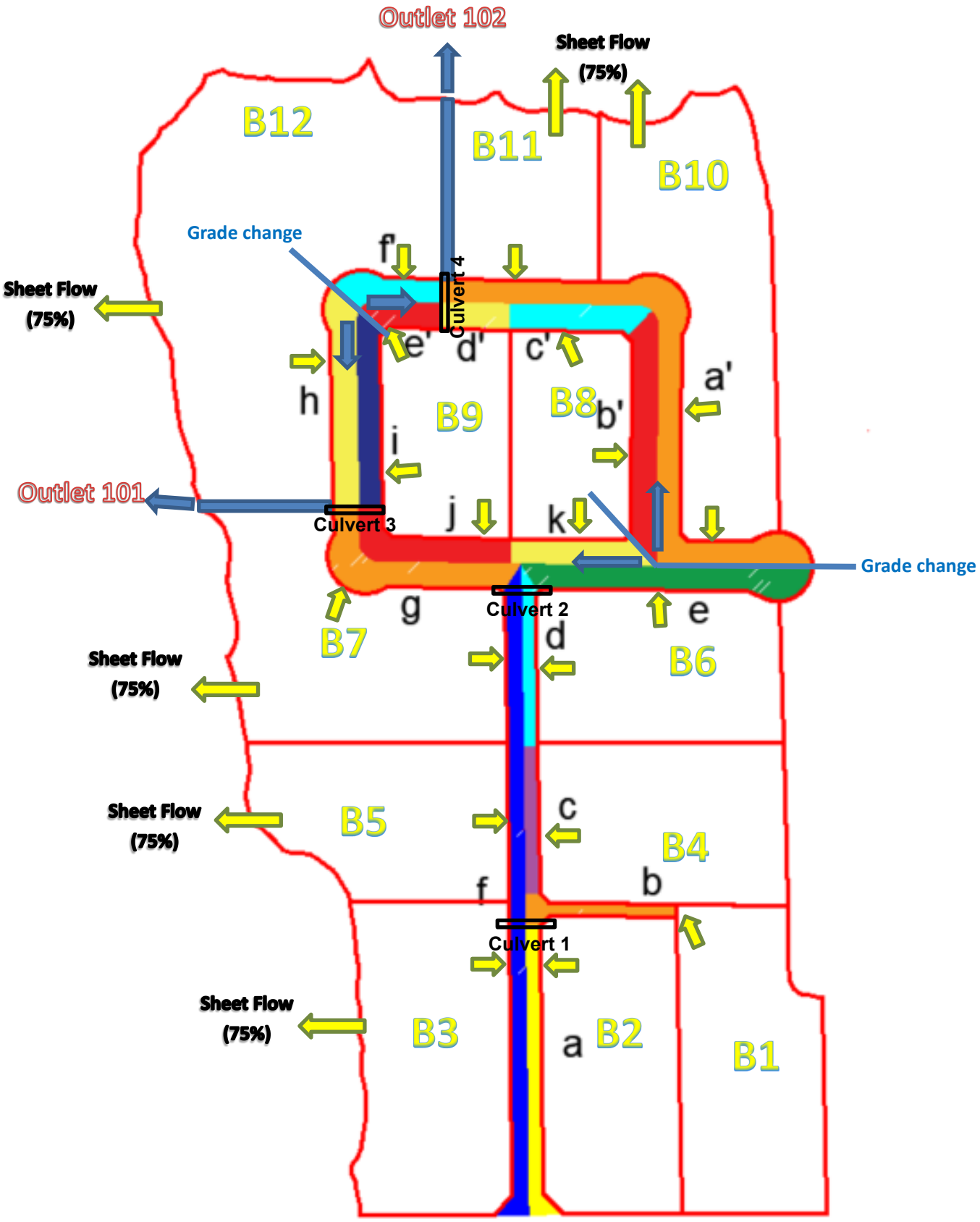

Pre-Development Stormwater Drainage
N
W
E
S
Flow Direction
Flow Direction
Flow Direction
Flow Direction
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
Legend
Tributary
Property Boundary
Wetland
DEM
Elevation (m)
253.79 - 256.34
256.35 - 258.4
258.41 - 260.57
260.58 - 262.79
262.8 - 267.61
0 4080 160 240 320
Meters


Pre-Development Stormwater Drainage
N
W
E
S
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
4884000
Legend
Property Boundary
Stream
Tributary
Sub-Catchment
1
2
3
4
5
6
0 4590 180 270 360
Meters

