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STORMWATER MANAGEMENT BRIEF

AT

**0TH FORKS ROAD,
CITY OF PORT COLBORNE, ONTARIO**

PREPARED FOR:

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June 28th, 2025

Table of Contents

1. Site Background.....	3
2. Site Investigation & In-Situ Permeameter Testing.....	6
2.1. Borehole Drilling	6
2.2. In-Situ Permeameter Testing.....	6
3. Pre-development site conditions	7
3.1. Topographic Elevation & Base Precipitation.....	7
3.2. Vegetation & Evapotranspiration	7
3.3. Precipitation Surplus (Recharge + Runoff).....	8
3.4. Stormwater Run-off	8
4. Stormwater Management Plan	10
4.1. Design Criteria	10
4.2. Proposed SWM Plan	11
4.3. Grass Swale.....	13
4.4. Volume control	14
5. Water Quality	15
5.1. Total Suspended solids	15
6. Reliance & Signature	16
7. References	17
APPENDIX I – SITE SURVEY PLAN	18
APPENDIX II – SITE GRADING & SWM PLAN	19
APPENDIX III – BOREHOLE DRILL LOG.....	20
APPENDIX IV – FIELD PERMEABILITY TEST.....	21
APPENDIX V –SIEVE TEST RESULTS	22
APPENDIX VI –CLIMATE DATA TABLE.....	23
APPENDIX VII –SUBWATERSHEDS WITHIN THE SITE.....	24
APPENDIX VIII–PRE- AND POST-DEVELOPMENT CATCHMENT AREA.....	25
APPENDIX IX – RUNOFF COEFFICIENT CALCULATIONS	26
APPENDIX X – STORMWATER MANAGEMENT CALCULATIONS.....	27

1. Site Background

King EPCM (the Engineer) was retained by Xue Ju Zheng (the Client) to carry out civil engineering designs for a proposed industrial development, including the creation of a Stormwater Management Brief. The property was located at 0th Forks Road, Part of Lots 23 to 25, Con 4, East of the Welland Canal, City of Port Colborne, Ontario (the Site). It is understood that the SWM is for the sole purpose of the application and construction of an industrial development (recycling construction waste, indoor/outdoor processing, outdoor storage, offices) with a trailer park on site. This report is to be submitted to the City of Port Colborne, Niagara Peninsula Conservation Authority (NPCA), and the Regional Municipality of Niagara (Niagara Region).

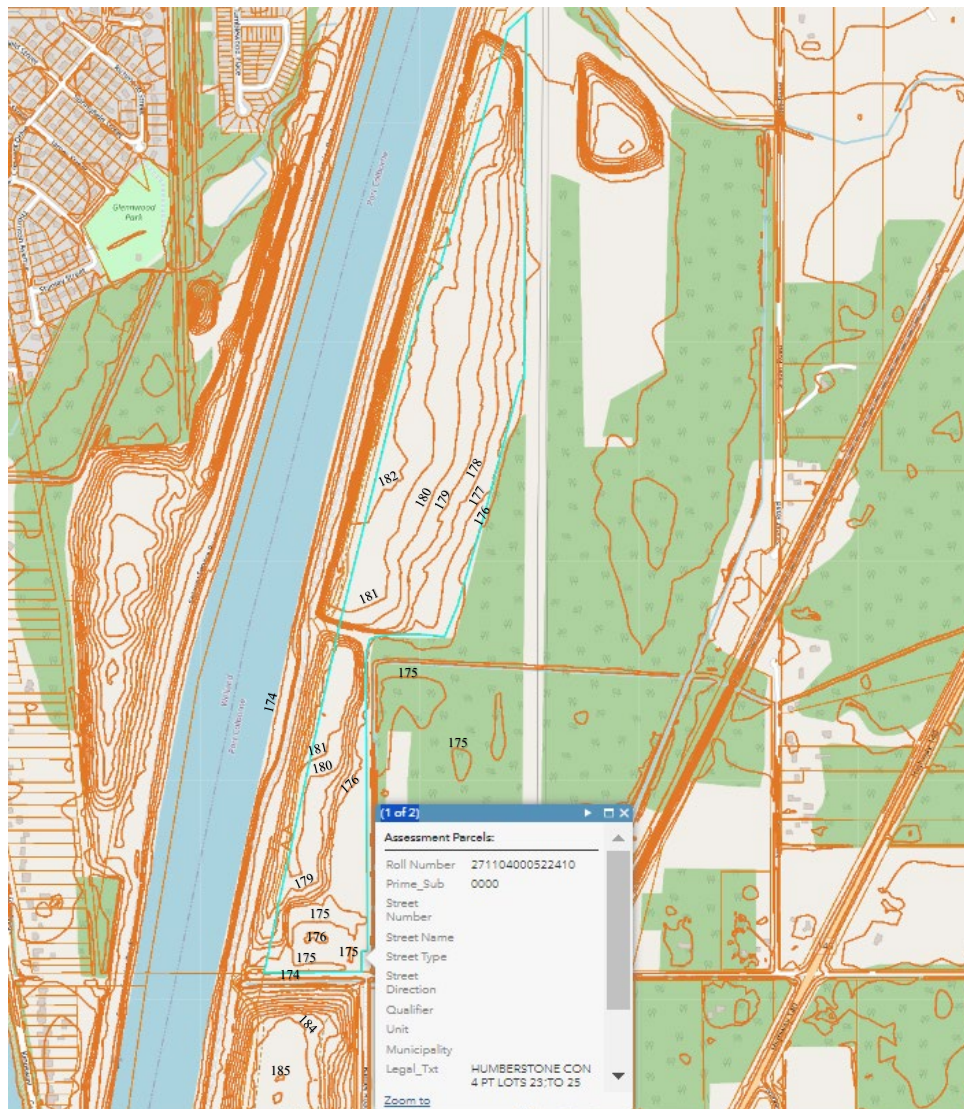


Figure 1 – Topographic map of Subject Area at 0th Forks Rd, Port Colborne, ON

The south portion of the parcel is in the watershed of the Indian Creek Municipal Drain and the north portion of the parcel is in the watershed of the Lyons Creek Municipal Drain. The north portion of the property is not abutting the Forkes Road allowance and has no access to it. The south limit of the parcel abuts the municipal drain.

The Site is considered to be dry industrial, using limited water supported on the property and designated as Rural Employment in the City's Official Plan. The Site is currently vacant, with two concrete pads and a gravel driveway in the southern portion.

The site property is considered irregular in shape, measures approximately 33.1 ha (Developed area = 2.17 ha), and is situated northwest of the intersection of Third Concession Rd and Ramey Rd, east of Welland Canal and Canal Rd, Port Colborne, Ontario. The Site is on industrial land use, outside of the designated urban area boundary of Port Colborne, and adjacent to a variety of natural heritage features mapped by Niagara Region as part of their Natural Environment System, including wetlands, woodlands, a shoreline area, permanent stream, and fish habitat, along with farmland in the southeast. The property is also mapped as part of the Provincial Natural Heritage System (PNHS). As such, these features are considered Key Natural Heritage Features (KNHF) and Key Hydrologic Features (KHF).

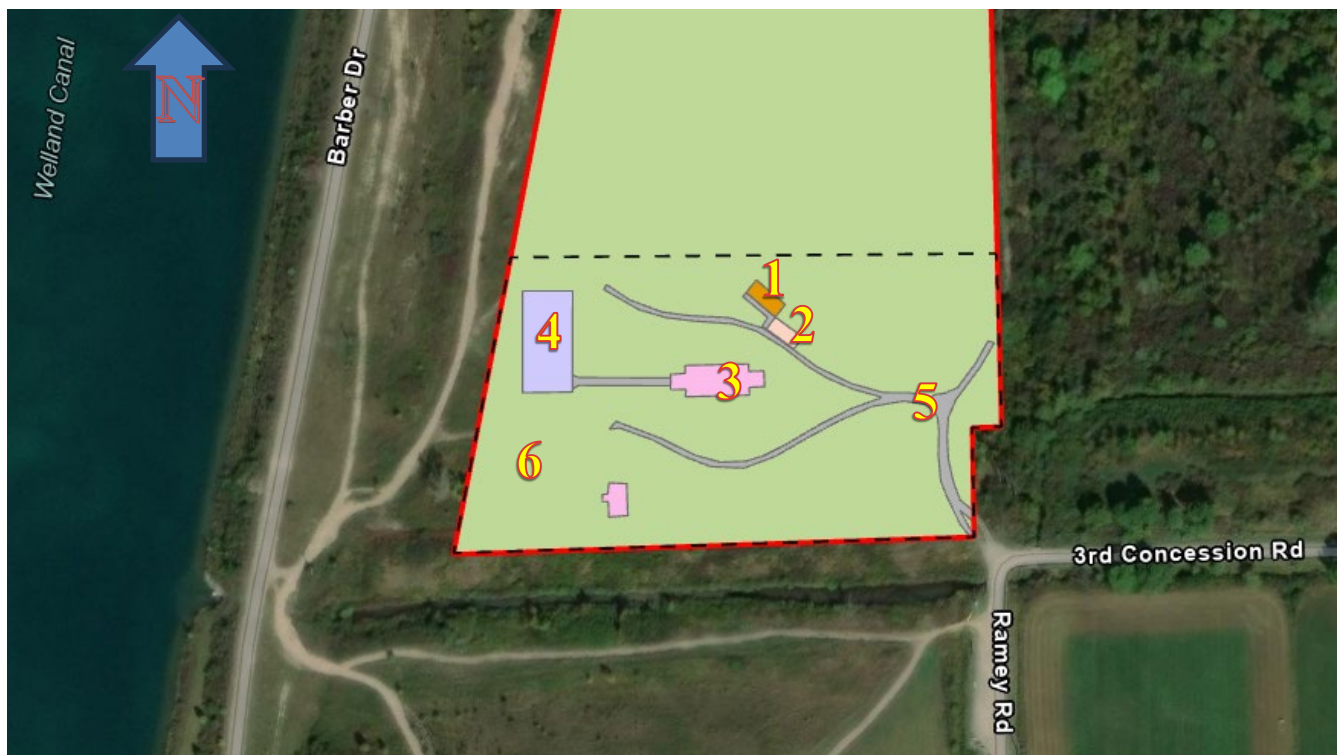


Figure 2 –Site Location Plan

In the above figure, the proposed developments within the site boundary developed area are displayed which are (Total Site area = 330,929.4 m² while the southern developed area = 21,656.3 m²):

- #1: Building (TIMP= 93.0 m²)
- #2: Parking (TIMP= 74.3 m²)
- #3: Concrete Pad (TIMP= 459.7 m²)
- #4: Storage Area (TIMP= 746.0 m²)
- #5: Gravel Driveway/Walkway (TIMP= 994.3 m²)

and the existing pervious areas which are located outside of the new proposed development within the site boundary:

- #6: Parkland (Grass and trees) = 19,289.0 m²



Figure 4–NPCA Regulated Area map, Subject Area at 0th Forks Rd, Port Colborne, ON

Since more than 31 ha of the site area is located outside of the proposed development and are Provincial Natural Heritage System (PNHS), such as Key Natural Heritage Features (KNHF) and Key Hydrologic Features (KHF), no SWM facilities will be proposed for those portions and the effective study area for the subject property would be 21,656.3 m².

2. Site Investigation & In-Situ Permeameter Testing

2.1. Borehole Drilling

One (1) shallow borehole (BH101) was drilled at the site property by King EPCM on March 5, 2024, on the west side of the southern portion of the property, to determine the septic infiltration rate of the soil. The detailed borehole drill log is in Appendix III, and shows the soil is comprised of moist brown silty clay without any groundwater.

2.2. In-Situ Permeameter Testing

Based on a field visit dated March 5, 2024, "field-saturated" hydraulic conductivity, K_{fs} , was achieved using the "Constant Head Well Permeameter" (CHWP) method. K_{fs} was conducted in the west, near BH101 using the ETC Standard Pask Permeameter Apparatus. The "Constant Head Well Permeameter" (CHWP) method was described in Appendix IV in detail.

It is understood that the in-situ infiltration test was not tested at the actual LID/septic bed bottom but based on borehole soil sample, it is in the Engineer's opinion as a geotechnical engineer that the soils perform similarly in hydrological infiltration potential.

The ETC Pask Permeameter was 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 K_{fs} is calculated as:

$$K_{fs101} = 2.5E-8 \text{ m/sec} = 2.5E-6 \text{ cm/sec}$$

K_{fs} were then corrected for temperature (for soil temperature=20°C):

$$K_{a101} = 1.61E-8 \text{ m/sec} = 1.61E-65 \text{ cm/sec}$$

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:

$$PT = 39.2 \text{ min / cm} \quad (\text{Infiltration Rate} = 15.3 \text{ mm/hour})$$

The engineer's opinion is to trust the values obtained from the OMMAH (1997), with an unfactored surface infiltration rate of 15.3mm/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 6.1 mm/hour, with a safety factor of 2.5.

See Appendix IV for more details, the calculations, and the graphs provided.

3. Pre-development site conditions

Prior to development, the property is considered nearly vacant with a Total Impervious Surface Area of less than 6.0% (~0.13 ha) near the southern portion, with grasses & trees for the remaining 94.0%. The site conditions prior to development can be broken down into several groups of information:

3.1. Topographic Elevation & Base Precipitation

- The site topography, ground cover, land use, and drainage patterns of the subject property were established through site visitation, interpretation of topographic maps, historical aerial photographs, and a topographic survey.
- The Site is situated in the physiographical region known as Haldimand clay plain along the shoreline of Lake Erie, where an intermixture of lacustrine clay and glacial clay till occurs above a shallow limestone bedrock of Bois Blanc formation.
- Site survey & topographic information are in Appendix I.
- The south portion of the site is in the watershed of the Indian Creek Municipal Drain and the north portion is in the watershed of the Lyons Creek Municipal Drain, with Clay Hydrologic Soil Group D. The north portion of the property is not abutting the Forkes Road allowance and has no access to it while the south limit of the parcel abuts the municipal drain.
- There are several interior ditches surrounding the developed area in the southern portion, which collect runoff and direct them towards the Indian Creek Drain, along the south boundary.
- Base yearly precipitation is 1000.9mm/year (EC Welland-Pelham monitoring station-1977 to 2006, Climate ID: 6139445, Appendix VI)
- The general surface drainage in the southern portion is towards the existing municipal drain along the southern site boundary (Indian Creek Municipal Drain).

3.2. Vegetation & Evapotranspiration

- The site property is considered irregular in shape, measures approximately 31 ha, and is located on the east side of Welland Canal/Canal Road, northwest of the intersection of Third Concession and Ramey Road, City of Port Colborne.
- The subject land is mostly grassing fields (>70%) with several tree cover in the eastern half of the site property (>10%).
- 1.9 ha / 89.0% of the Site (developed area) is grassy lawn along with dense tree covers.
- Estimated site actual evapotranspiration is 583.3mm/year (Appendix VI)

3.3. Precipitation Surplus (Recharge + Runoff)

- Net surplus (precipitation surplus) = $1000.9 - 583.3 = 417.6$ mm/year for pervious area
- Recharge for pervious area based on MOEE Hydrogeological Tech Info, 1995, Table 2: Infiltration factors:
 - Topography (Flat land) = 0.3
 - Soil (clay mixed with gravel) = 0.15
 - Cover = 0.15 (Parkland)
 - MOE infiltration factor = 0.6 (Parkland)
 - Recharge rate for pervious area would be $0.6 * 417.6 = 250.6$ mm/year (Parkland)*

*** It is the engineer's opinion that all site runoffs in the southern portion which will be developed in the future, will be directed to the existing municipal drain and discharged into the Welland Canal.**

- Storm runoff from the Parkland area = $417.6 - 250.6 = 167$ mm/year
- Total area-weighted average storm runoff = 167 mm/year
- Total area-weighted average recharge = 250.6 mm/year
- Total area-weighted average precipitation surplus (recharge + runoff) = 417.6 mm/year.

3.4. Stormwater Run-off

- There are no city-maintained stormwater sewers, with stormwater generally infiltrated by clay soils or collected by the interior ditches and discharged to the Municipal Indian Creek Drain, along the southern boundary.
- The historical well records and a new monitoring borehole drilled by the engineer within the Site, near the proposed development, confirm that the seasonal groundwater level in this property is low.
- All minor & major storm site drainage is to be collected on-site and outlet towards the municipal drain (Indian Creek).
- Based on an in-situ infiltration test using ETC Pask Permeameter Apparatus, the infiltration rate of the upper clay layer mixed with gravel was $39.2 \text{ min/cm} = 0.26 \text{ mm/min} = 15.3 \text{ mm/hour}$.
- Native permeable infiltration rate should use a non-factored infiltration rate of 15.3 mm/hour because the predominant soil material of this site is composed of clay mixed with gravel to a depth of more than 1 meter and a factored engineering design infiltration rate of 6.1 mm/hour with a safety factor of 2.5 for infiltration purposes.
- According to the NPCA Stormwater Management Policies and Guidelines (Section 7.2), when estimating flows for designing SWM structures, modelling approach can be as simple as estimating peak flows with the Rational Method to sophisticated hydrologic and hydraulic computer models. The Rational Method is one of the earlier developed methods of calculating peak flows from the IDF curve. The Modified Rational Method also has been developed to improve the accuracy of the standard rational method to take account of the variable runoff coefficients in the catchment and the losses in rainfall and storage in the system.

