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STORMWATER MANAGEMENT

AT

25 PAGEWOOD CRT,
TOWN OF EAST GWILLIMBURY, ON

PREPARED FOR:

WeLeap Investments Ltd.

March 14th, 2025

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1. Site Background

King EPCM (the Engineer) was retained by Jonathan Benczkowski (Owner) for geotechnical and environmental engineering services, including the creation of a Stormwater Management Plan (SWM). The property is located at 25 Pagewood Crt., Town of East Gwillimbury, Ontario (the Site). It is understood that the SWM is for the sole purpose of the application and construction of 4 industrial units for the manufacturing and sale of architectural stones with a new driveway to access the buildings plus walkway and parking lots (10177.1 m^2) while the southern portion of the subject property is known as natural heritage and meadow and protected by fence (1585.4 m^2). This report is to be submitted to the Town of East Gwillimbury, Lake Simcoe Region Conservation Authority (LSRCA), and the Regional Municipality of York (York Region).

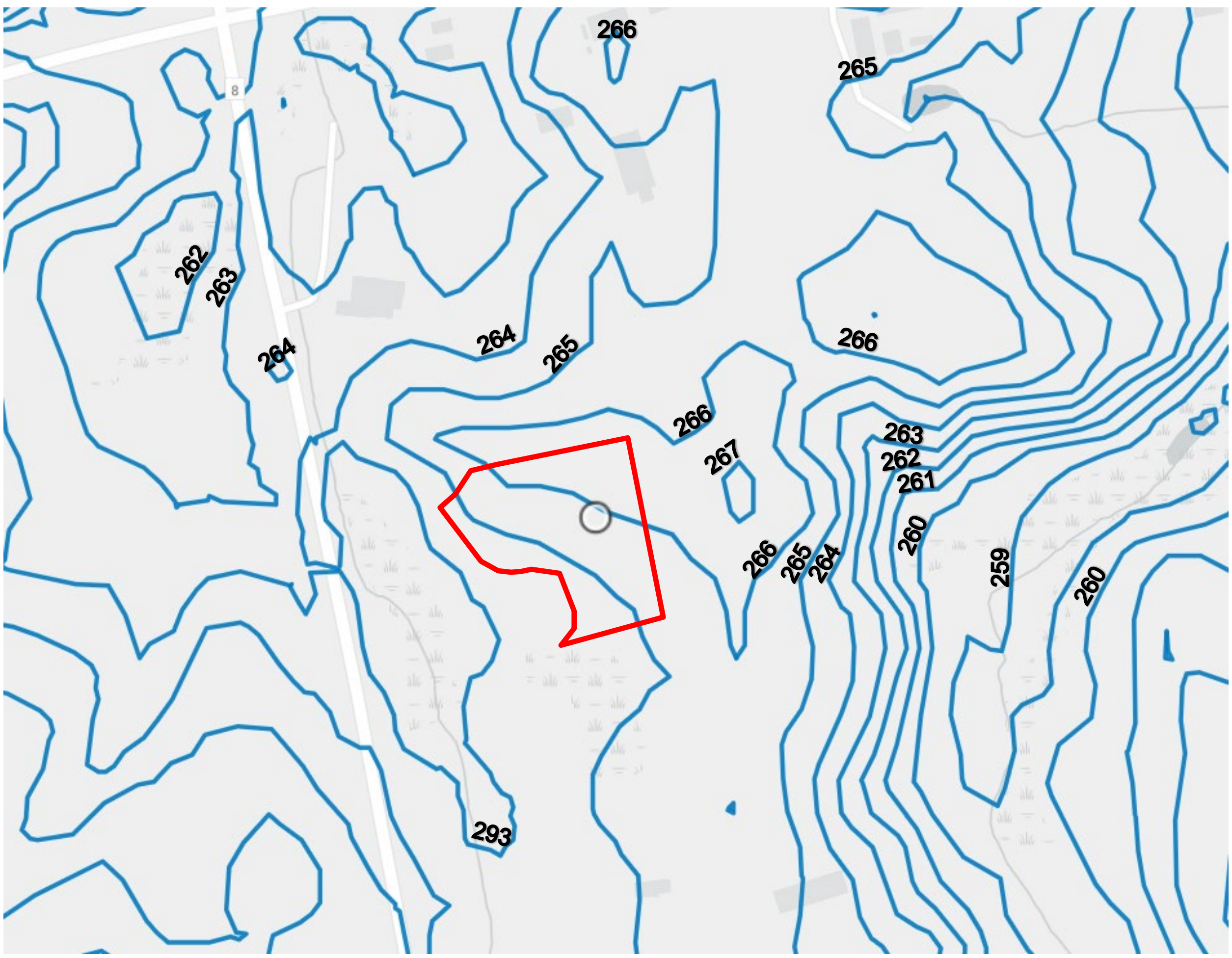

Figure 1 – Topographic map of 25 Pagewood Crt., Town of East Gwillimbury, ON

The site property is considered as two separate trapezoids which are connected from the smaller bases. The southern trapezoid is completely permeable consisting of grass and trees, but the northern part will be developed and is located the south of Pagewood Crt, east of Woodbine Avenue, between Mt. Albert Rd. and Herald Rd., within the Town of East Gwillimbury. The site was previously used for agricultural purposes, while the southern portion was vacant with existing trees and meadows. The proposed project is composed of four industrial units for the manufacturing and sale of architectural stones with a new driveway to access the buildings plus walkway and parking lots (9740.5 m^2 building, walkway, parking, and driveway + 436.6 m^2 septic bed area = 10177.1 m^2 Total Developed area) while the southern portion of the subject property is known as natural heritage and meadow (1585.4 m^2), post-development TIMP = 9181.5 m^2 = 78.1% (Figure 2).

This Site is located on a downhill of a rolling hill at the western end within an Ecologically Significant Groundwater Recharge Area (ESGRA) and a Significant Groundwater Recharge Area (SGRA). As well, the property is identified as being within the Recharge Management Area (WHPA Q1 & Q2) per the South Georgian Bay Lake Simcoe Source Protection Plan.