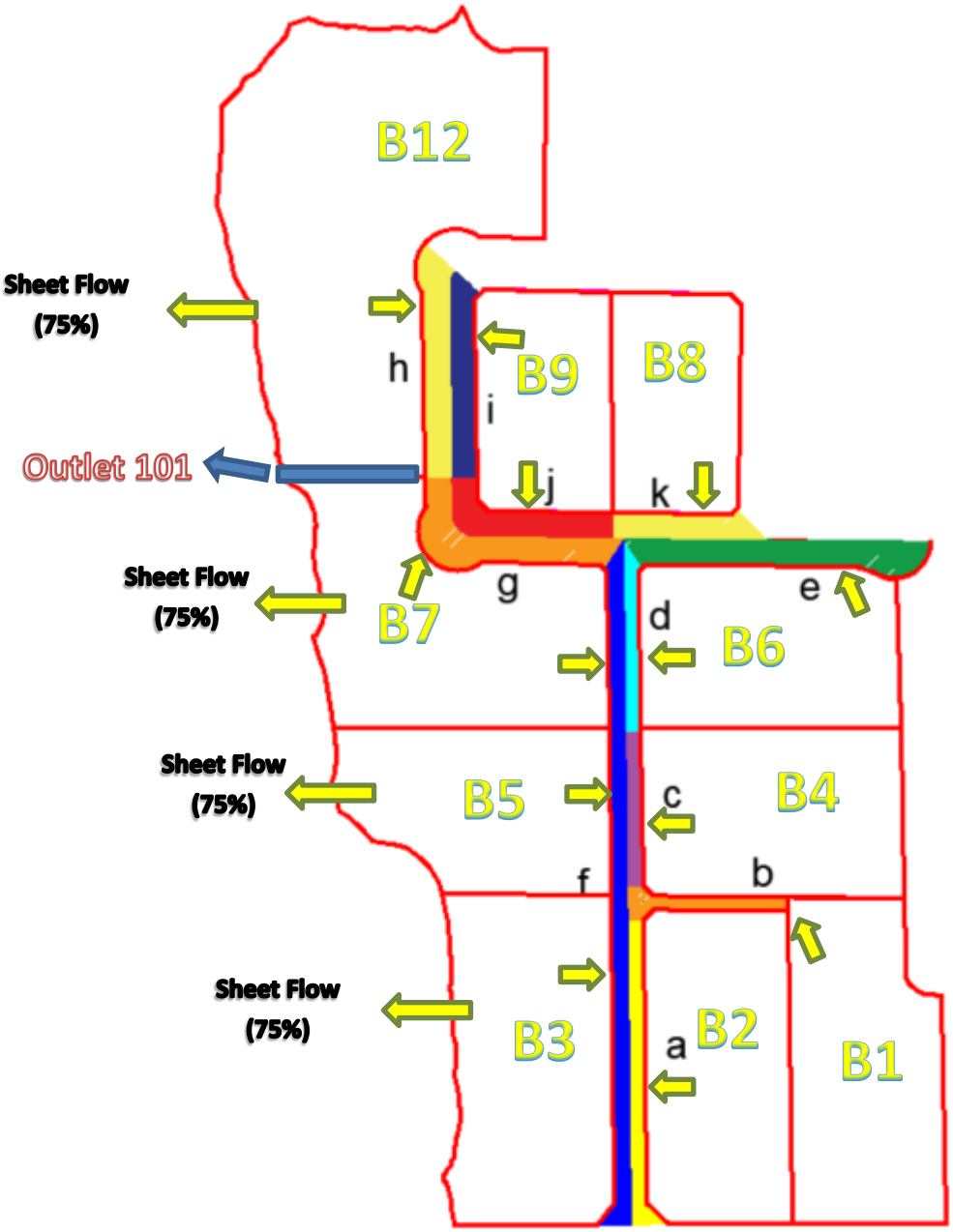

Post-Development Stormwater Drainage
N
W
E
S
Flow Direction
Ditch
Flow Direction
Ditch
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
4884000
Legend
Property Boundary
Blocks
Creek
Wetland
Municipal Roadway
Industrial Blocks
0 50 100 200 300 400
Meters


Post-Development Stormwater Drainage
N
W
E
S
Flow Direction
Ditch
Flow Direction
Ditch
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
4884000
Legend
Property Boundary
Blocks
Creek
Wetland
Municipal Roadway
Woodland
Industrial Blocks
0 50 100 200 300 400
Meters