NPCA recommended simple calculation method such as the Rational or Modified Rational Method when there is total development area of less than 100 hectares, and the Engineer have used 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 year, 1 in 5 year, 1 in 10 year, 1 in 25 year, 1 in 50 year and the 1 in 100 year design storm events, peak flow conditions are calculated as follows:
- Peak flow rate Modified Rational Formula: $Q_p = (0.001/3600) * A * C * C_a * i$
 - A = area in $m^2 = 21,656.3m^2$
 - C = runoff coefficient = 0.39 (Clay, parkland & grass, 6.0% TIMP, flat, MTO 1997 design chart 1.07 for rural & urban and Table 4.1, Niagara Region SWM Guidelines. See Appendix IX for more details).
 - C_a = Antecedent Precipitation Factor = 1.0 for 2, 5 and 10 year, 1.10 for 25 year, 1.20 for 50 year, and 1.25 for 100 year.
 - i = average rainfall intensity in mm/hour using the Rational Method with a,b,c values of 3-parameters design storm as: $Intensity = a/(t+c)^b$, Where, t is time of concentration in hour; a , b and c are constant (Table 1).
- The minimum time of concentration per City standards is 10 minutes which will be used in the following hydrological calculations for both pre and post-development scenarios.
- See Table 1: Pre-development peak flows for the 1:2-year through 1:100-year design storms based on the Modified Rational Method with a,b,c values of 3-parameters design storm as: $Intensity = a/(t+c)^b$ and $Q = 2.8CC_aIA$. The Modified Rational Method calculations are included in Appendix VI for reference.

Table 1 - Existing peak flows and rainfall intensities based on the Rational Method

Design Storm	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
a	755	830	860	900	960	1020
b	0.789	0.777	0.763	0.745	0.736	0.731
c	8	7.3	6.5	5.2	5.1	4.7
Intensity (mm/hr)	77.2	90.6	101.3	118.5	130.2	143.0
Total Runoff Q_p (l/s)	179.2	210.4	235.2	302.8	362.8	415.1

IDF Parameters	Return Period (year)					
	2	5	10	25	50	100
a	755	830	860	900	960	1020
b	0.789	0.777	0.763	0.745	0.736	0.731
c	8	7.3	6.5	5.2	5.1	4.7
Duration						
5 min	99.78	118.09	133.41	159.53	175.02	193.76
10 min	77.19	90.60	101.29	118.51	130.18	142.99
15 min	63.61	74.38	82.77	95.89	105.47	115.44
30 min	42.81	49.87	55.27	63.40	69.97	76.32
1 hr	27.05	31.53	34.97	40.05	44.41	48.40
2 hr	16.42	19.21	21.41	24.63	27.46	29.96
4 hr	9.74	11.47	12.87	14.93	16.74	18.30
6 hr	7.14	8.43	9.51	11.10	12.48	13.67
12 hr	4.17	4.96	5.64	6.66	7.53	8.28
24 hr	2.42	2.91	3.34	3.98	4.54	5.00

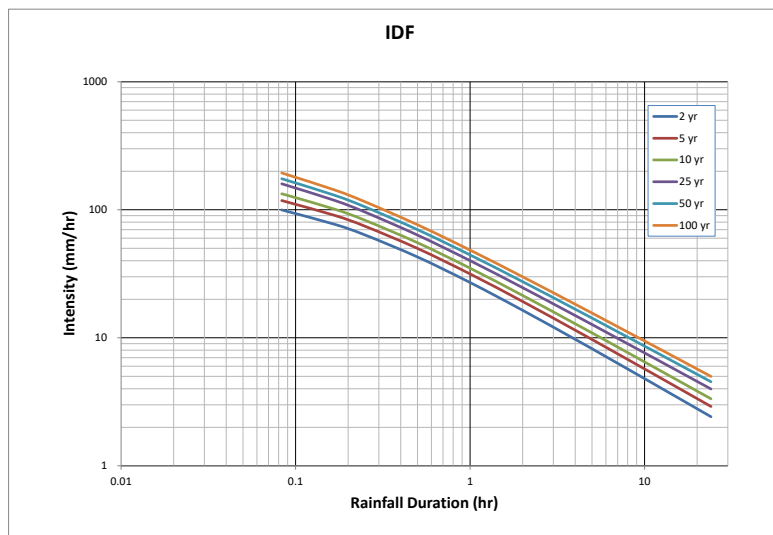


Figure 5- Intensity-Frequency-Duration (IDF) Curves calculated from Table 1 above

4. Stormwater Management Plan

The SWM plan has been prepared in accordance with the MOE Stormwater Management Planning and Design Manual, City of Port Colborne Standards and NPCA Stormwater Stormwater Management Policies and Guidelines as detailed in Section 12. The SWM plan is subject to review and approval by the City, NPCA 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 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;
- The storage requirements for the grass swale have been calculated based on Table 3.2 of the Stormwater Management Planning and Design Manual (MOE, 2003) to ensure it meets water quality treatment objectives; and
- Protect life and property from flooding and erosion.

4.2. Proposed SWM Plan

The Client has proposed the implementation of industrial development (recycling construction waste, indoor/outdoor processing, outdoor storage, offices) with a trailer park on site, mostly on the existing impervious concrete pad, on the southwest corner of the Site, while the existing site consists of primarily open, undeveloped lands to the north with sparse trees and shrubs and only the south portion of the site, directly adjacent to the intersection of Ramey Rd and Third Concession has been developed, with site property post-development Total Impervious Surface Area (TIMP) = 2367.3 m² / 10.9%.

Due to the low groundwater level in the historical boreholes and newly drilled borehole logs, no groundwater observed during borehole drilling in the last month, and low draining soil surrounding the site, the Engineer proposes that rooftop downspouts along with runoff captured by driveway/parking will be directed to the pervious grass area to allow infiltration of stormwater runoff generated by the impervious areas, and then directed towards the existing interior ditches including proposed check-dams through sheet flow, 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 municipal drain.

Below is a summary of the proposed site conditions:

- Total developed area = 21,656.3 m²
- Total pervious surface area (grassy lawn+trees) = 19,289 m² (89.1%)
- Total impervious surface area (TIMP) = 2367.3 m² (10.9%)
 - 93.0m² – new building
 - 841.3m² – existing driveways
 - 37.3m² – proposed walkway
 - 115.7m² – proposed driveway
 - 74.3m² – proposed parking lot
 - 459.7m² – existing concrete pad
 - 746.0m² – proposed outdoor storage
- The existing property is not serviced by any storm sewer or stormwater management facilities, except the main branch of the Indian Creek municipal drain along the southern boundary.
- All roof leaders will be discharged into the grassy lawn through splash pads and infiltrate into the ground.
- The runoff captured by the existing driveway/proposed parking/walkway will be directed into the grass area through the grading and if there is any overflow in less frequent events, the

water is directed to the interior ditches through the sheet flow and flows south towards the Indian Creek.

- Post-development peak flow conditions are calculated as follows:
- Peak flow rate Modified Rational Formula: $Q_p = (0.001/3600) * A * C * C_a * i$
 - A = area in m^2 = 21,656.3 m^2
 - C = runoff coefficient = 0.42 (Clay, grassy lawn & trees, 10.9% TIMP, flat, MTO 1997 design chart 1.07 for rural & urban and Table 4.1, Niagara Region SWM Guidelines, See Appendix IX for more detail)
 - C_a = Antecedent Precipitation Factor = 1.0 for 2, 5 and 10 year, 1.10 for 25 year, 1.20 for 50 year, and 1.25 for 100 year.
 - i = average rainfall intensity in mm/hour using the Rational Method with a,b,c values of 3-parameters design storm.
 - The minimum time of concentration per city standards is 10 minutes.
- See Table 2: Post-development peak flows for the 1:2-year through 1:100-year design storms based on the Modified Rational Method along with appropriate pre-development peaks flow for the developed catchment area. The Modified Rational Method calculations are included in Appendix X for reference.

Table 2 – Proposed peak flows based on the Modified Rational Method

Design Storm	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
Intensity (mm/hr)	77.2	90.6	101.3	118.5	130.2	143.0
Developed Catchment Runoff Q_p (l/s)	193.0 (179.2)	226.5 (210.4)	253.2 (235.2)	325.9 (302.8)	390.5 (362.8)	446.8 (415.1)
Allowable Runoff Q_p (l/s)	179.2	210.4	235.2	302.8	362.8	415.1

Note: (179.2) denotes pre-development peak flow.

• **LIDs Dimensions:**

- A Grass Swale started from the middle driveway to the north and then continue along the northern limit of the developed area and directed to the south towards the existing culvert which directed flows to the existing municipal drain. The swale has a 0.5 m depth, 1.2m bottom width, and 200m Length, with a static storage capacity of approximately 60 m^3 between check dams to control post-development peak flows to pre-development levels for

all storms up to and including the 100-year storm and for detaining 25 mm storm for 24 hours and (Fig 6).

- It can be concluded that the designed capacity of the proposed LID is equal to 60m³ and it can be able to manage most of the minor and major storms with 1:2 to 1:100 year return periods (See Appendix X for more details).

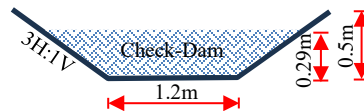


Figure 6- Proposed Enhanced Swale Cross-Sections

4.3. Grass Swale

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., roadways). A grassed swale is a graded and engineered landscape feature appearing as a linear, shallow, open channel with a trapezoidal or parabolic shape. The swale is vegetated with flood-tolerant, erosion-resistant plants.

Grass swales (also referred to as vegetated open channels or water quality swales) are conveyance channels engineered to capture and treat the water quality volume for a drainage area. They differ from a normal drainage channel or conventional swale because they incorporate specific features that enhance stormwater pollutant removal effectiveness. Swales are designed with limited longitudinal slopes to force the stormwater flow to be slow and shallow, thus allowing for particulates to settle and limiting the effects of erosion. Berms and/or check dams installed perpendicular to the flow path promote settling and infiltration.

In swales, stormwater is slowed by the strategic placement of check-dams, that encourage ponding, and these ponds in turn facilitate water quality improvements through infiltration, filtration, and sedimentary deposition. Collected stormwater is expected to drain away through the soil within several hours or days.

- Flatbed swales (Trapezoidal shape) with check dams along the western site boundary (for Retained Lot) and along the western/eastern site boundaries (for Severed Lot) and designed to receive a short duration of 4 hours 25mm storm event, per the City's standards. Total number of check dams will be three.
- Swales are designed at 0.5 m depth, 1.2 m bottom width at 3H:1V sloped sides, cross-sectional area of 1.35 m².
- Rockfill check dams with a pond height of 0.29m, and usually installed based on ditch grading along the swale (See Table 3 for details)
- The total static storage volume behind check dams (#3) is approximately 60 m³, and the dynamic infiltration volume per hour is 3.7 m³/hour from the total area of the channel bottom.
- Swales grade north, east and south at roughly 0.5%, 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.
- Swales collect stormwater from all impermeable surfaces (rooftops/pavements).

- Assumes 100% of rooftops and captured volumes from the pavements are infiltrated/recharged into groundwater and the rest would be directed to the downstream waterway.
- Drawdown time is 47.5 hours for the ponding water with a maximum water depth of 0.29 m with a conservative engineering infiltration rate of 6.1 mm/hour for the ditch bed.

$$\text{Drawdown Time (hr)} = 290 \text{ (mm)} / 6.1 \text{ (mm/hr)} = \underline{47.5 \text{ hr}}$$



Figure 7 – Sample Grass Swale with Check Dams (LID)

Table 3 – Design of Check Dam for Retainment

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)
200	0.5	58	3	60	176.0	3.7

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

** Water Volume Capacity = $(h^2 \times (h \times z + B)) / 2S$

4.4. Volume control

In Table 4 the total runoff volume control target for post development scenario (25mm storm event) and volume capacity of proposed LIDs are shown. As can be seen, the total volume of stormwater runoff produced due to a 25 mm rainfall event from the total impervious area of the developed area is 59.3 m³, which is less than the storage capacity intended for storm runoff at the site 60 m³ (only static storage capacity). The below table shows that runoff volume reduction is met, and the post-construction runoff volume shall be captured and retained on the site from a 25 mm rainfall event from the total impervious area.

Table 4 – Runoff volume control target (25 mm) and values provided by post-development design (m³/s)

Parameter	Developed Area					
	Building	Driveway/Walkway	Parking	Storage	Concrete Pad	Total
TIMP Area (m ²)	93	994.3	74.3	746	459.7	2367.3
25 mm runoff volume control (m ³)	2.33	24.9	1.9	18.7	11.5	59.3
Grass Swale LID volume capacity = 60m ³ (static storage capacity) versus a volume control target of 59.3m ³ .						

5. Water Quality

5.1. Total Suspended Solids

The development will result in an overall increase in impervious surfaces – including parking and rooftop areas – that will require quality control treatment in accordance with the Ministry of Environment, Conservation, and Parks (MECP) Stormwater Management Planning and Design Manual 2003. To achieve the water quality objectives, stormwater management best management practices (BMPs) have been selected for this project.

Stormwater quality controls will be required to implement in accordance with requirements of the Niagara Peninsula Conservation Authority (NPCA) with a targeted Total Suspended Solids (TSS) removal of 80% on an annual loading basis from all runoff leaving the proposed development site based on the post-development level of imperviousness, in accordance with provincial policy, while an infiltration facility can achieve enhanced 80% S.S. Removal by an infiltration measure (grass swale) with a range of 25m³/Ha for 35% levels of imperviousness (MOE 2003 SWM PDM Table 3.2). With TIMP equal to 10.9% for the site property, required storage volumes may be obtained by extrapolating the values provided in Table 3.2, which is about 19.0m³/Ha for a grass swale. The proposed site property has a total water storage volume of 60m³ @ maximum 0.29m depth of check dams (Total number of check dams # 3) along with 1.2m bottom width, and 0.5m depth of the swale while using MOE 2003 Table 3.2, the minimum storage volume required is 41.1m³. Therefore, on that initial basis, 80% TSS removal is preliminarily met.

The proposed post-development has all impervious areas discharging into the proposed vegetative filter strips (grassy lawn) and swales+check dams (LID) Thus, the efficiency would be:

$$\text{Initial TSS} = 1.0$$

$$\text{TSS Load Removed by Vegetative Filter Strip} = 1.0 \times 30\% \text{ Removal Rate} = 0.3 \text{ or } 30\%$$

$$\text{Remaining TSS Load Downstream of Vegetative Filter which drained to Swale (LID)} = 1.0 - 0.3 = 0.7$$

$$\text{TSS Load Removed by Grass Swale (LID)} = 0.7 \times 80\% \text{ Removal Rate} = 0.56$$

$$\text{Final TSS Load Downstream of Grass Swale which drained to Municipal Drain} = 0.7 - 0.56 = 0.14$$

$$\text{Rooftops \& Parking Lots \& Driveways TSS Removal Rate} = 1.0 - 0.14 = 0.86 \text{ or } 86\%$$

The proposed SWM Grass Swales along with vegetative filter strips (grassy lawn) between the swales and impervious gravel pavement will treat the post-development flows to the required MOE quality standard, with a TSS removal rate of approximately 86%.

In this project, proposed grass swales along with three proposed check-dams have been selected as BMPs. In other words, the grassy lawn area along with check-dams provides water quantity and quality control for the majority of the site.

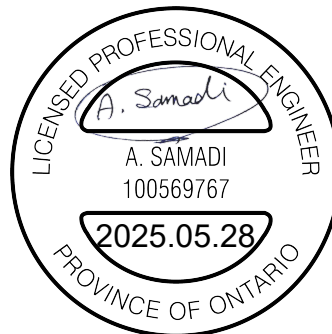
6. Reliance & Signature

This report is the intellectual property of King EPCM, and has been prepared for the sole use of Xue Ju Zheng (the 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 City of Port Colborne, Niagara Peninsula Conservation Authority (NPCA), and Regional Municipality of Niagara (Niagara Region) in regards to the Client's industrial development project at 0th Forks Road, Part of Lots 23 to 25, Con 4, East of the Welland Canal, City of Port Colborne, ON.