This site was located within the Schomberg Clay physiographic region and the Black River Subwatershed, with a shallow slope from northeast to southwest at a 2 - 4% gradient. The material of the site was mainly sandy clay with a lower silt layer and low permeability.

The Black River is located almost entirely within the Regional Municipality of York, except for a small portion of the subwatershed in the Durham Region. The subwatershed is 375 km^2 in area, and over 89% of its area lies within York Region. It lies approximately 160km in length in a south-north direction from the Oak Ridges Moraine in the south to Lake Simcoe in the north. Neighboring subwatersheds include the East Holland River to the south, the Pefferlaw River to the east, the Georgina Creeks to the northwest, and the Maskinonge River to the west.

This site is situated within the Black River Subwatershed, and it is also in close proximity to Harrison Creek (Black River Subwatershed) to the west. The Black River subwatershed is one of the largest subwatersheds in the Lake Simcoe basin. The subwatershed covers an area of 375 km^2 to the south of Lake Simcoe. The main tributaries of the subwatershed include Harrison Creek, Mount Albert Creek, Vivian Creek, and Zephyr Creek. As with many of the subwatersheds south of Lake Simcoe, the headwaters of the Black River originate in the Oak Ridges Moraine.

The Black River is one of the healthiest systems in the Lake Simcoe watershed. The largest component of land cover in the Black River subwatershed is natural heritage features at approximately 51%. Thirty-eight percent of this natural cover is occupied by forested lands. Of the lands that have been subject to alteration by humans, agriculture is the main land use, and there is a small amount of urban development. Intensive and non-intensive agriculture comprise approximately 39% of the land base.

The Black River subwatershed is one of 18 subwatersheds of Lake Simcoe, which drain into the southeastern portion of Lake Simcoe. It is also one of five major tributaries that account for 60 percent of the total drainage to Lake Simcoe.

There are no waterbodies within the site property, except a sideyard ditch along the lot boundary with the pond block in the west (Figure 3).