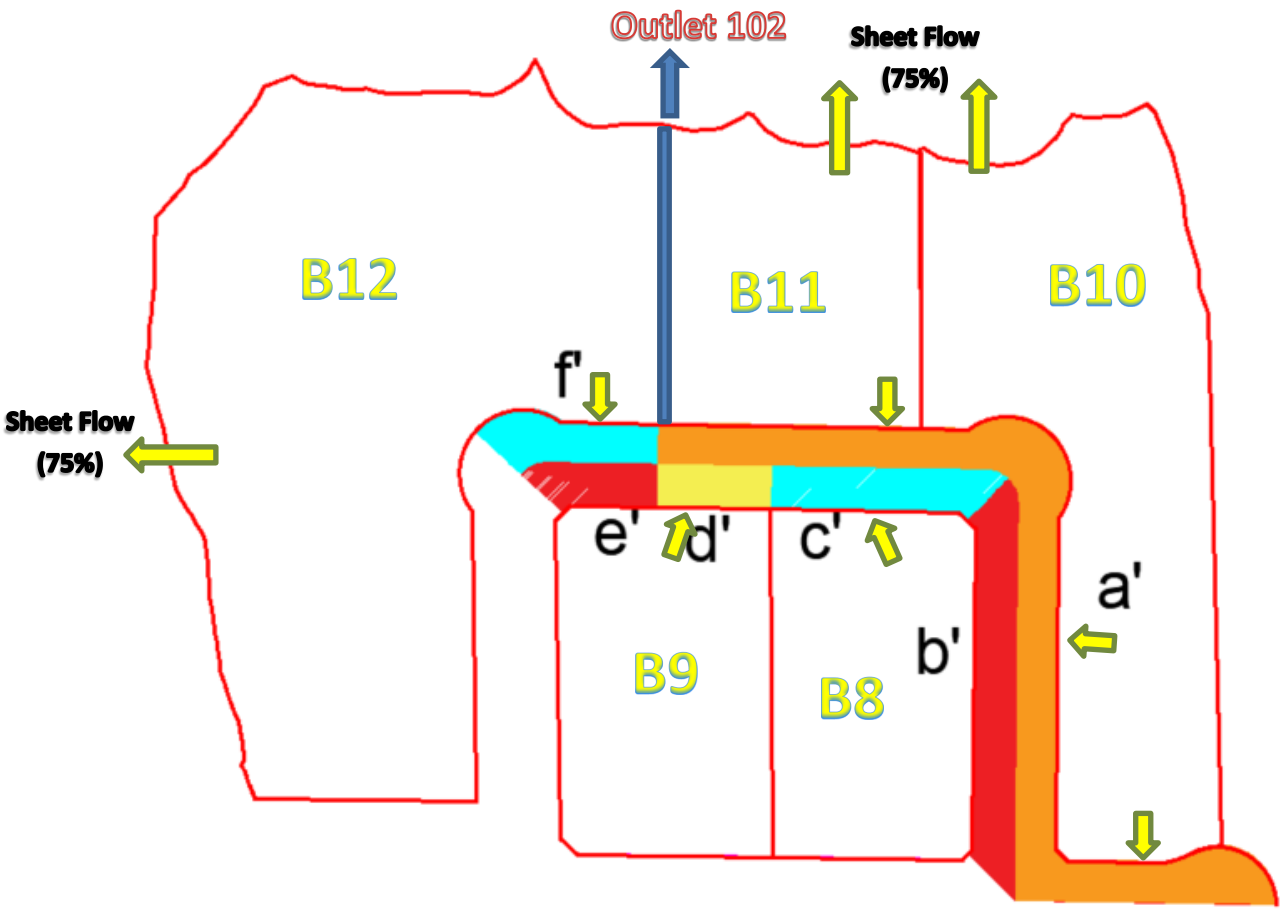
Post- Development Stormwater Drainage



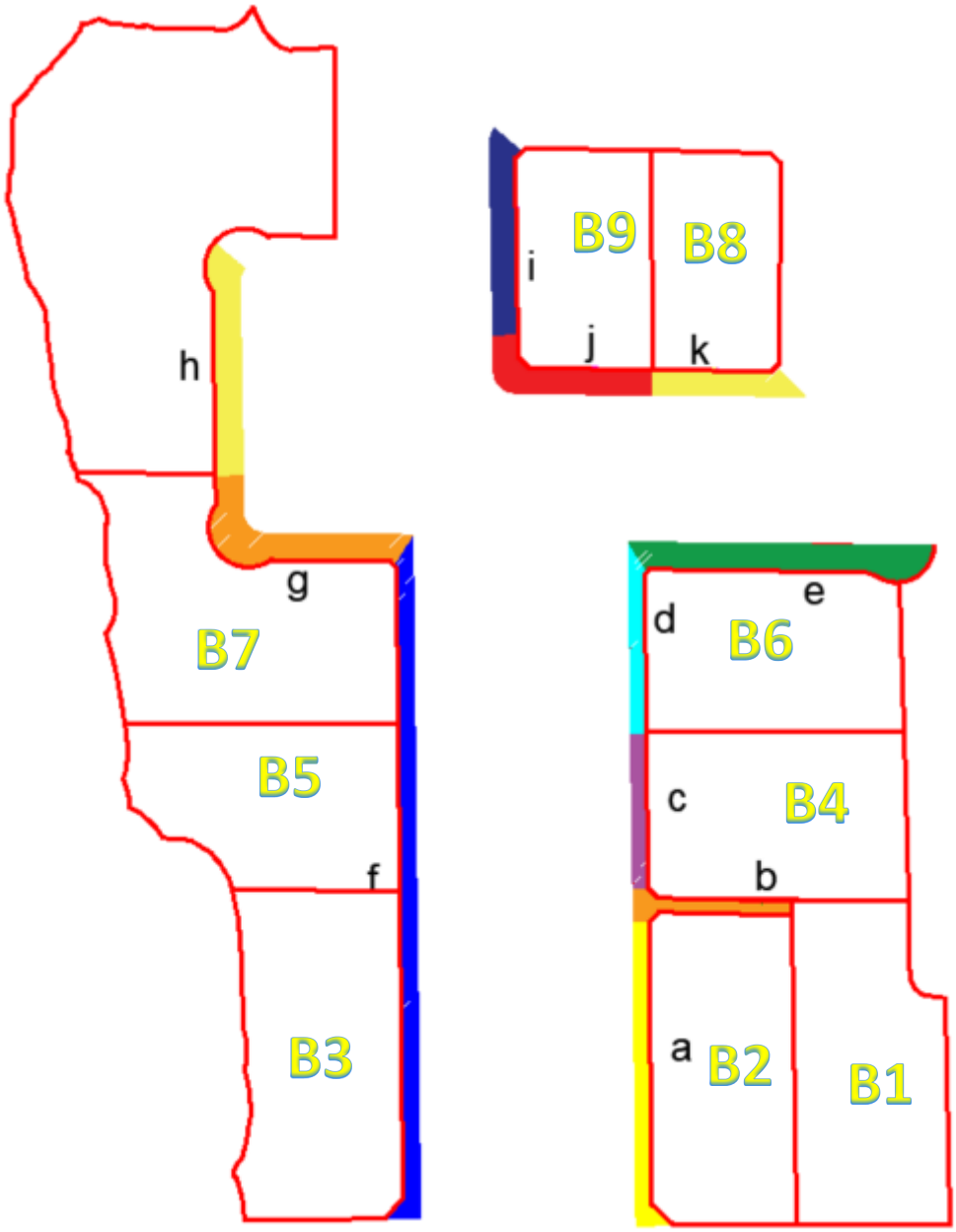
Drainage Layout for Driveway sub-catchments, conveyance ditch and industrial outlets