Respectfully,



Amir Samadi, PhD, P.Eng.
Senior Engineer – Water Resources
King EPCM



7. References

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The Ministry of Transportation (MTO), (1997). Drainage Management Manual.

The City of Port Colborne. (2015). Storm Sewer System Infrastructure Needs Study, Prepared by Associated Engineering (Ont.) Ltd. in conjunction with: GeoAdvice Engineering Inc. and M. Fortin Associates.

TRCA and CVC, (2010). Low Impact Development Stormwater Management Planning and Design Guide. Toronto Region Conservation Authority.

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

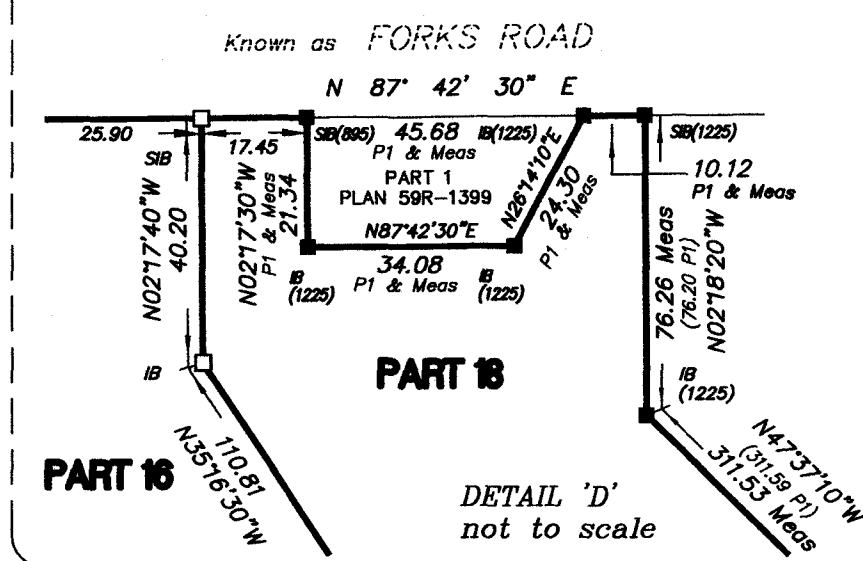
APPENDIX I – SITE SURVEY PLAN

Plan of Survey of
PART OF LOTS 23, 24 AND 25
AND PART OF THE ROAD ALLOWANCE BETWEEN LOTS 24 AND 25
CONCESSION 4

Geographic Township of Humberstone
CITY OF PORT COLBORNE
REGIONAL MUNICIPALITY OF NIAGARA

SCALE 1 : 2500
Rasch & Hyde Ltd.
Ontario Land Surveyors

C O N C E S S I O N 5



I REQUIRE THIS PLAN TO BE
DEPOSITED UNDER THE LAND
TITLES ACT.

DATE
MARCH 18, 2015

HAROLD D. HYDE
ONTARIO LAND SURVEYOR

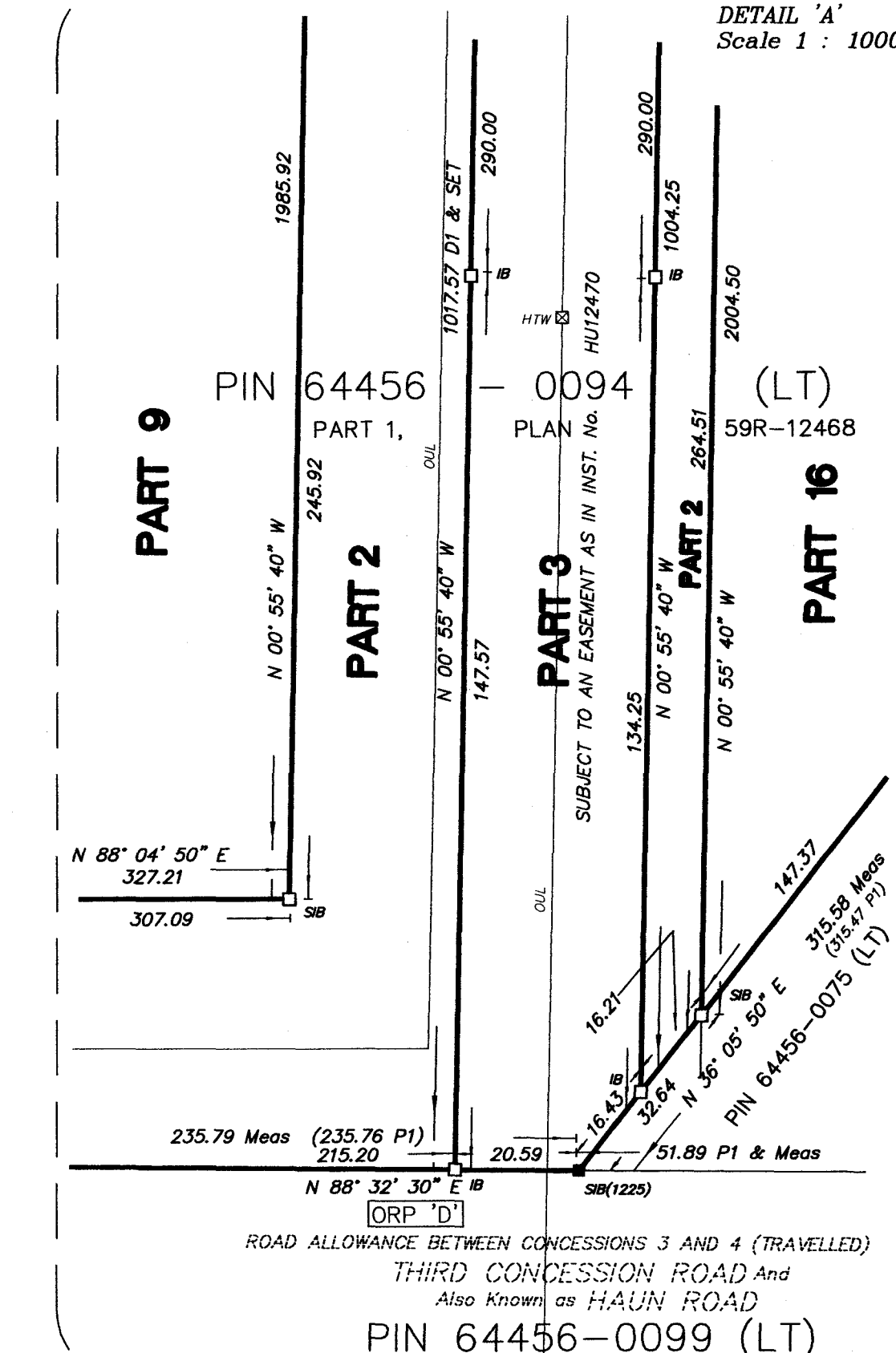
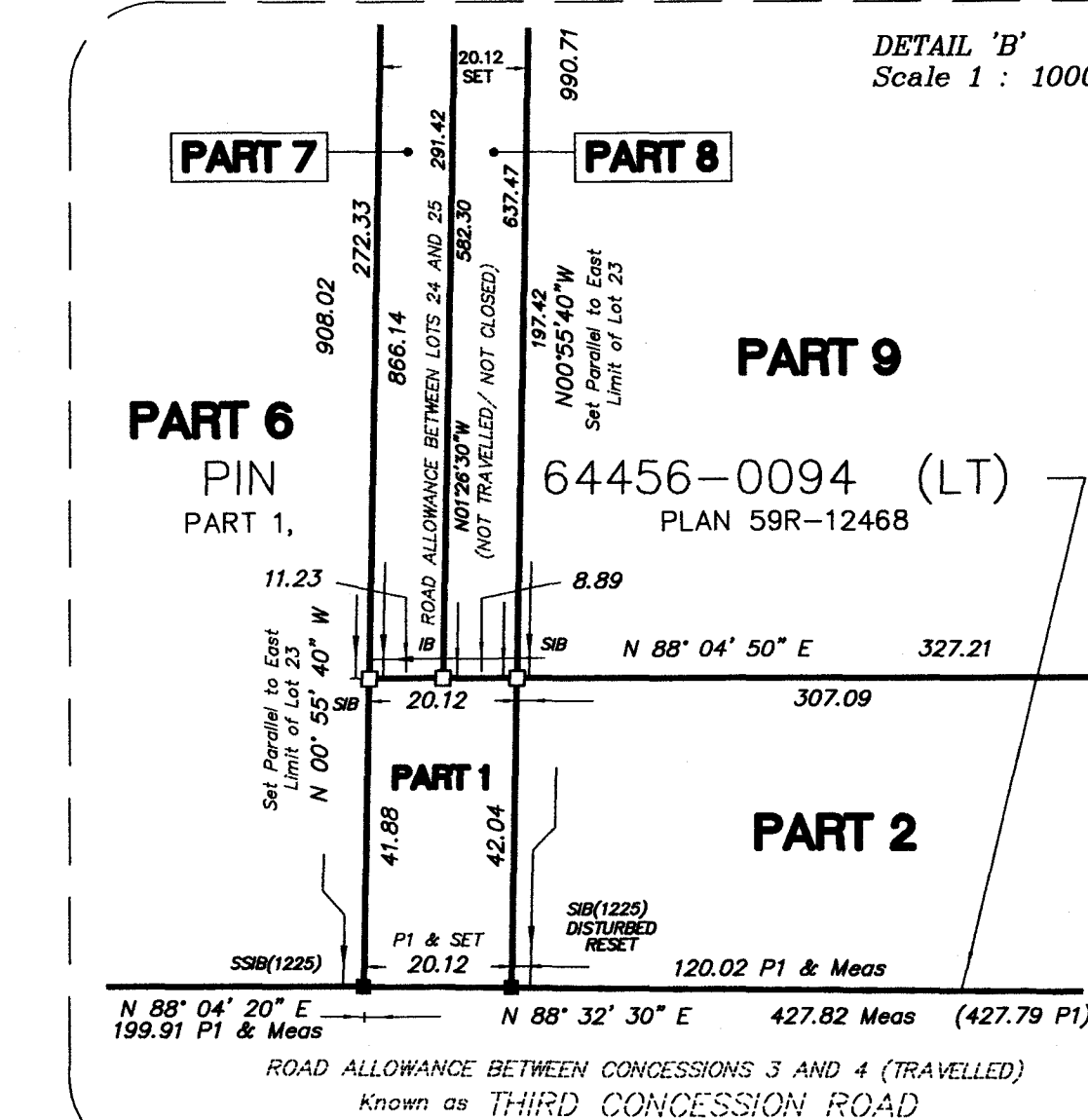
PLAN 59R-15310

RECEIVED AND DEPOSITED
March 18/15

REPRESENTATIVE FOR LAND REGISTRAR
FOR THE LAND TITLES DIVISION
OF NIAGARA SOUTH (No. 59)

SCHEDULE				
PART	LOCATION	CON	AREA (±)	PIN NO
1	ROAD ALLOWANCE BETWEEN LOTS 24 AND 25		844.2 SQ.M	
2			11.7609 Ha.	
3			3.0947 Ha.	
4	PART OF LOT 24		1410 SQ.M	
5			1948 SQ.M	
6	PART OF LOT 25		9.3134 Ha.	
7			1.0912 Ha.	
8	ROAD ALLOWANCE BETWEEN LOTS 24 AND 25		7342 SQ.M	
9			23.7780 Ha.	
10			21.3523 Ha.	
11			7831 SQ.M	
12	PART OF LOT 24		1146 SQ.M	
13			180.5 SQ.M	
14			1957 SQ.M	
15			2529 SQ.M	
16	PART OF LOTS 23 AND 24		64.3294 Ha.	
17			737.0 SQ.M	
18	PART OF LOT 23		1.9922 Ha.	

PART 3 - IS SUBJECT TO HYDRO EASEMENT AS IN INST. No. HU12470



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

- LEGEND
- DENOTES SURVEY MONUMENT PLANTED
 - DENOTES SURVEY MONUMENT FOUND
 - DENOTES STANDARD IRON BAR (25mmX25mmX120cm)
 - DENOTES SHORT STANDARD IRON BAR (25mmX25mmX60cm)
 - DENOTES IRON BAR (15mmX15mmX80cm)
 - DENOTES ROUND IRON BAR (20mm DIA X 60cm)
 - DENOTES IRON TUBE
 - DENOTES CUT STONE MONUMENT
 - DENOTES WITNESS
 - DENOTES ORIGIN UNKNOWN
 - DENOTES PROPERTY IDENTIFIER NUMBER
 - DENOTES RASCH & HYDE O.L.S.
 - DENOTES D. B. SEARLES SURVEYING LTD.
 - DENOTES DOUGLAS & LANE O.L.S.
 - DENOTES J. E. LANTHER, O.L.S.
 - DENOTES ST. LAWRENCE SEAWAY AUTHORITY
 - DENOTES CANADIAN NATIONAL RAILWAYS
 - DENOTES ARCHIBALD, GRAY AND MACKAY LIMITED
 - DENOTES PLAN 59R-12468
 - DENOTES INST. No. HU12470 (PIN 64456-0094(LT))
 - DENOTES HYDRO/UTILITY POLE
 - DENOTES OVERHEAD UTILITY/HYDRO LINE
 - DENOTES HYDRO TOWER
 - DENOTES OBSERVED REFERENCE POINTS
 - DENOTES GUY ANCHOR
 - DENOTES GUY POLE
 - DENOTES BURIED 0.30 BELOW GRADE

NOTES
AREAS NOTED IN SCHEDULE ARE ±
ALL DISTANCES ARE ROUNDED TO NEAREST 0.01 meter
COORDINATES CANNOT, IN THEMSELVES, BE USED TO RE-ESTABLISH CORNERS OR BOUNDARIES SHOWN ON THIS PLAN.
DISTANCES ARE ADJUSTED GROUND DISTANCES AND CAN BE CONVERTED TO GRID DISTANCES BY MULTIPLYING BY THE AVERAGE COMBINED SCALE FACTOR (CSF = 0.99983563)
ALL COORDINATES ARE IN METRES AND ARE UTM-ZONE 17, (NAD 83-CSR5 (Epoch 1997.0)) (CENTRAL MERIDIAN 81° WEST LONGITUDE).
COORDINATE VALUES ARE TO AN URBAN ACCURACY IN ACCORDANCE WITH SECTION 14 (2) OF O. REG 216/10.

INTEGRATION DATA		
POINT	NORTHING	EASTING
ORP 'A' (1225)	4754410.272	643843.479
ORP 'B' (1225)	4756454.651	644317.493
ORP 'C' (1321)	4756456.915	644405.813
ORP 'D' (1321)	4754426.111	644398.418

ROTATION NOTE
TO COMPARE THE GRID BEARINGS SHOWN ON THIS PLAN WITH THE BEARINGS SHOWN ON P1 APPLY A COUNTER-CLOCKWISE ROTATION OF 01° 01' 00" TO THE BEARINGS SHOWN P1. BEARINGS NOMINALLY AGREE WITH P1 WHEN THE ROTATION IS APPLIED UNLESS NOTED OTHERWISE.

BEARING NOTE
BEARINGS HEREON ARE GRID, UTM ZONE 17, (NAD 83-CSR5 (Epoch 1997.0)) DERIVED FROM OBSERVED REFERENCE POINTS (CRPS) USING THE CAN-NET VRS NETWORK AND ARE REFERRED TO THE CENTRAL MERIDIAN OF UTM ZONE 17 (81° WEST LONGITUDE).

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 LAND TITLES ACT AND THE REGULATIONS MADE UNDER THEM.
2. THIS SURVEY WAS COMPLETED ON THE 15th DAY OF DECEMBER 2014.
DATE
MARCH 18, 2015
HAROLD D. HYDE
ONTARIO LAND SURVEYOR

RASCH & HYDE LTD.
Ontario Land Surveyors
P.O. Box 6, 1333 Highway #5 East, Unit 8
DUNVILLE, ONT. M1A 2Y1
905-774-7188
(FAX 905-774-4000)
R. DESMOND RASCH O.L.S.
SCALE 1 : 2500
SURVEY : 14F004
DRWN BY : A.W. / G.P.

PLAN 59R-15310

RECEIVED AND DEPOSITED

REPRESENTATIVE FOR LAND REGISTRAR

FOR THE LAND TITLES DIVISION

OF NIAGARA SOUTH (No. 59)

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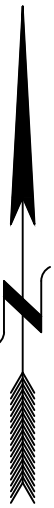
PLAN 59R-15310

RECEIVED AND DEPOSITED

REPRESENTATIVE FOR LAND REGISTRAR

FOR THE LAND TITLES DIVISION</

APPENDIX II – SITE GRADING & SWM PLAN



LEGEND:



PROPERTY LINE

Municipality Notes

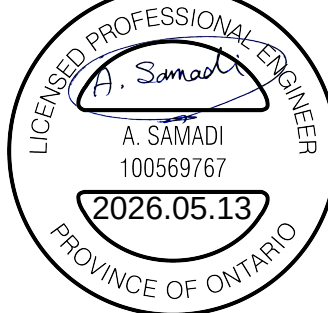
DRAWN

S.L/S.P

DATE _____

MAY 04, 2026

STAMP



KEY MAP

N.T.S.

AD

SUBJECT SITE

THIRD CONCESSION

KING

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

CLIENT

XUE JU (JACK) ZHENG
2789475 ONTARIO INC.

PROJECT NAME

0TH FORKS ROAD

PROJECT LOCATION

**0TH FORKS ROAD
COLBORNE**

PRINT TITLE

SITE KEY PLAN

FILE No.

EGR - 1.1

No.	ISSUED FOR:	DATE	DRAW BY	CHECKED BY
V1	ISSURED FOR PERMITS	APR 29, 2024	SL	TW
V4	ISSURED FOR PERMITS	JUN 26 2025	SP	AA
V5	ISSURED FOR PERMITS	JUL 22 2025	SP	TW
V7	ISSURED FOR PERMITS	JAN 07 2026	SP	TW
V10	ISSURED FOR PERMITS	MAY 04 2026	SP	TW

SCALE 1:5000

$$1\text{ cm} = 50\text{ m}$$

0m

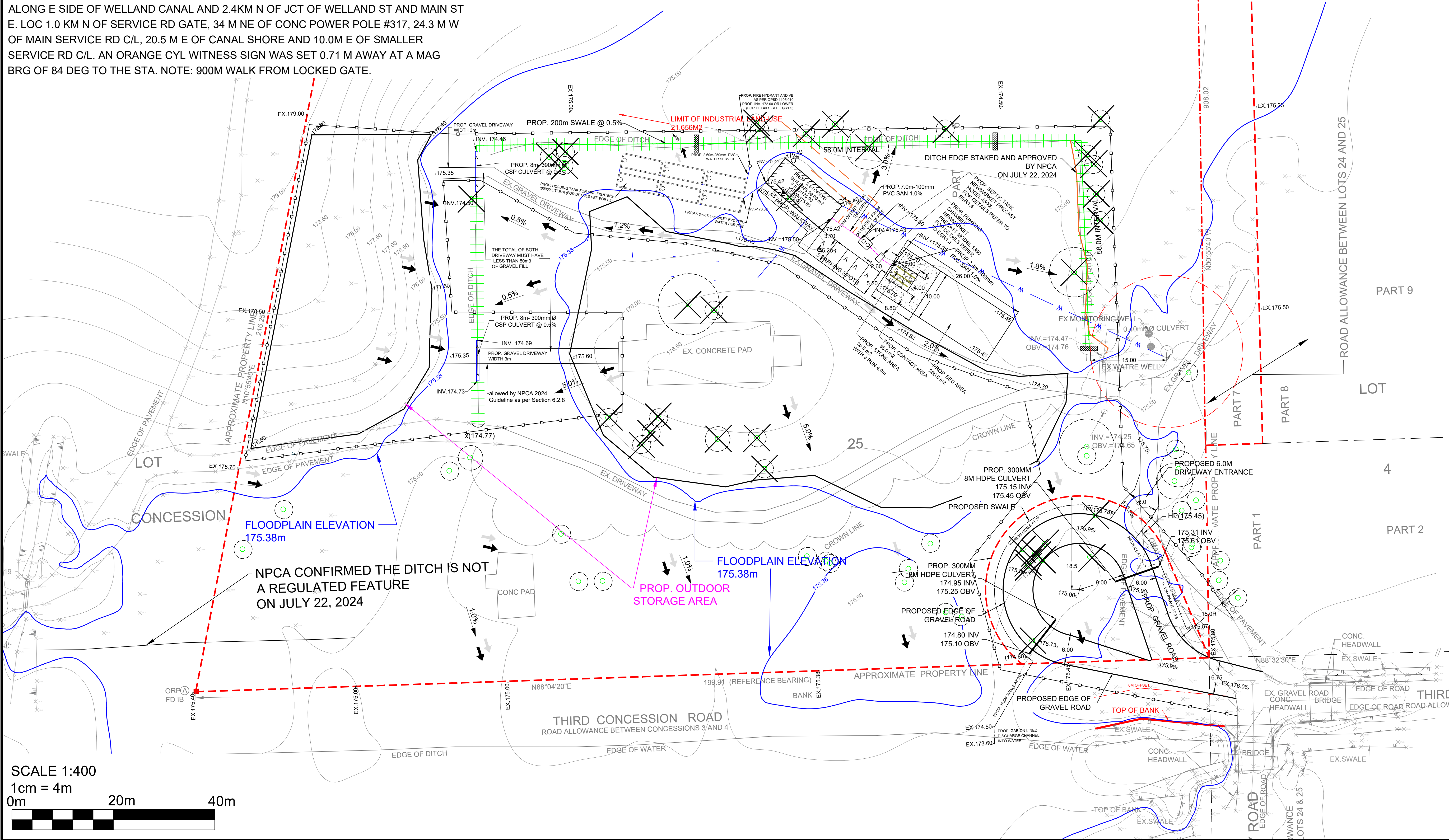
250m

500m



GENERAL NOTES:

- TOPO SURVEY WAS CONDUCTED ON MARCH 7, 2024 AND OCTOBER 1, 2025 BY KING EPCM. NATURAL HERITAGE FEATURE STAKING SURVEY WAS CONDUCTED ON JULY 22, 2024
- ALL MEASUREMENTS STATED IN METRES.
- PROPERTY BOUNDARY WAS REFERENCED BY RASCH + HYDE LTD., DATED MARCH 18, 2015.
- THIS IS NOT A PLAN OF SURVEY. DO NOT USE FOR BOUNDARY.
- ELEVATIONS ARE GEODETIC, OBTAINED FROM GNSS AND USING CAN-NET DERIVED FROM RTK SOLUTION AND GEOID MODEL HTv.2.0 SOLUTION, REFERRED TO CANADIAN GEODETIC VERTICAL DATUM OF 1928 (CGVD28:78) AND BENCHMARK No. 0011980U014, HAVING AN ELEVATION OF 173.81M . THE BENCHMARK IS IN MANHOLE ALONG E SIDE OF WELLAND CANAL AND 2.4KM N OF JCT OF WELLAND ST AND MAIN ST E. LOC 1.0 KM N OF SERVICE RD GATE, 34 M NE OF CONC POWER POLE #317, 24.3 M W OF MAIN SERVICE RD C/L, 20.5 M E OF CANAL SHORE AND 10.0M E OF SMALLER SERVICE RD C/L. AN ORANGE CYL WITNESS SIGN WAS SET 0.71 M AWAY AT A MAG BRG OF 84 DEG TO THE STA. NOTE: 900M WALK FROM LOCKED GATE.



LEGEND:

x 175.54

175.00

CONC.

IB

ORP

PROPERTY LINE

EXISTING ELEVATION

CONTOUR LINE

WATERWELL

CONCRETE

IRON BAR

OBSERVED REFERENCE POINTS

CHECK DAM

GRASS SWALE

TREE TO BE RETAINED

TREE TO BE REMOVED

Municipality Notes

DRAWN

TW

DATE

MAY 04, 2026

KEY MAP

N.T.S.

SUBJECT SITE

THIRD CONCESSION ROAD

RAMSEY ROAD

Quad Niagara ATV club

STAMP

LICENCED PROFESSIONAL ENGINEER

A. Samadi

A. SAMADI

100569767

2026.05.13

PROVINCE OF ONTARIO

CLIENT

XUE JU (JACK) ZHENG

2789475 ONTARIO INC.

PROJECT NAME

0TH FORKS ROAD

PROJECT LOCATION

0TH FORKS ROAD

COLBORNE

PRINT TITLE

SITE PLAN

FILE No.

EGR - 1.2

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V1	ISSURED FOR PERMITS	APR 29, 2024	SL	TW
V4	ISSURED FOR PERMITS	JUN 26 2025	SP	AA
V5	ISSURED FOR PERMITS	JUL 22 2025	SP	TW
V8B	ISSURED FOR PERMITS	MAR 18, 2026	SP	TW
V10	ISSURED FOR PERMITS	MAY 04, 2026	SP	TW

SCALE 1:400

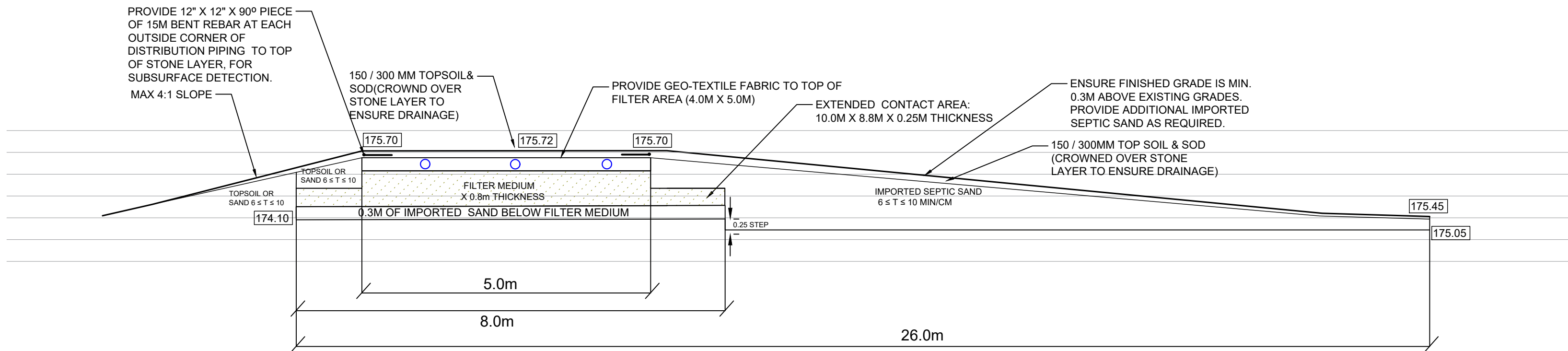
1cm = 4m

0m 20m 40m

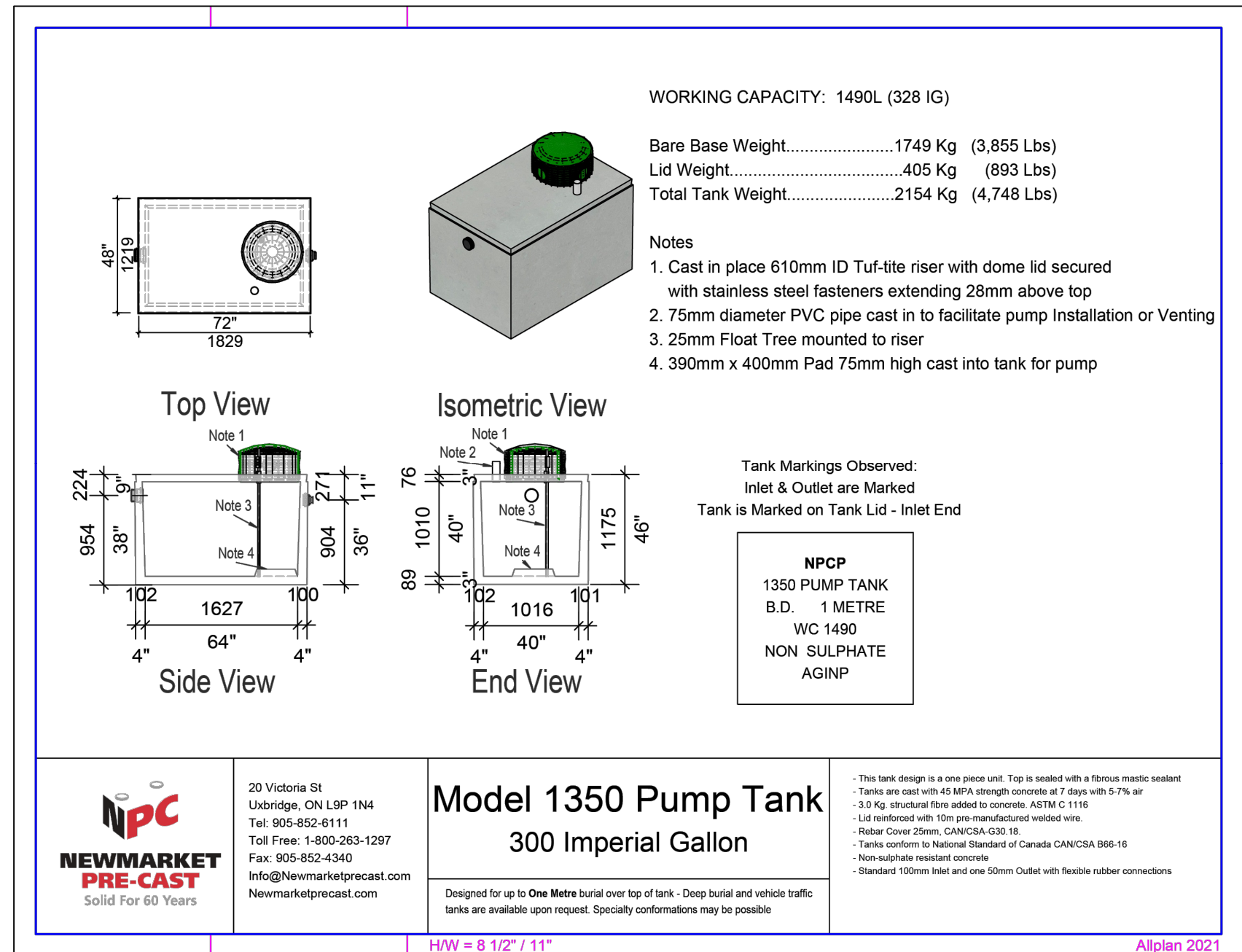
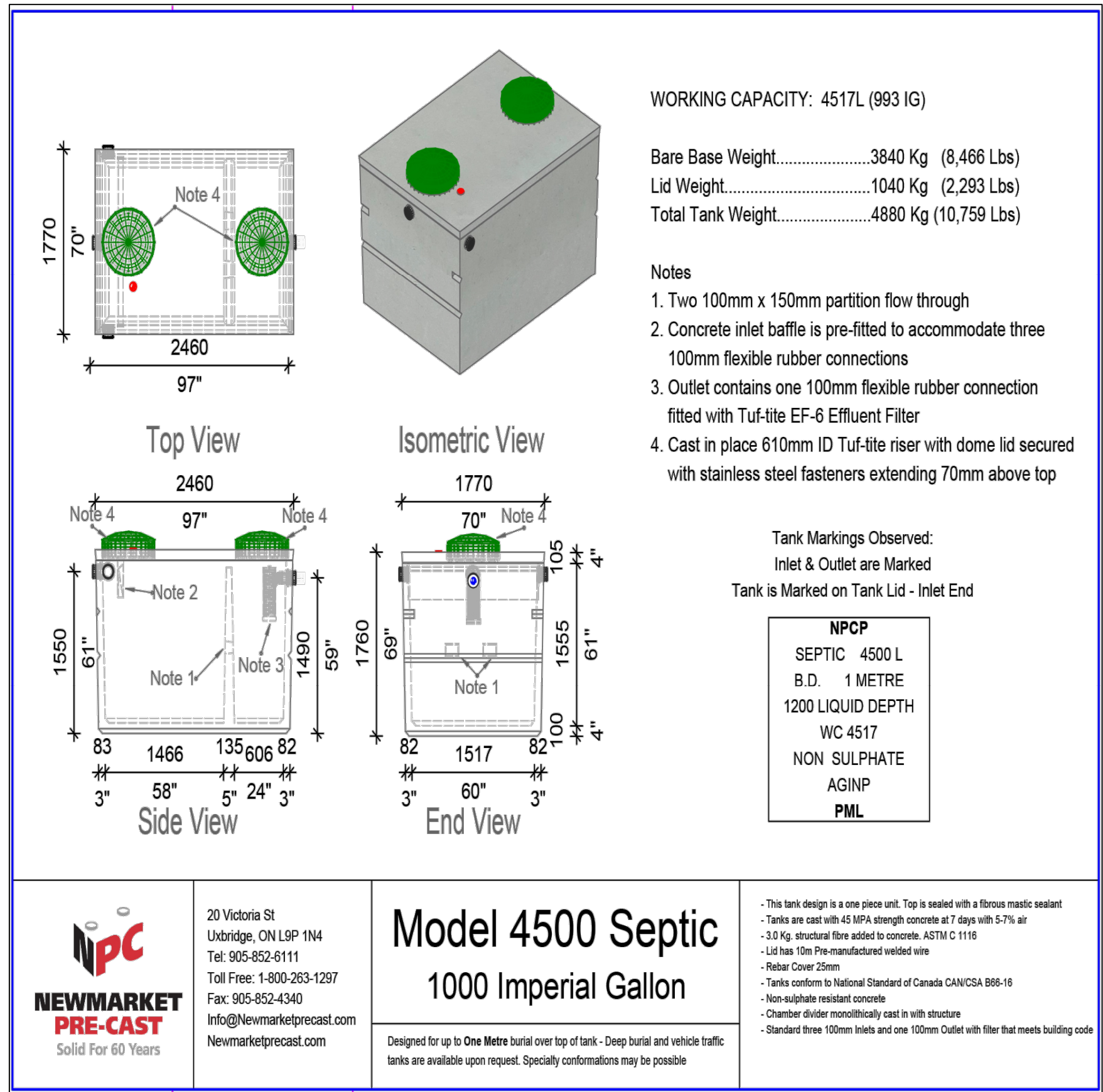
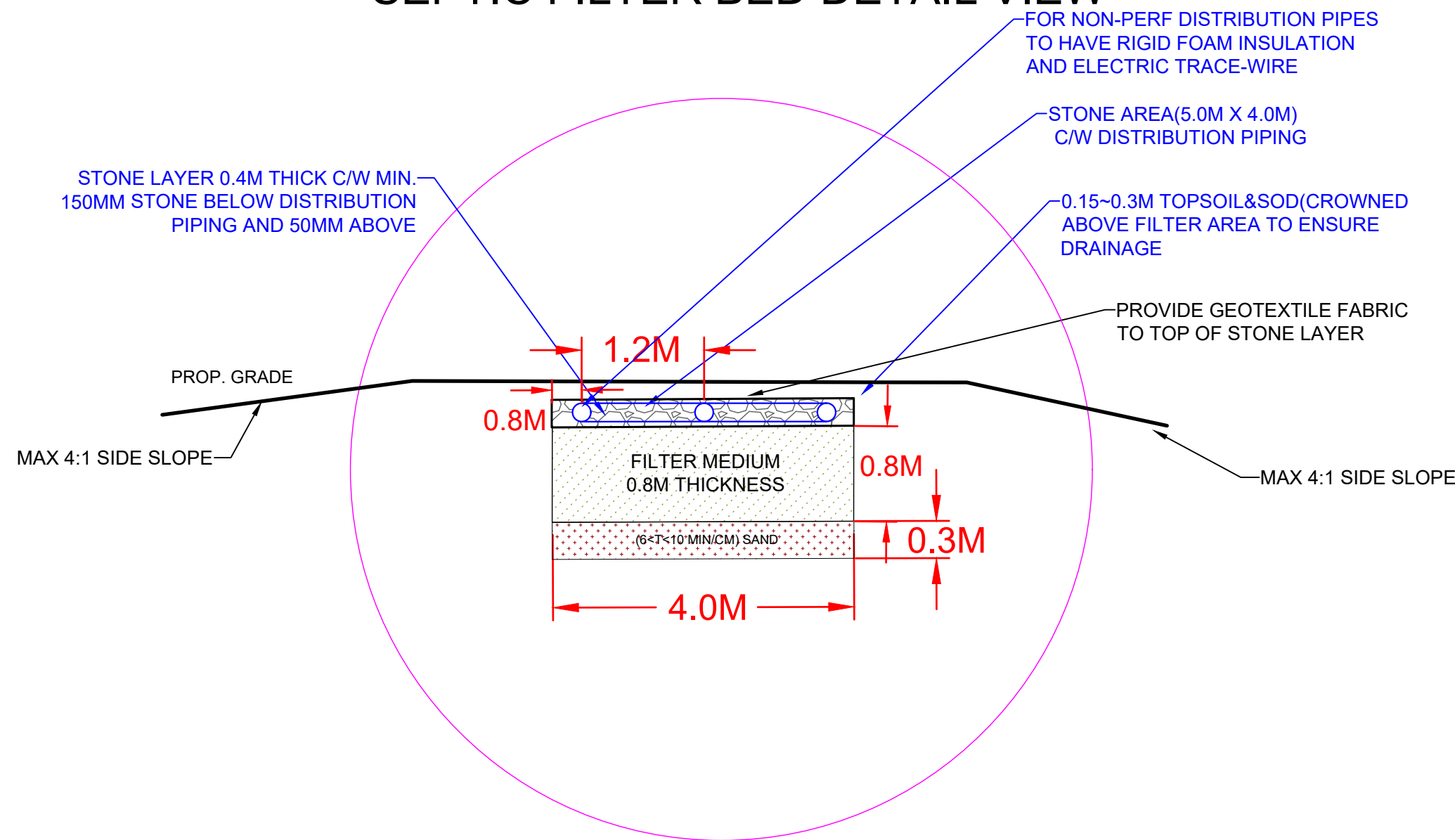
VIEWING EAST