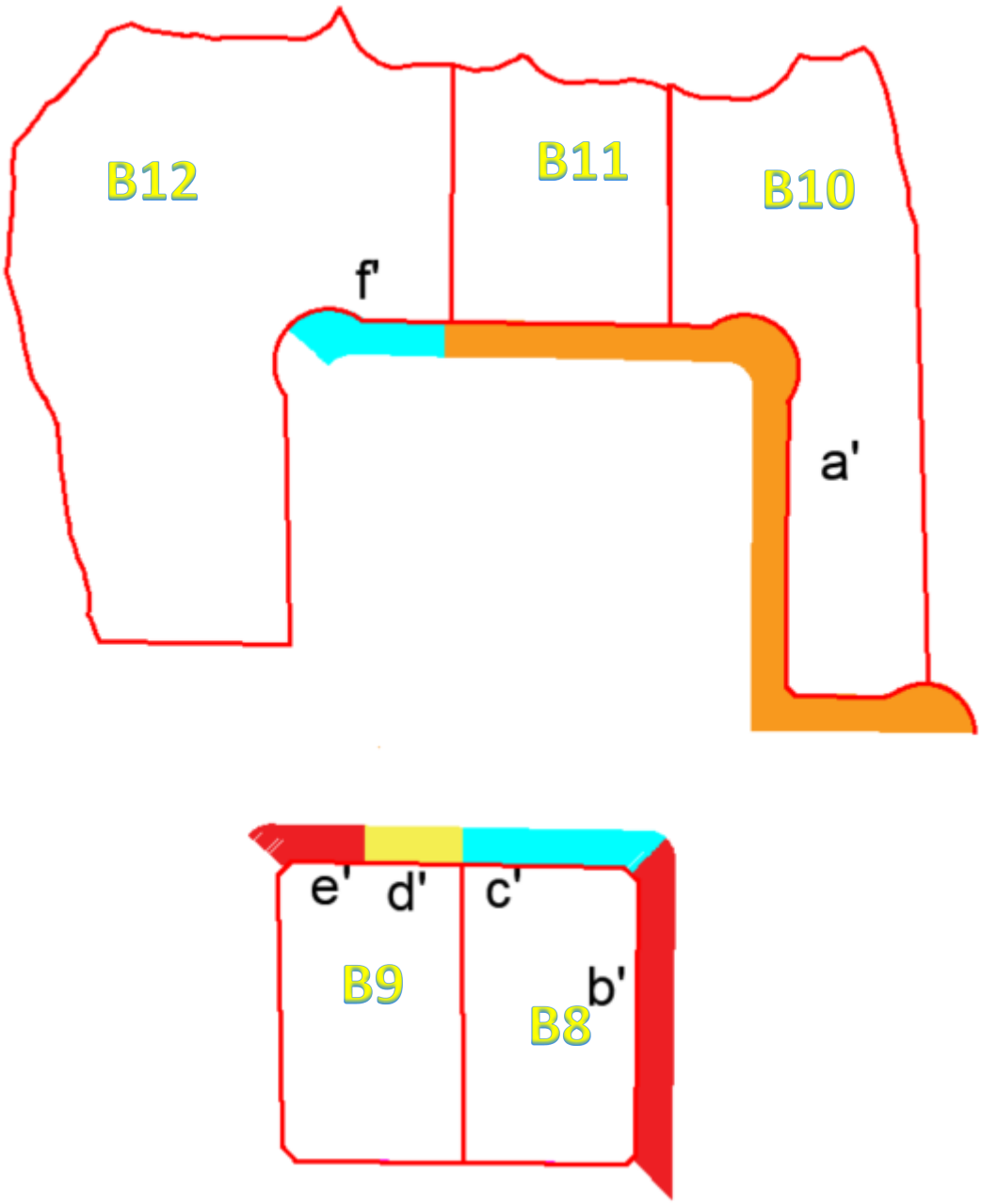
Drainage Layout –Southern Portion (Catchment 101)



Drainage Layout – Northern Portion (Catchment 102)