A- NORTH

A'- SOUTH



DETAIL A (N.T.S)
SEPTIC FILTER BED DETAIL VIEW



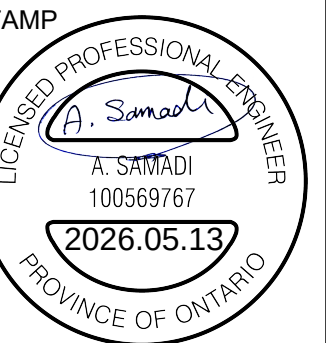
Municipality Notes

DRAWN

S.L./S.P.

DATE _____

MAY 04, 2026



KEY MAP

N.T.S.

Quad Niagara ATV club

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

CLIENT

XUE JU (JACK) ZHENG
2789475 ONTARIO INC.

PROJECT NAME

0TH FORKS ROAD

PROJECT LOCATION

**0TH FORKS ROAD
COLBORNE**

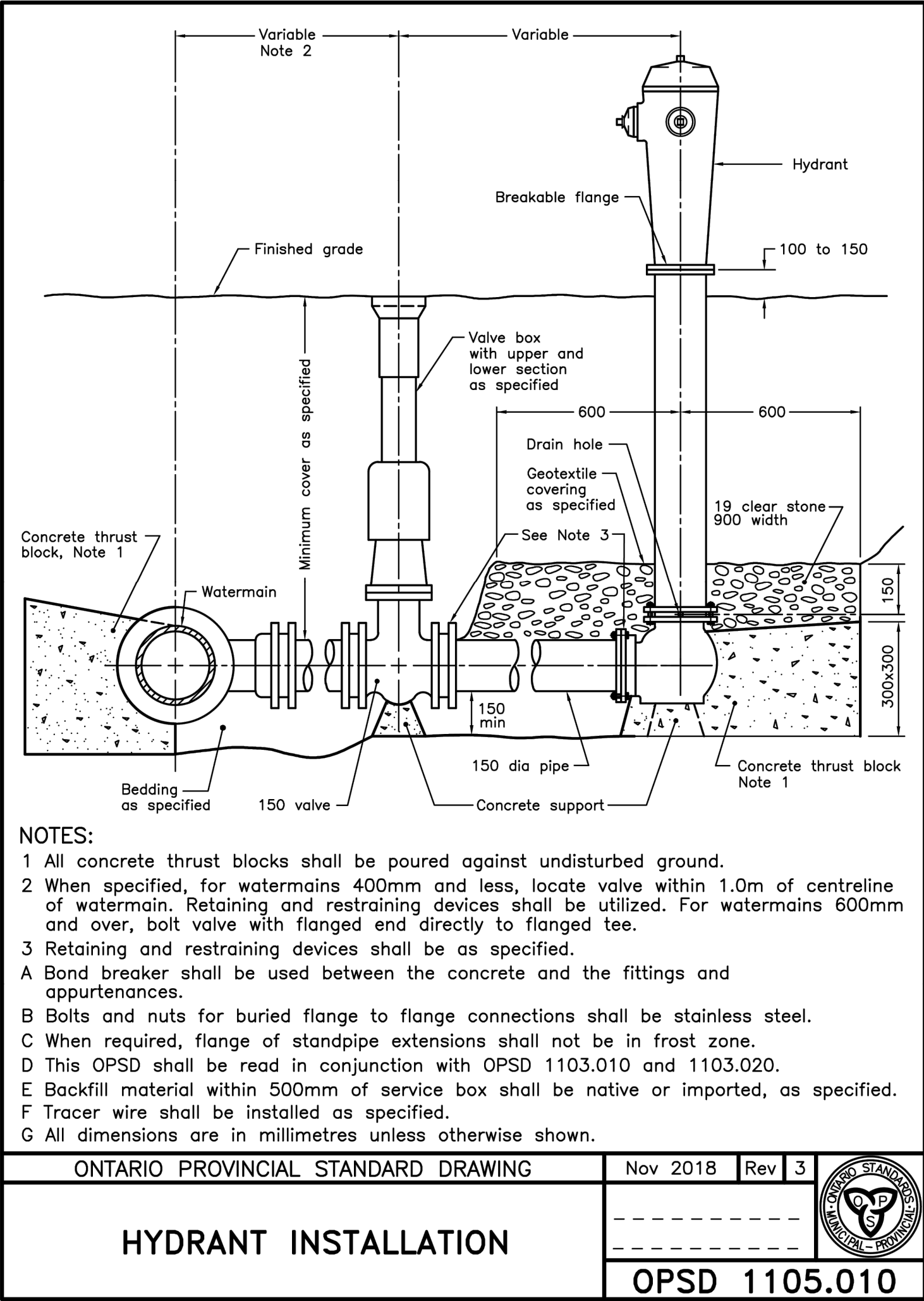
PRINT TITLE

SEPTIC DETAIL

FILE No.

EGR - 1.4

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V1	ISSURED FOR PERMITS	APR 29, 2024	SL	TW
V4	ISSURED FOR PERMITS	JUN 26 2025	SP	AA
V5	ISSURED FOR PERMITS	JUL 22 2025	SP	TW
V7	ISSURED FOR PERMITS	JAN 07 2026	SP	TW
V10	ISSURED FOR PERMITS	MAY 04 2026	SP	TW



60,000 LITRE CONCRETE WATER HOLDING TANK MODEL H60S

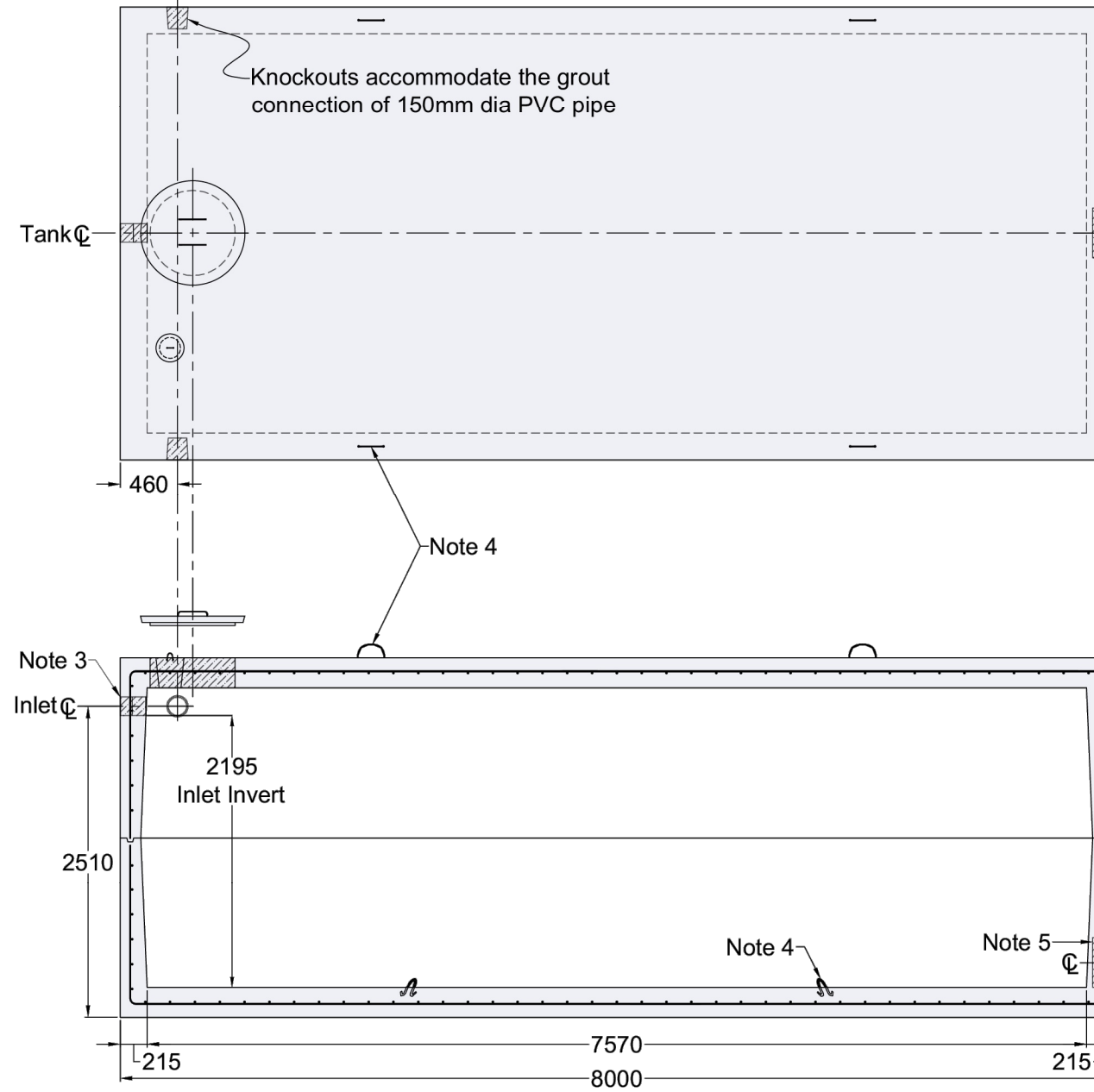
CONSTRUCTION DETAILS:

Concrete: 35 MPa at 28 Days, 5 to 8% Air Entrainment.

Reinforcing: Designed for a maximum 1 metre burial over the top slab in firm soil.
Optional reinforcing for CHBDC vehicular loading available upon request.

WEIGHT:
Top Section - 31,500 kg
Bottom Section - 31,500 kg

CAPACITY:
Per Vertical Metre - 24,917 Litres
To Underside of Roof Slab - 60,299 Litres

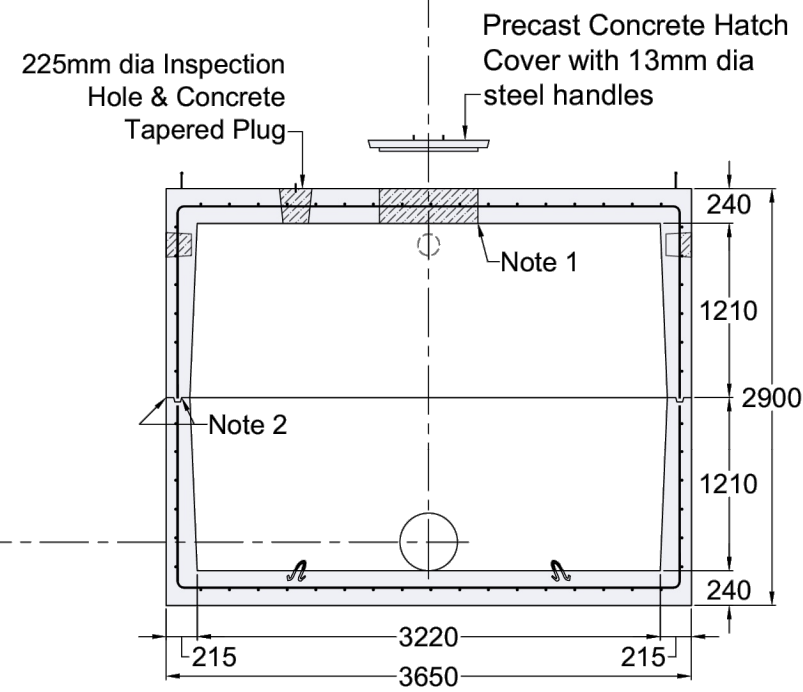


NOTES:

1. Large 685 mm diameter roof access openings facilitate tank maintenance. Unless otherwise specified when ordered this tank will be shipped with 840 mm diameter concrete roof access cover only. Please see [Access Riser](#) section for available riser & hatch options.
2. Close tolerance of tongue & groove joint & fibrous mastic sealant ensures a solid structural and watertight seal. Primer & Mastic Band are supplied with each tank for application to the external surface of the tank over the joint between sections. All sealant is to be applied by the installing contractor.
3. Flexible watertight inlet pipe connector to accommodate 150 mm diameter PVC pipe. Size and location of connections are customizable.
4. Top & bottom section lifting points four places.
5. Knockout suitable to accommodate a flexible, watertight pipe connectoin for 250, 275 or 300 mm diameter PVC pipe.