Main streams – Southern Portion



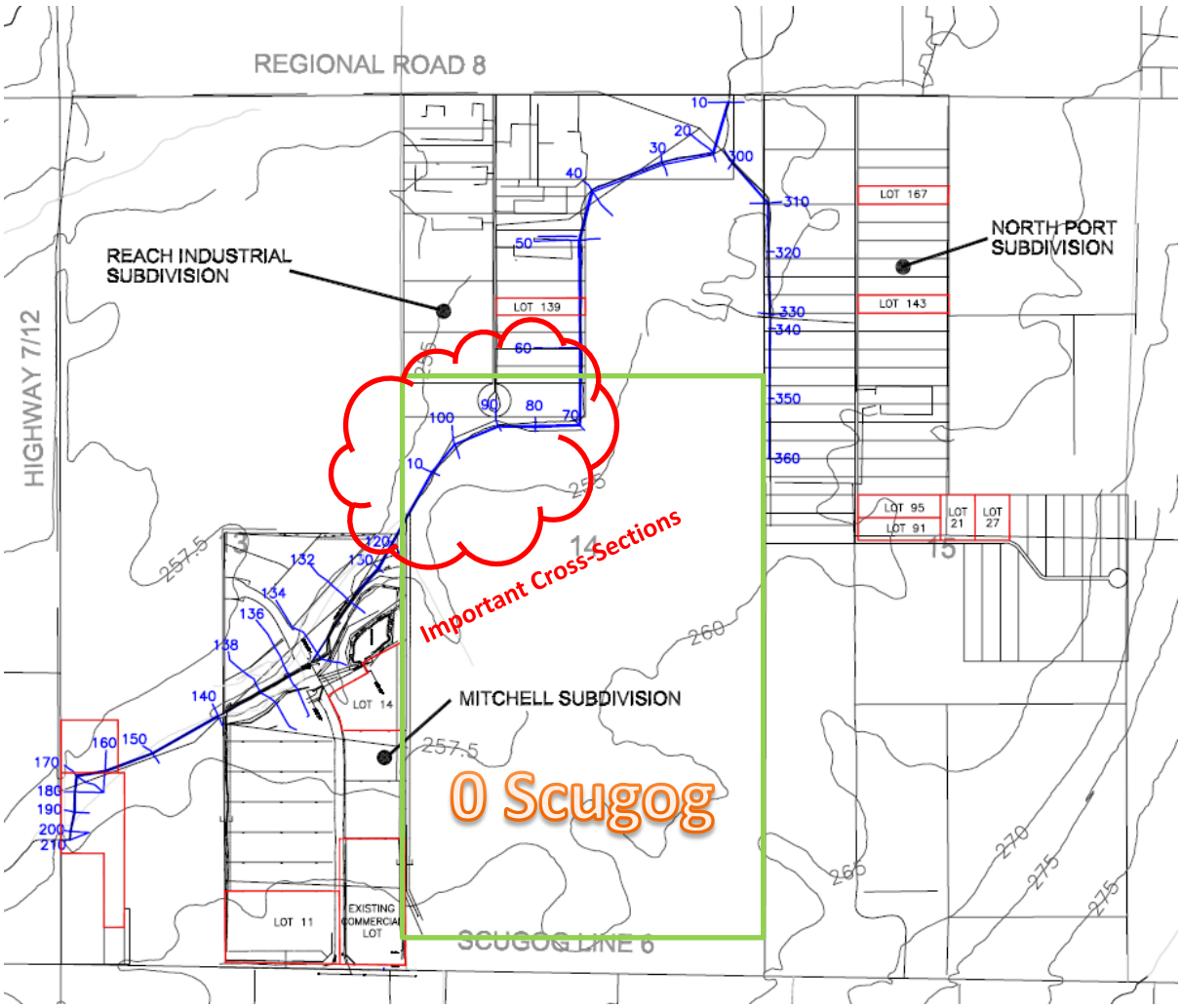
Main streams – Northern Portion

APPENDIX XV–FLOODING REVIEW/FLOOD INUNDATION MAP

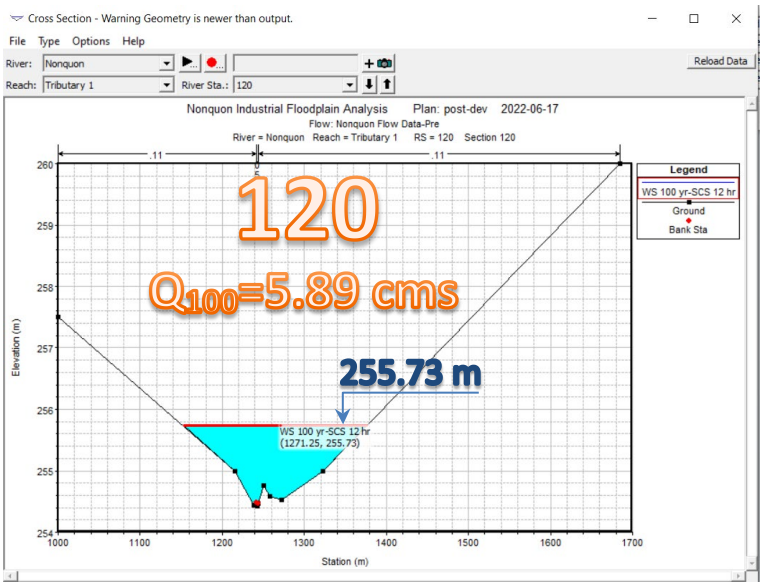
Flooding Review- 0 Scugog Line 6

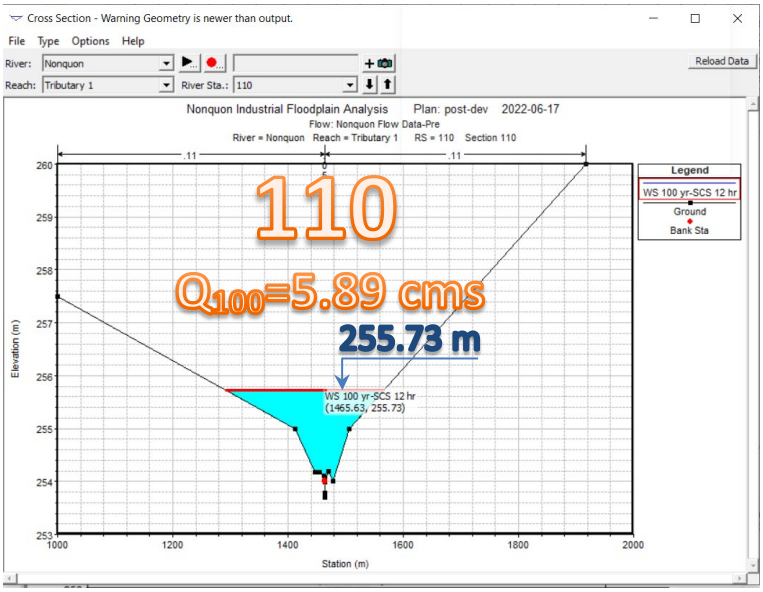
Cited in:

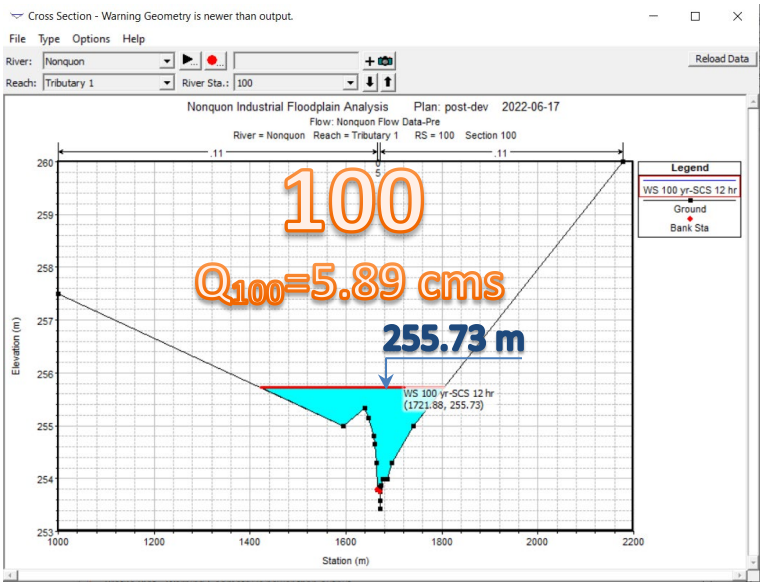
1. Greenland International Consulting Ltd. & Stantec Consulting Ltd. (2005). Nonquon River Subwatershed Study and Nonquon Industrial Tributary Area, Master Drainage Plan.
2. Hec-Ras model developed by the last firms.



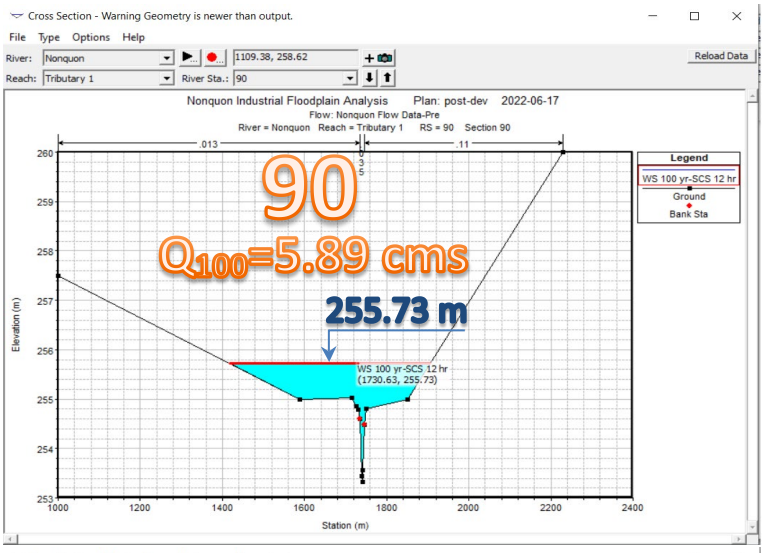
Hec-Ras Model- Cross Section locations

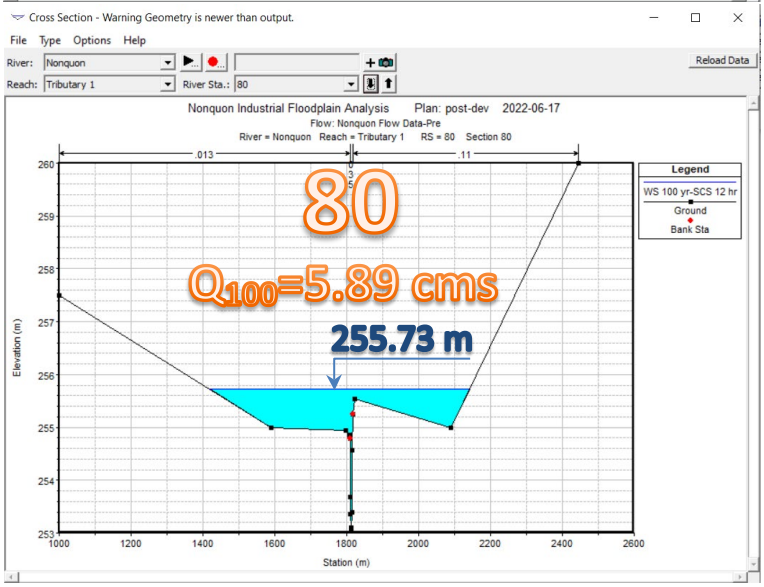


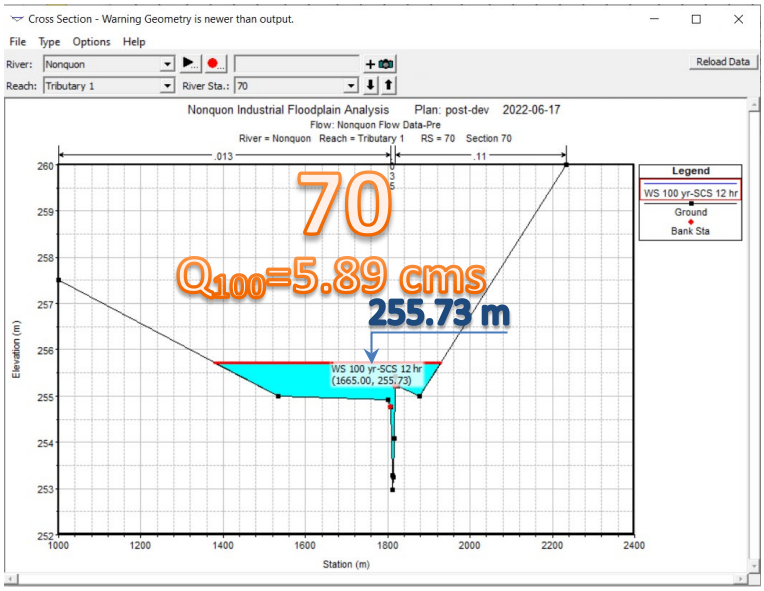




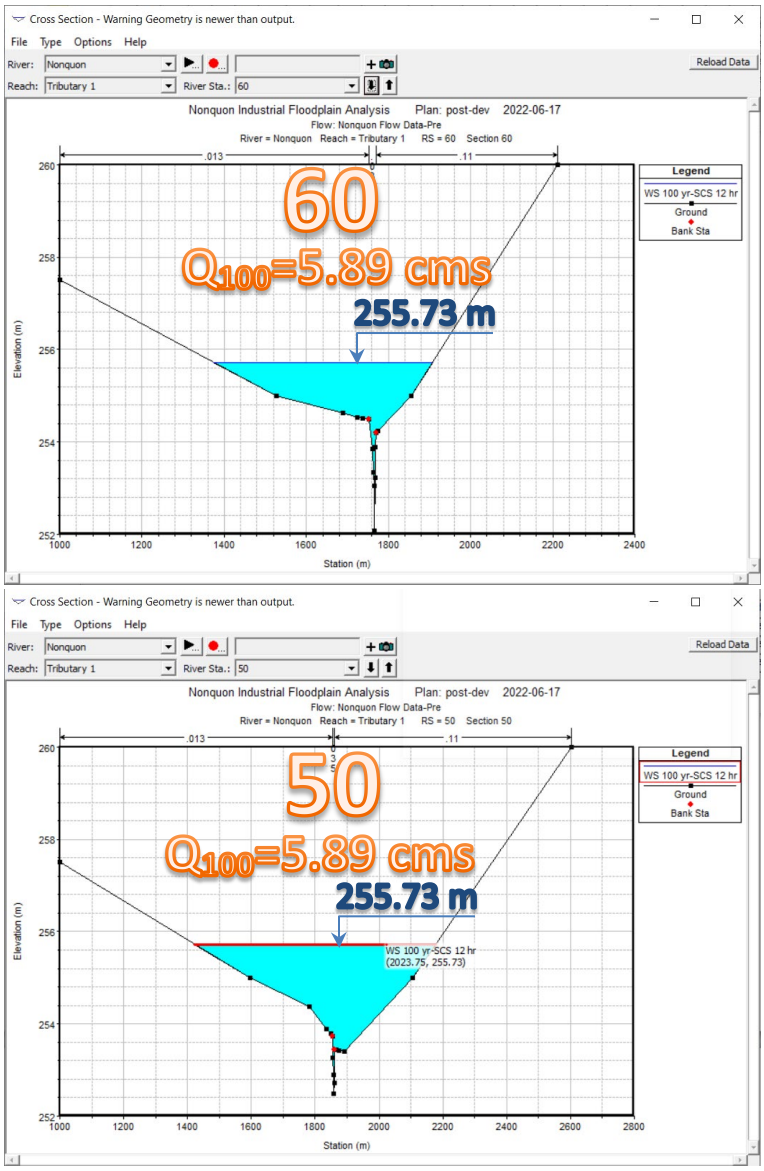
Important Cross Sections in the northwest corner