Some Available Options:

- Aluminum ladder rungs to the floor. Consult with the factory as to how this will effect the size and location of the access opening.
- Mechanical connection of the tank sections to enhance water tightness and resistance to frost heave.



Dimensions in mm
N.T.S.

*Product designed for a Maximum 1 Metre burial over the top slab in firm soil away from any area of vehicular traffic.

For recommended installation procedures refer to Wilkinson [Installation Guidelines](#).

WARNING! IMPROPER INSTALLATION ESPECIALLY IN UNSTABLE SOIL CAN RESULT IN THE STRUCTURAL FAILURE OF THIS PRODUCT

November 15, 2021

Municipality Notes

DRAWN

S.L./S.P.

DATE

MAY 04, 2026

STAMP



KEY MAP

N.T.S.



KING E P C M

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

CLIENT

XUE JU (JACK) ZHENG
2789475 ONTARIO INC.

PROJECT NAME

0TH FORKS ROAD

PROJECT LOCATION

0TH FORKS ROAD
COLBORNE

PRINT TITLE

FIRE HYDRANT DETAIL

FILE No.

EGR - 1.5

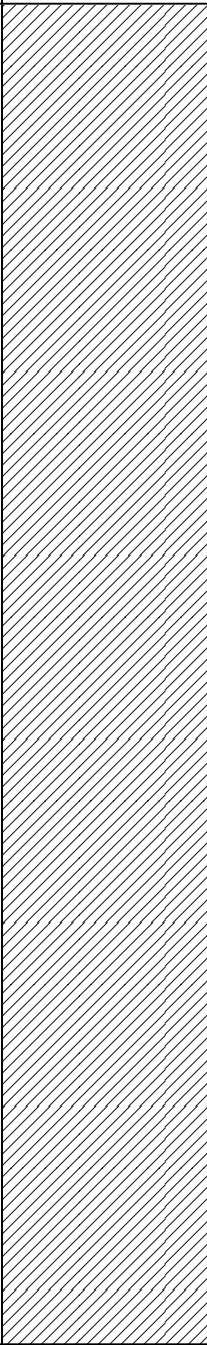
No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V1	ISSURED FOR PERMITS	APR 29, 2024	SL	TW
V4	ISSURED FOR PERMITS	JUN 26 2025	SP	AA
V5	ISSURED FOR PERMITS	JUL 22 2025	SP	TW
V7	ISSURED FOR PERMITS	JAN 07 2026	SP	TW
V10	ISSURED FOR PERMITS	MAY 04 2026	SP	TW

APPENDIX III – BOREHOLE DRILL LOG

PROJECT NUMBER	DRILLING COMPANY King EPCM	COORDINATES
PROJECT NAME 0th Forks Rd	DRILLER Leng	COORD SYS
CLIENT	DRILL RIG	SURFACE ELEVATION
ADDRESS 0th Forks Rd, Port Colborne	DRILLING METHOD Hand Auger	WELL TOC None
DRILLING DATE 03/05/2024	TOTAL DEPTH 0.5 m	LOGGED BY Leng
LICENCE NO. C-7691	DIAMETER 1 in	CHECKED BY Tony Wang, P Eng, Principal Engineer

COMPLETION	CASING	SCREEN
-------------------	---------------	---------------

COMMENTS

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Additional Observations	Elevation (m)
0.02		USCS:CL	Brown stiff clay mixed with gravel		-0.02
0.04					-0.04
0.06					-0.06
0.08					-0.08
0.1					-0.1
0.12					-0.12
0.14					-0.14
0.16					-0.16
0.18					-0.18
0.2					-0.2
0.22					-0.22
0.24					-0.24
0.26					-0.26
0.28					-0.28
0.3					-0.3
0.32					-0.32
0.34					-0.34
0.36					-0.36
0.38					-0.38
0.4					-0.4
0.42	-0.42				
0.44	-0.44				
0.46	-0.46				
0.48	-0.48				
0.5			Termination Depth at: 0.5 m	No groundwater was found within 0.5 meters End of the hole - auger refusal	-0.5
0.52					-0.52

APPENDIX IV – 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 achieved through this method can be less than or equal to half of K_s (Saturated hydraulic conductivity) due to the partial blocking of soil pores by air bubbles and it is preferred over K_s in 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 done by the ETC Soils Pask Permeameter, which is an extended single-head analysis method, and the calculations procedure used here is 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 the 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):

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

In which:

K_{fs} = the calculated permeability from the field test

Table 1. Parameters used

Parameter	Description	Near BH101
		SW
Soil Texture Factor (α^*) in cm^{-1}	Compacted, structureless, clayey materials such as landfill caps and liners, lacustrine, or marine sediments.	0.01
R in cm/min	Quasi-steady state (constant) rate of fall of water in permeameter reservoir (Measured in the site)	0.1
μ_k/μ_a	Temperature Correction Factor ($t= 20^\circ\text{C}$)	0.644
C	Shape factor	1.27
X in cm^2	The cross-sectional area of the permeameter reservoir	12.80
H in cm	Height of air inlet hole from the 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 Slow Soils, the K_{fs} was calculated as:

$$K_{fs1} = 2.5E-8 \text{ m/sec} = 2.5E-6 \text{ cm/sec}$$

Then the temperature-corrected permeability would be calculated using equation 2 as follows:

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

In which:

K_a = corrected permeability adjusted for design temperature conditions

Then using the temperature correction factor (for $t=20^\circ\text{C}$) from the manual:

$$K_{a1} = 1.61E-8 \text{ m/sec} = 1.61E-6 \text{ cm/sec}$$

The field permeability data sheet is in the following.

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 the 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 Eq. 3.

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

Where:

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

And a is test hole radius, $\bar{H}$ is the average water level (ponding depth) in the test hole over the time interval, Δt , ψ_a 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, $K(\psi)$ 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 $m = 1.05E-06$ (Very Strong capillarity category):

$$PT_1 = m K_{a1}^{-1} = \frac{(1.05E-6)}{(1.61E-8)} = 65.2 \text{ min/cm} \quad (\text{Infiltration Rate} = 9.2 \text{ mm/hour})$$

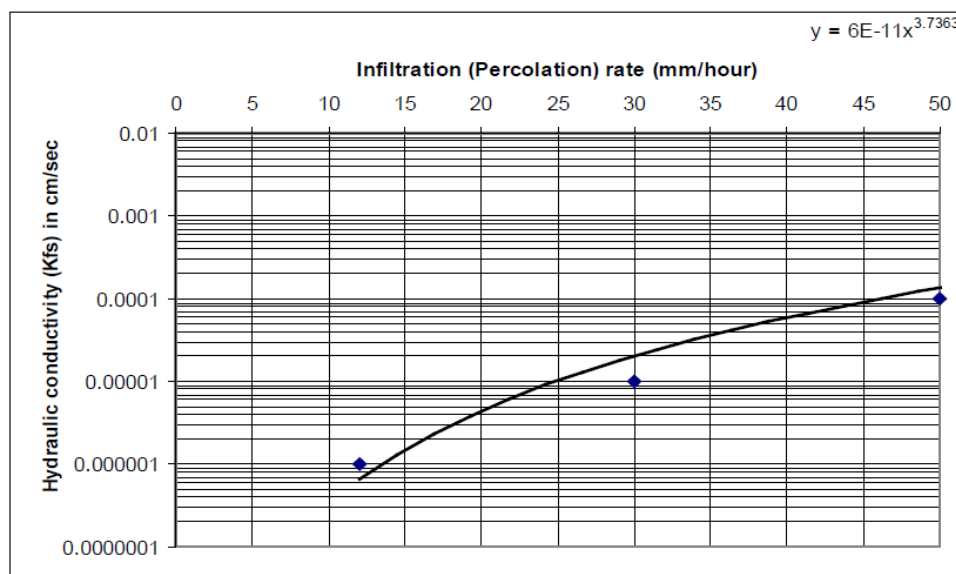
3. Percolation time/infiltration rate for design (OMMAH, 1997)

Despite the newer academic papers published by Reynolds et al. (2015), TRCA and other Conservation Authorities often still review the design of infiltration basins based on historical 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:

$$PT_1 = 39.2 \text{ min / cm (Infiltration Rate} = 15.3 \text{ mm/hour)}$$

When comparing the OMMAH result with that obtained by Reynolds et al. (2015) formula ($PT = 65.2 \text{ min/cm}$), the two methods of conversion are completely different results, especially for slow soils and it seems that the values of the first method have underestimated the infiltration rate and Perc Time. As per the TRCA Stormwater Management Criteria guideline, the engineer's opinion is to trust the value obtained from the second method (OMMAH, 1997) which was proposed by TRCA to convert measured hydraulic conductivity to percolation time and/or infiltration rate. Therefore, the engineer's opinion is to trust the values obtained from the second method (OMMAH, 1997), with an unfactored infiltration rate = 15.3 mm/hour which confirms the upper soil layer (Stiff clay mixed with gravel) has a low permeability within this property.

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 upper soil formation is stiff clay soils mixed with gravel, with an unfactored infiltration rate = 15.3 mm/hour, down to the depth. Since 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 soils (compacted) to a depth of more than 1 meter, which has a very low permeability rate.

Based on borehole data, the soil layer remains consistent with stiff clay type, including the soil layers 1.5 meters below the proposed bottom of the BMP. This means that based on below 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 15.3 mm/hour, the mean infiltration rate measured in the slowest underlying soil horizon is 6.1 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: 0th Forks Rd, Port Colborne

PID #: _____

TEST PIT#: _____

TECHNICIAN: Leng

DATE: 3/5/2024

WEATHER/TEMPERATURE: Sunny/20c

FIELD PERMEABILITY TEST #:

D - reservoir diameter (cm)	Slow	Soil Texture	_____
d - well hole diameter (cm)	_____	Soil Structure	_____
H - height of water in well (cm)	_____	α^* (cm-1)	_____
Depth below ground surface(cm)	_____	C - Factor	_____

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		32		
5	5	31.6	0.4	0.08
10	5	31.2	0.4	0.08
15	5	30.1	1.1	0.22
20	5	29	1.1	0.22
25	5	28.5	0.5	0.1
30	5	28.2	0.3	0.06
35	5	27.7	0.5	0.1
40	5	27.2	0.5	0.1
45	5	26.7	0.5	0.1
50	5	26.7	0	0
55	5	26.7	0	0
60	5	26.7	0	0

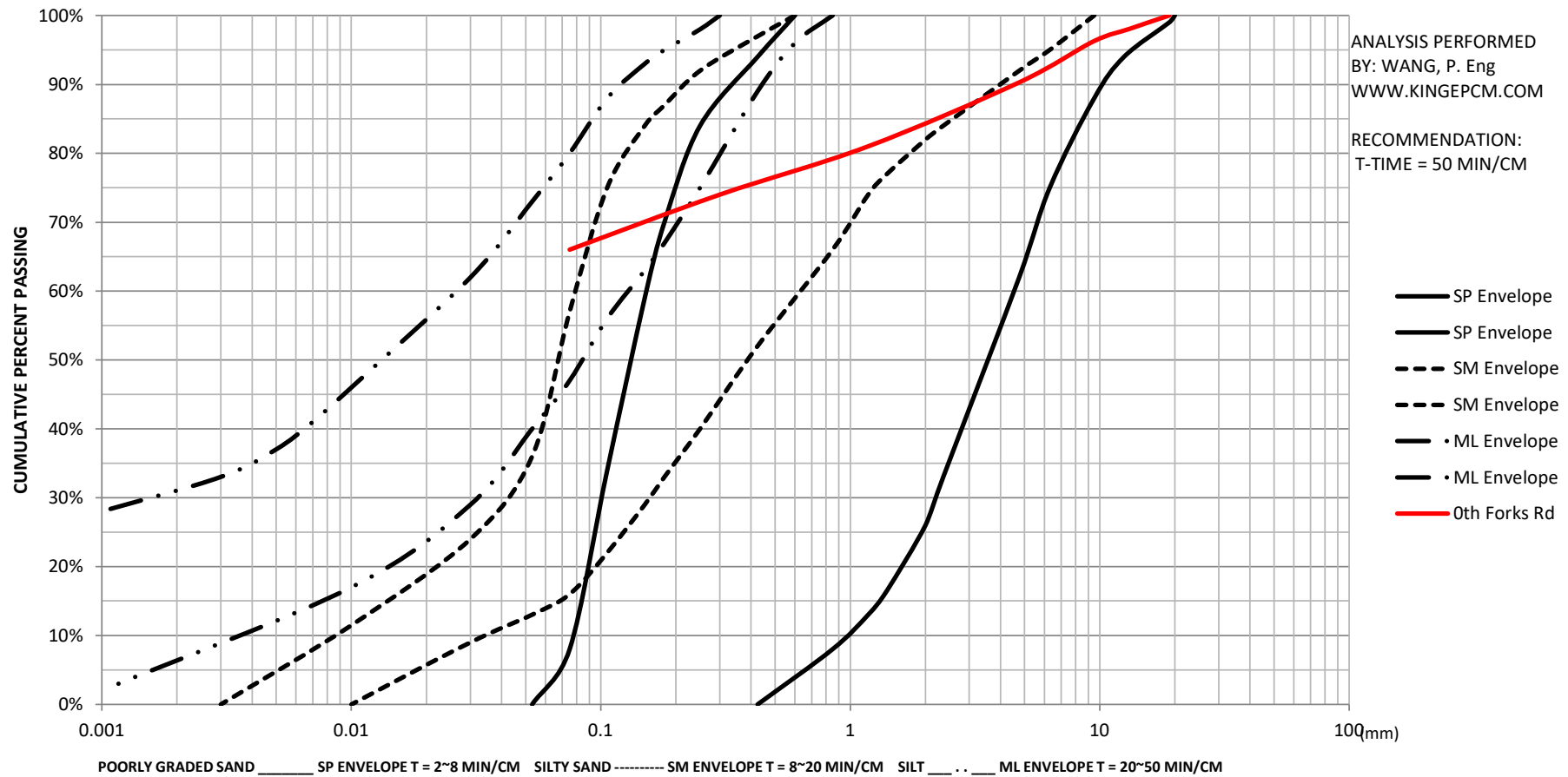
Quasi Steady-State Rate of Fall(R) = **0.1** cm/min

APPENDIX V –SIEVE TEST RESULTS

GRAIN SIZE DISTRIBUTION-UNIFIED SOIL CLASSIFICATION SYSTEM

0th Forks Road, Port Colborne

MAR 07 2024



APPENDIX VI –CLIMATE DATA TABLE

Climate Data

Welland

Climate Station ID: 6139445

Year	Month	Ave. T (°C)	Ave P (mm)
1977-2006	1	-4.7	81.7
1977-2006	2	-4.0	57.6
1977-2006	3	0.7	70.9
1977-2006	4	7.3	77.6
1977-2006	5	13.5	81.6
1977-2006	6	18.7	80.4
1977-2006	7	21.6	82.8
1977-2006	8	20.8	84.4
1977-2006	9	16.6	104.1
1977-2006	10	10.1	89.7
1977-2006	11	4.6	96.3
1977-2006	12	-1.2	93.8

Climate Data

Welland

Climate Station ID: 6139445

Month	PET	P	P-PET	Soil Moisture	AET	PET-AET	Snow Storage	Surplus
January	9.6	81.7	45.3	198.1	9.6	0.1	40.8	42.3
February	11.3	57.6	44.6	199.6	11.3	0.0	42.4	43.1
March	21.5	70.9	79.6	200.0	21.5	0.0	12.3	79.2
April	39.7	77.6	50.2	199.8	39.7	0.0	0.0	50.4
May	72.4	81.6	9.1	189.6	72.4	0.0	0.0	19.3
June	105.3	80.4	-24.9	160.7	103.1	2.2	0.0	6.2
July	125.2	82.8	-42.4	125.3	115.7	9.5	0.0	2.5
August	101.6	84.4	-17.3	115.7	91.3	10.3	0.0	2.6
September	60.5	104.1	43.6	144.9	58.2	2.3	0.0	16.8
October	32.2	89.7	57.4	175.6	32.2	0.0	0.0	26.8
November	17.3	96.3	78.9	191.9	17.3	0.0	0.1	62.5
December	11.0	93.8	69.0	196.7	11.0	0.0	14.0	64.2
Annual Rate (mm/yr)	607.7	1000.9	393.2	2097.9	583.3	24.3	109.6	415.9