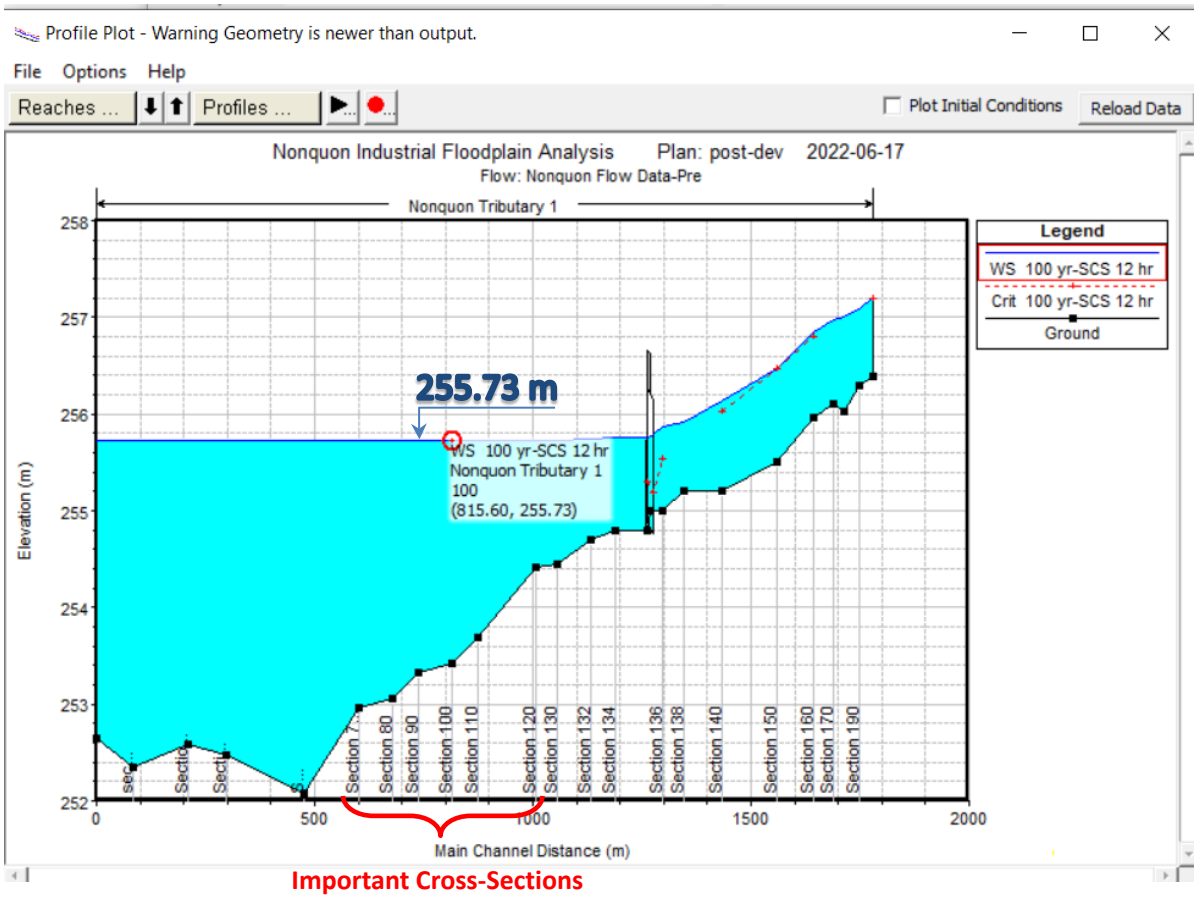




Important Cross Sections in the northwest corner



Important Cross Sections in the northwest corner



Important Cross-Sections

Longitudinal profile (100 year)