APPENDIX VII –SUBWATERSHEDS WITHIN THE SITE

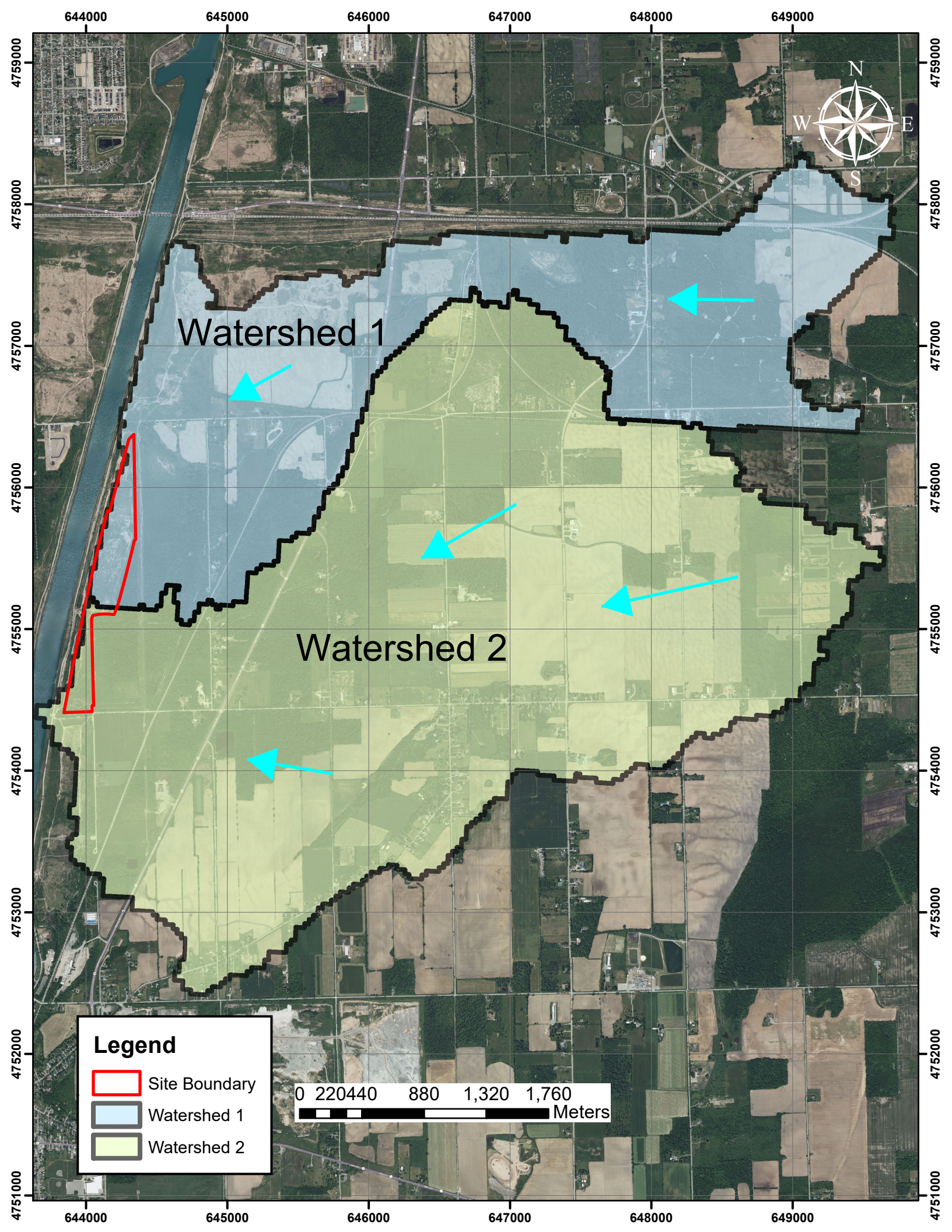


Figure 2-3. Niagara Region Subwatershed Areas

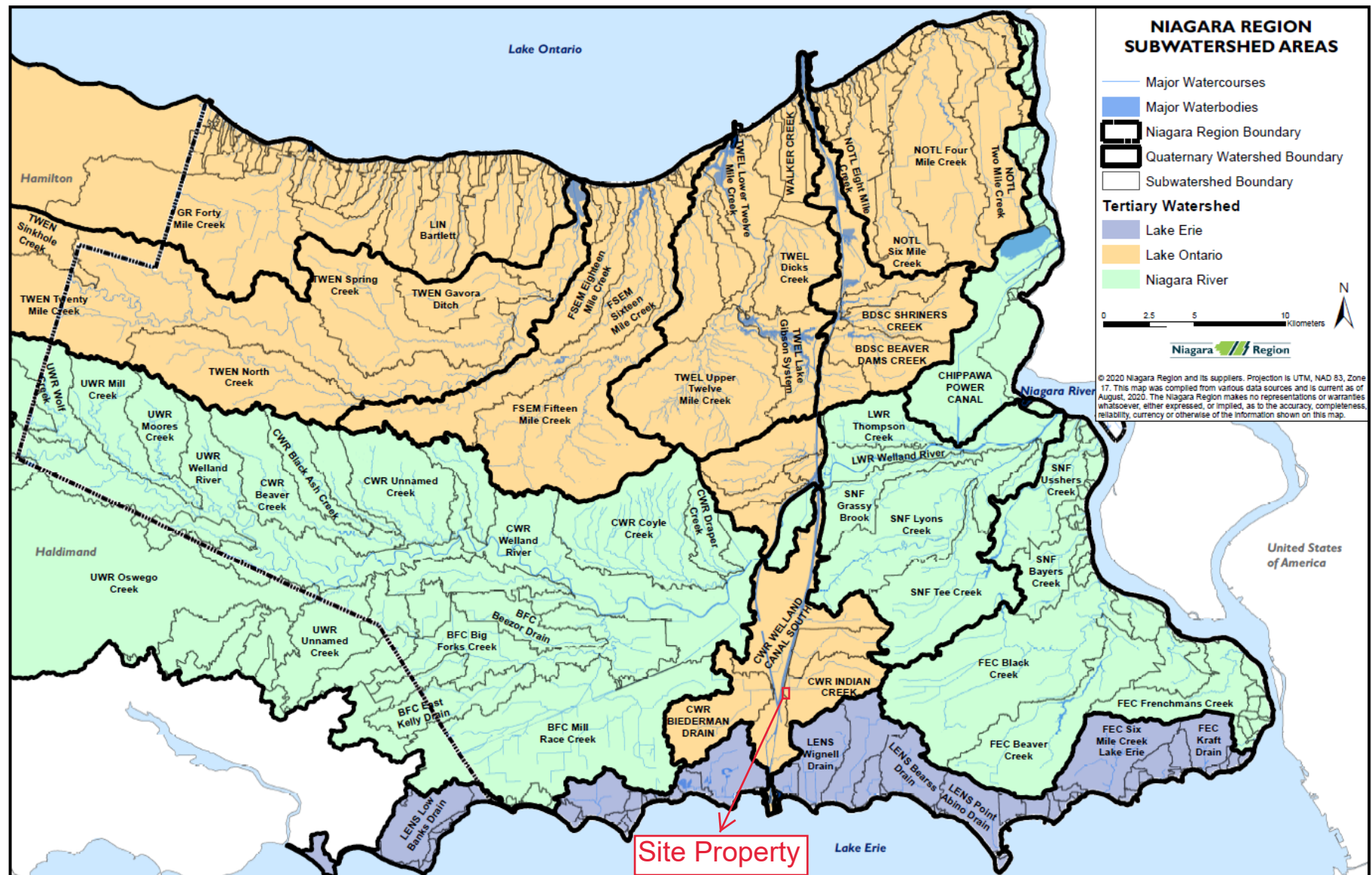


Figure 2-1. Niagara Region – Tertiary Watershed Areas

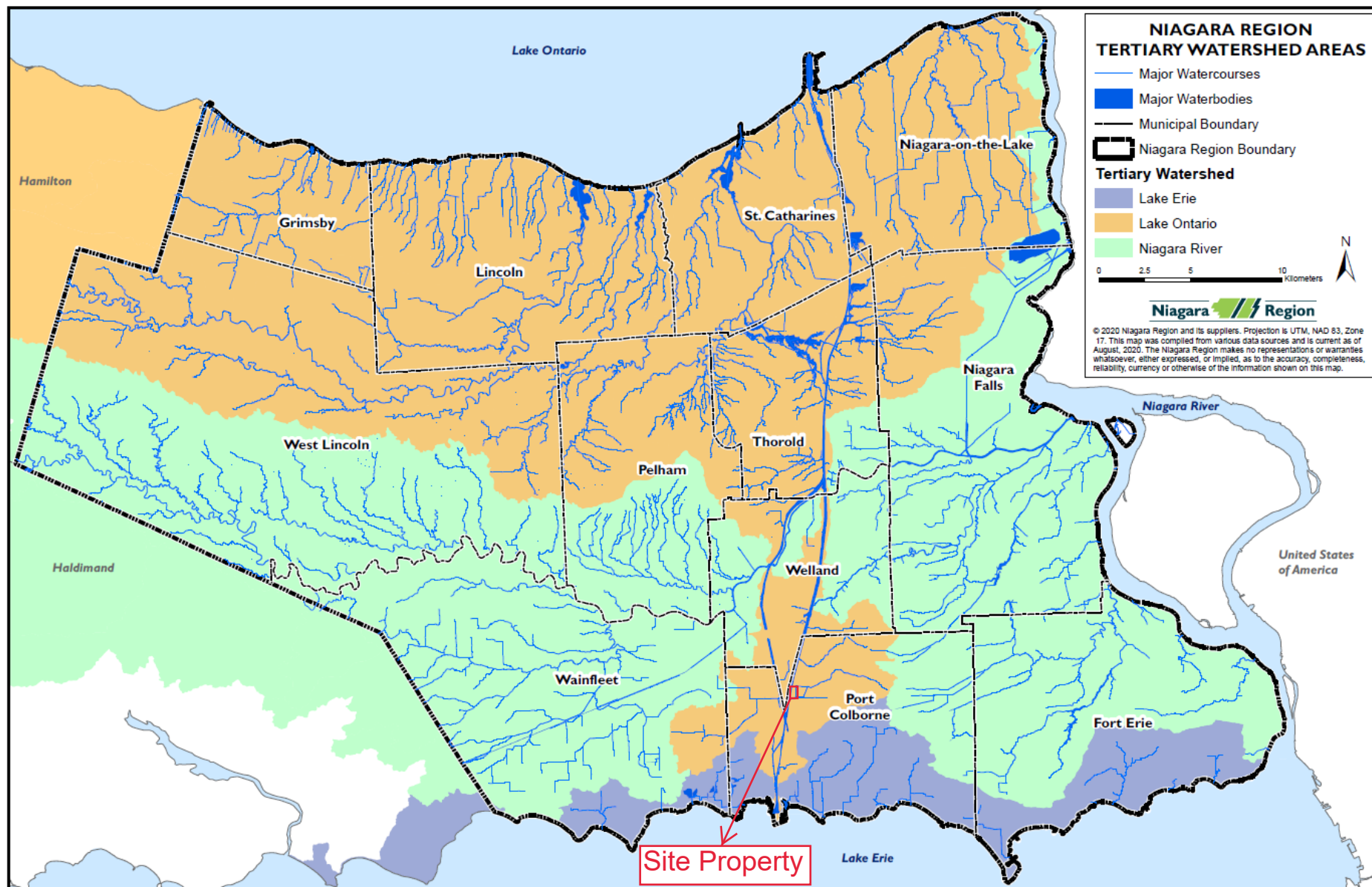
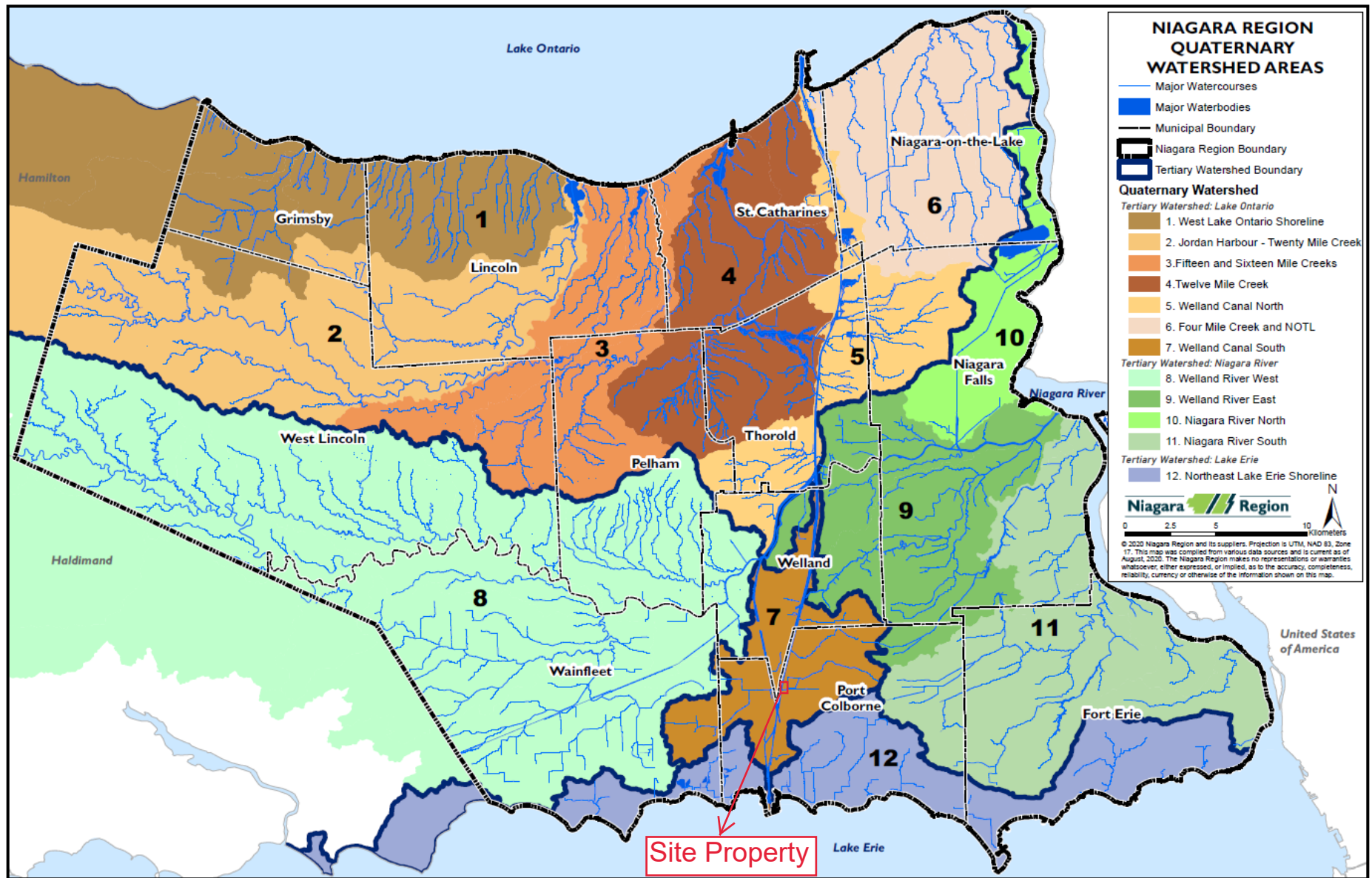
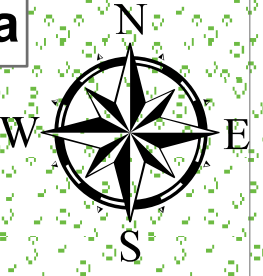