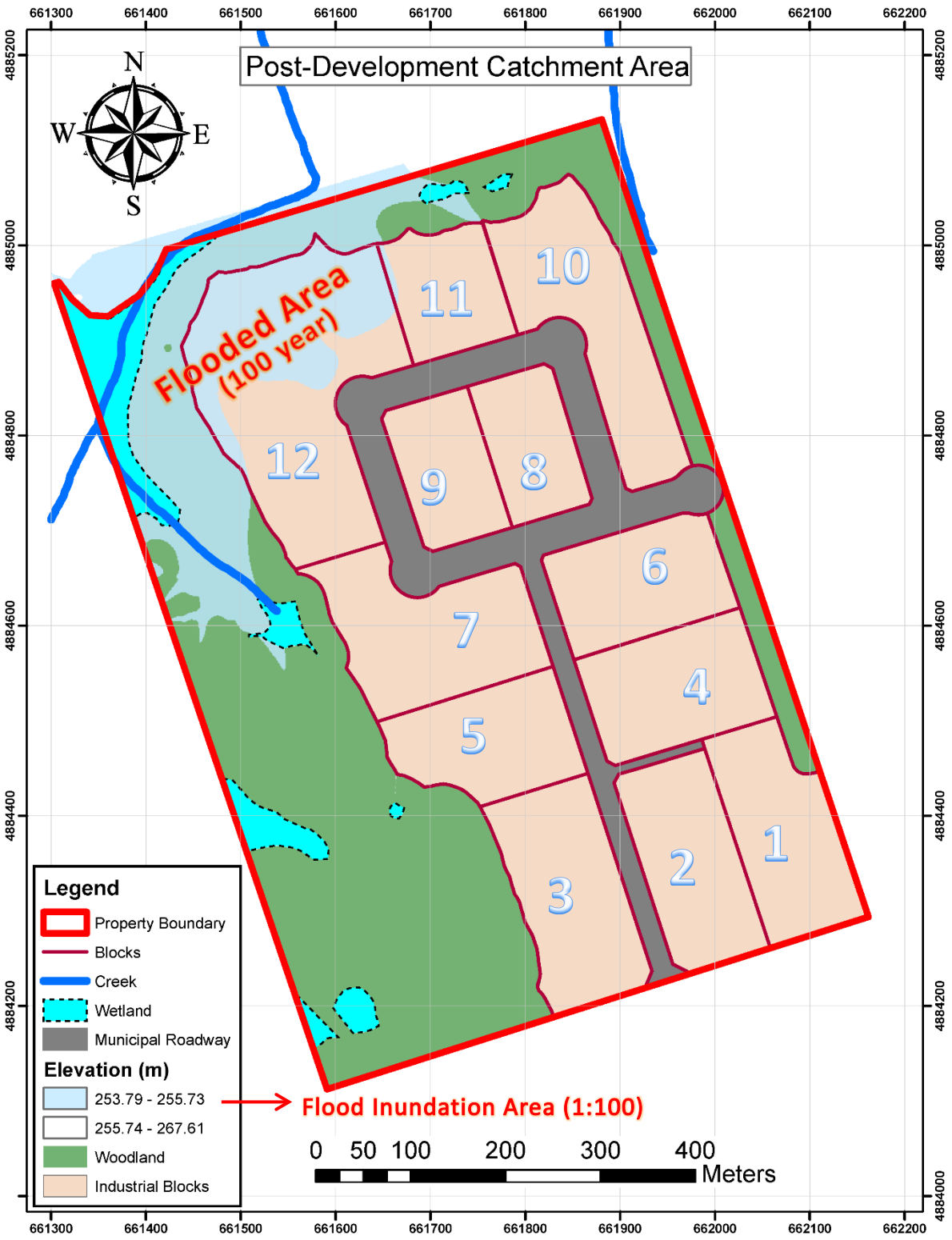

Post-Development Catchment Area
N
W
E
S
Flooded Area
(100 year)
10
11
12
9
8
6
7
4
5
3
2
1
Legend
Property Boundary
Blocks
Creek
Wetland
Municipal Roadway
Elevation (m)
253.79 - 255.73
255.74 - 267.61
Woodland
Industrial Blocks
Flood Inundation Area (1:100)
0 50 100 200 300 400
Meters
661300
661400
661500
661600
661700
661800
661900
662000
662100
662200
4885200
4885000
4884800
4884600
4884400
4884200
4884000

Table 5.10 Reduction in 5-year Event Water Surface Elevations for Post-Development Scenario

NITA Tributary / Cross-section	For Low and High Downstream Water Surface Conditions					
	Water Surface Elevation Post-Development (m)		Water Surface Elevation Pre-Development (m)		Reduction (m)	
	Low	High	Low	High	Low	High
Tributary '1'						
200	256.96	256.96	257.1	257.1	0.14	0.14
190	256.78	256.78	256.92	256.92	0.14	0.14
180	256.66	256.66	256.86	256.86	0.2	0.2
170	256.58	256.58	256.81	256.81	0.23	0.23
160	256.33	256.33	256.59	256.59	0.26	0.26
150	256.01	256.01	256.26	256.26	0.25	0.25
140	255.8	255.8	255.97	255.97	0.17	0.17
138	255.66	255.66	255.77	255.77	0.11	0.11
136	255.31	255.31	255.45	255.45	0.14	0.14
134	255.27	255.27	255.38	255.38	0.11	0.11
132	254.95	254.95	255.05	255.07	0.1	0.12
130	254.76	254.76	254.92	254.88	0.16	0.12
120	254.69	254.69	254.89	254.79	0.2	0.1
110	254.21	254.21	254.86	254.73	0.65	0.52
100	254.06	254.06	254.86	254.72	0.8	0.66
90	253.78	253.78	254.85	254.7	1.07	0.92
80	253.71	253.71	254.84	254.69	1.13	0.98
70	253.61	253.61	254.84	254.68	1.23	1.07
60	253.47	253.47	254.84	254.68	1.37	1.21
50	253.4	253.4	254.84	254.68	1.44	1.28
40	253.37	253.37	254.84	254.68	1.47	1.31
30	253.35	253.35	254.84	254.68	1.49	1.33
Tributary '2'						
360	257.16	257.16	257.36	257.36	0.2	0.2
350	256.54	256.54	256.78	256.78	0.24	0.24
340	255.81	255.81	256.05	256.05	0.24	0.24
330	255.15	255.15	255.4	255.43	0.25	0.28
320	254.65	254.65	254.93	254.87	0.28	0.22
310	254.13	254.13	254.83	254.67	0.7	0.54
300	253.12	253.12	254.84	254.68	1.72	1.56