Figure 2-2. Niagara Region – Quaternary Watershed Areas



APPENDIX VIII–PRE- AND POST-DEVELOPMENT CATCHMENT AREA

Pre-Development Catchment Area



Indian Creek

Indian Creek

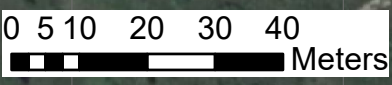
Indian Creek

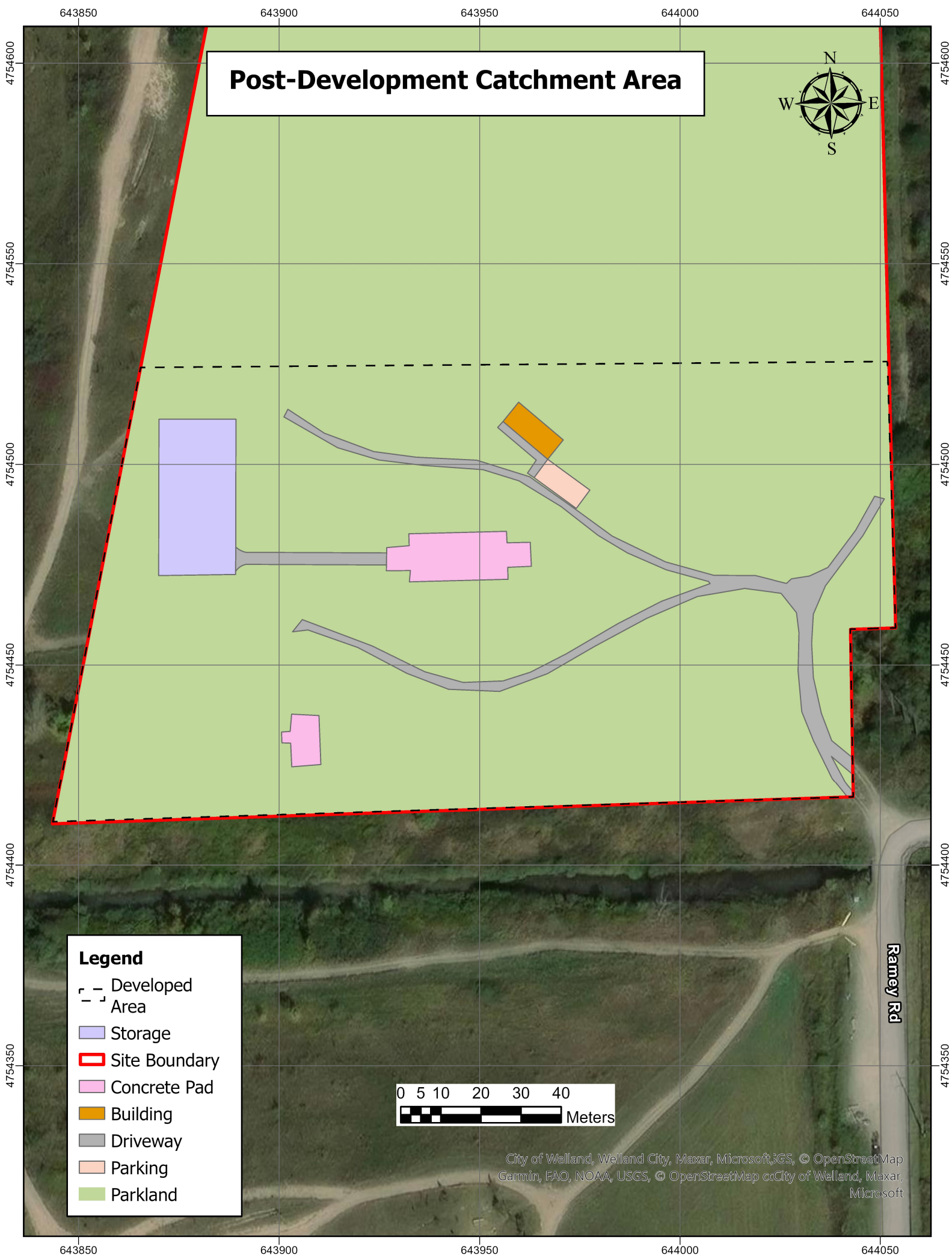
Ramey Rd

Ramey Rd

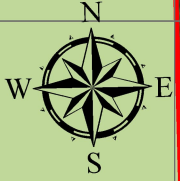
Legend

-  Site Boundary
-  Concrete Pad
-  Driveway
-  Parkland

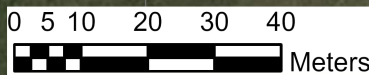




Post-Development Catchment Area



- Legend**
- Developed Area
 - Storage
 - Site Boundary
 - Concrete Pad
 - Building
 - Driveway
 - Parking
 - Parkland



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Garmin, FAO, NOAA, USGS, © OpenStreetMap ccCity of Welland, Maxar,
Microsoft

APPENDIX IX – RUNOFF COEFFICIENT CALCULATIONS

PRE-DEVELOPMENT (Runoff coefficient)

Ref.: Runoff Coefficients, Section 22, Appendix C, MTO Drainage Management Manual 1997 design chart 1.07 for rural & urban and Table 4.1, Stormwater Management Guidelines, Niagara Region, 2022.

Entire Site Boundary

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m ²)	A × C	A × % Imp.
Concrete Pad	0.95	100	459.7	436.7	459.7
Driveway	0.95	100	841.3	799.2	841.3
Parkland	0.35	0.0	329628.4	115369.9	0.0
Weighted Average			330929.4	0.35	0.39%

Developed Area

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m ²)	A × C	A × % Imp.
Concrete Pad	0.95	100	459.7	436.7	459.7
Driveway	0.95	100	841.3	799.2	841.3
Parkland	0.35	0.0	20355.3	7124.3	0.0
Weighted Average			21656.3	0.39	6.0%

POST-DEVELOPMENT (Runoff coefficient)

Ref.: Runoff Coefficients, Section 22, Appendix C, MTO Drainage Management Manual 1997 design chart 1.07 for rural & urban and Table 4.1, Stormwater Management Guidelines, Niagara Region, 2022.

Entire Site Boundary

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m ²)	A × C	A × % Imp.
Building	0.95	100	93	88.4	93
Driveway/Walkway	0.95	100	994.3	944.6	994.3
Parking	0.95	100	74.3	70.6	74.3
Concrete Pad	0.95	100	459.7	436.8	459.7
Storage	0.95	100	746	708.7	746
Parkland	0.35	0.0	328,562.1	114,996.7	0.0
Weighted Average			330929.4	0.35	0.72%

Developed Area

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m ²)	A × C	A × % Imp.
Building	0.95	100	93	88.4	93
Driveway	0.95	100	994.3	944.6	994.3
Parking	0.95	100	74.3	70.6	74.3
Concrete Pad	0.95	100	459.7	436.8	459.7
Storage	0.95	100	746	708.7	746
Parkland	0.35	0.0	19,289.0	6751.1	0.0
Weighted Average			21656.3	0.42	10.9%

APPENDIX X – STORMWATER MANAGEMENT CALCULATIONS

STORMWATER MANAGEMENT CALCULATIONS

Pre-Development Peak Flows

City of Port Colborne Modified Rational Method
Storm (yrs) Coeff A Coeff B Coeff C $Q = CaIA/360$

2	755	0.789	8.0
5	830	0.777	7.3
10	860	0.763	6.5
25	900	0.745	5.2
50	960	0.736	5.1
100	1020	0.731	4.7

Where:

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

Area # Developed Area

Area 2.17 ha
Runoff Coefficient 0.39

Time of Concentration 10.0 min

Return Rate 2 year
Peaking Coefficient 1.0
Rainfall Intensity 77.19 mm/hr

Pre-Development Peak Flow 179.25 l/s

Return Rate 5 year
Peaking Coefficient 1.0
Rainfall Intensity 90.60 mm/hr

Pre-Development Peak Flow 210.39 l/s

Return Rate 10 year
Peaking Coefficient 1.0
Rainfall Intensity 101.29 mm/hr

Pre-Development Peak Flow 235.22 l/s

Return Rate 25 year
Peaking Coefficient 1.1
Rainfall Intensity 118.51 mm/hr

Pre-Development Peak Flow 302.75 l/s

Return Rate 50 year
Peaking Coefficient 1.2
Rainfall Intensity 130.18 mm/hr

Pre-Development Peak Flow 362.78 l/s

Return Rate 100 year
Peaking Coefficient 1.25
Rainfall Intensity 142.99 mm/hr

Pre-Development Peak Flow 415.07 l/s

Post-Development Peak Flows

City of Port Colborne Modified Rational Method
Storm (yrs) Coeff A Coeff B $Q = CaIA/360$

2
5
10
25
50
100

755	0.789	8.0
830	0.777	7.3
860	0.763	6.5
900	0.745	5.2
960	0.736	5.1
1020	0.731	4.7

Where:

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

Area # Developed Area

Area 2.17 ha
Runoff Coefficient 0.42

Time of Concentration 10.0 min

Return Rate 2 year
Peaking Coefficient 1.0
Rainfall Intensity 77.19 mm/hr

Post-Development Peak Flow 192.97 l/s

Return Rate 5 year
Peaking Coefficient 1.0
Rainfall Intensity 90.60 mm/hr

Post-Development Peak Flow 226.50 l/s

Return Rate 10 year
Peaking Coefficient 1.0
Rainfall Intensity 101.29 mm/hr

Post-Development Peak Flow 253.22 l/s

Return Rate 25 year
Peaking Coefficient 1.1
Rainfall Intensity 118.51 mm/hr

Post-Development Peak Flow 325.92 l/s

Return Rate 50 year
Peaking Coefficient 1.2
Rainfall Intensity 130.18 mm/hr

Post-Development Peak Flow 390.54 l/s

Return Rate 100 year
Peaking Coefficient 1.25
Rainfall Intensity 142.99 mm/hr

Post-Development Peak Flow 446.83 l/s

Allowable Release Rate

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

Where:

Q = Allowable Post-Development Release Rate

Q_{PRE} = Pre-Development Flow

Q_U = Post-Development Uncontrolled Flow

Storm	Pre-Development Flow	Post- Development Uncontrolled Flow	Allowable Outflow
(yrs)	Q _{pre} (l/s)	Q _U (l/s)	Q (l/s)
2	179.2	0.0	179.2
5	210.4	0.0	210.4
10	235.2	0.0	235.2
25	302.7	0.0	302.7
50	362.8	0.0	362.8
100	415.1	0.0	415.1

Quantity Control Volume Calculations

Developed catchment Area

Drainage Area (ha)	2.17
Runoff Coeff. (C)	0.42
Time of Concentration	10 min
Time Step	1 min
Controlled Release Rate (Q_c) (l/s)	179.2 - 415.1
Max. Storage Required (m3)	59

Results		
Storm Event	Storage	Time
(yr)	(m3)	(min)
2	21.7	5
5	26.2	5
10	30.3	5
25	42	4
50	50.1	4
100	59	4

Required Storage Volumes (2yr)

T	$I = A/(t+B)^C$	$Q_p = 0.0028(C \cdot Cat \cdot A)$	$V_p = Q_p \cdot T.60$	$V_c = Q_c \cdot T.60$	$V = V_p - V_c$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(l/s)	(m3)	(m3)	(m3)
1	133.37	336.1	20.2	10.8	9.4
2	122.73	309.3	37.1	21.5	15.6
3	113.84	286.9	51.6	32.3	19.4
4	106.29	267.8	64.3	43.0	21.3
5	99.78	251.4	75.4	53.8	21.7
6	94.11	237.2	85.4	64.5	20.9
7	89.13	224.6	94.3	75.3	19.0
8	84.70	213.5	102.5	86.0	16.4
9	80.75	203.5	109.9	96.8	13.1
10	77.19	194.5	116.7	107.5	9.2

21.7 : Maximum Storage Volume

Required Storage Volumes (5yr)

T	$I = A/(t+B)^C$	$Q_p = 0.0028(C \cdot Cat \cdot A)$	$V_p = Q_p \cdot T.60$	$V_c = Q_c \cdot T.60$	$V = V_p - V_c$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(l/s)	(m3)	(m3)	(m3)
1	160.31	404.0	24.2	12.6	11.6
2	146.75	369.8	44.4	25.2	19.1
3	135.55	341.6	61.5	37.9	23.6
4	126.14	317.9	76.3	50.5	25.8
5	118.09	297.6	89.3	63.1	26.2
6	111.13	280.1	100.8	75.7	25.1
7	105.05	264.7	111.2	88.4	22.8
8	99.67	251.2	120.6	101.0	19.6
9	94.89	239.1	129.1	113.6	15.5
10	90.60	228.3	137.0	126.2	10.7

26.2 : Maximum Storage Volume

Required Storage Volumes (10yr)

T	$I = A/(t+B)^C$	$Q_p = 0.0028(C \cdot Cat \cdot A)$	$V_p = Q_p \cdot T.60$	$V_c = Q_c \cdot T.60$	$V = V_p - V_c$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(l/s)	(m3)	(m3)	(m3)
1	184.85	465.8	28.0	14.1	13.8
2	168.02	423.4	50.8	28.2	22.6
3	154.35	389.0	70.0	42.3	27.7
4	143.00	360.4	86.5	56.5	30.0
5	133.41	336.2	100.9	70.6	30.3
6	125.19	315.5	113.6	84.7	28.9
7	118.05	297.5	124.9	98.8	26.1
8	111.78	281.7	135.2	112.9	22.3
9	106.24	267.7	144.6	127.0	17.5
10	101.29	255.2	153.1	141.1	12.0

30.3 : Maximum Storage Volume

Required Storage Volumes (25yr)

T	$I = A/(t+B)^C$	$Q_p = 0.0028(C.Ca.I.A)$	$V_p = Q_p \cdot T.60$	$V_c = Q_c \cdot T.60$	$V = V_p - V_c$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(l/s)	(m3)	(m3)	(m3)
1	231.16	640.8	38.4	18.2	20.3
2	206.79	573.2	68.8	36.3	32.5
3	187.69	520.3	93.7	54.5	39.2
4	172.27	477.5	114.6	72.7	42.0
5	159.53	442.2	132.7	90.8	41.8
6	148.79	412.5	148.5	109.0	39.5
7	139.61	387.0	162.5	127.2	35.4
8	131.65	364.9	175.2	145.3	29.8
9	124.68	345.6	186.6	163.5	23.1
10	118.51	328.5	197.1	181.6	15.5

Maximum Storage Volume

Required Storage Volumes (50yr)

T	$I = A/(t+B)^C$	$Q_p = 0.0028(C.Ca.I.A)$	$V_p = Q_p \cdot T.60$	$V_c = Q_c \cdot T.60$	$V = V_p - V_c$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(l/s)	(m3)	(m3)	(m3)
1	253.67	767.1	46.0	21.8	24.3
2	226.85	686.0	82.3	43.5	38.8
3	205.89	622.6	112.1	65.3	46.8
4	188.98	571.5	137.2	87.1	50.1
5	175.02	529.3	158.8	108.8	49.9
6	163.27	493.7	177.7	130.6	47.1
7	153.23	463.4	194.6	152.4	42.2
8	144.53	437.1	209.8	174.1	35.7
9	136.91	414.0	223.6	195.9	27.7
10	130.18	393.7	236.2	217.7	18.5

Maximum Storage Volume

Required Storage Volumes (100yr)

T	$I = A/(t+B)^C$	$Q_p = 0.0028(C.Ca.I.A)$	$V_p = Q_p \cdot T.60$	$V_c = Q_c \cdot T.60$	$V = V_p - V_c$
Time	Rainfall Intensity	Runoff	Runoff Vol.	Controlled Release Vol.	Storage Vol.
(min)	(mm/hr)	(l/s)	(m3)	(m3)	(m3)
1	285.80	900.3	54.0	24.9	29.1
2	253.94	799.9	96.0	49.8	46.2
3	229.39	722.6	130.1	74.7	55.4
4	209.80	660.9	158.6	99.6	59.0
5	193.76	610.4	183.1	124.5	58.6
6	180.35	568.1	204.5	149.4	55.1
7	168.95	532.2	223.5	174.3	49.2
8	159.12	501.2	240.6	199.2	41.4
9	150.54	474.2	256.1	224.1	31.9
10	142.99	450.4	270.2	249.0	21.2

Maximum Storage Volume

Swale Size Calculation

S = 0.5%, Width = 1.2m, Depth = 0.5m, Freeboard = 0.1 m

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

0th Forks Rd			
Swale Flow Capacity			
Inputs		Results	
Bottom width, b	1.2 m	Flow area, a	0.9600 m ²
Side slope 1 (horiz./vert.)	3	Wetted perimeter, P _w	3.7298 m
Side slope 2 (horiz./vert.)	3	Hydraulic radius, R _h	0.2574 m
Manning roughness, n ? ☐ Strickler ☐ B/B (See notes)	0.03	Velocity, v	0.9537 m/s
Channel slope, S	0.5 % rise/run	Flow, Q	915.5675 l/s
Flow depth, y	0.40 m	Velocity head, h _v	0.0464 m
Bend Angle ? (for riprap sizing)	0	Top width, T	3.6000 m
Rock specific gravity (2.65)	2.65	Froude number, F	0.59
Design rock size, D50 ☐ Isbash ☐ Maynard ☐ Searcy * 1.25 (See notes)	0 m	Average shear stress (tractive force), tau	12.6196 N/m ²
		n for design rock size per Strickler	0.0277
		n for design rock size per Blodgett	0.0390
		n for design rock size per Bathurst	0.0252
		Blodgett vs. Bathurst	Blodgett
		Required bottom angular rock size, D50 (Isbash & MC) ?	0.0345 m
		Required side slope 1 angular rock size, D50 (Isbash & MC) ?	0.0364 m
		Required side slope 2 angular rock size, D50 (Isbash & MC) ?	0.0364 m
		Required angular rock size, D50 (Maynard, Ruff, and Abt 1989)	0.0306 m
		Required angular rock size, D50 (Searcy 1967)	0.0200 m