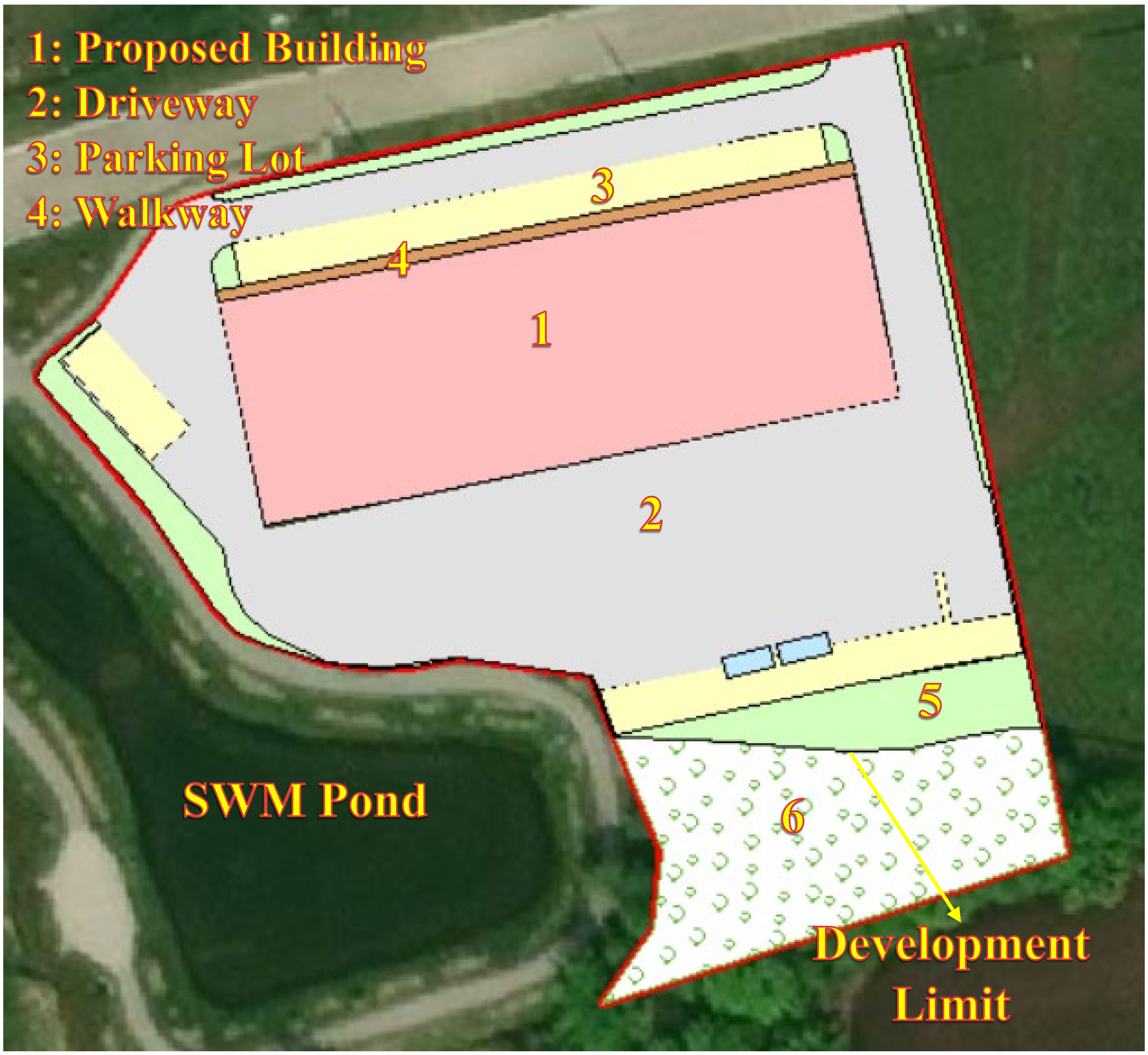

Figure 2 –Site Location Plan

In the above figure, the proposed developments are displayed which are:
#1: Building (TIMP = 3118 m^2)
#2: Driveway (TIMP = 4818.8 m^2)

#3: Parking lot (TIMP = 1108.5 m^2)
#4: Walkway (TIMP = 136.2 m^2)

and the future landscaped pervious areas which are located within the site boundary and around the proposed driveway plus a small portion of natural heritage at the south:
#5: Lawn (Total area = 995.6 m^2)
#6: Natural Heritage (Total area = 1585.4 m^2)

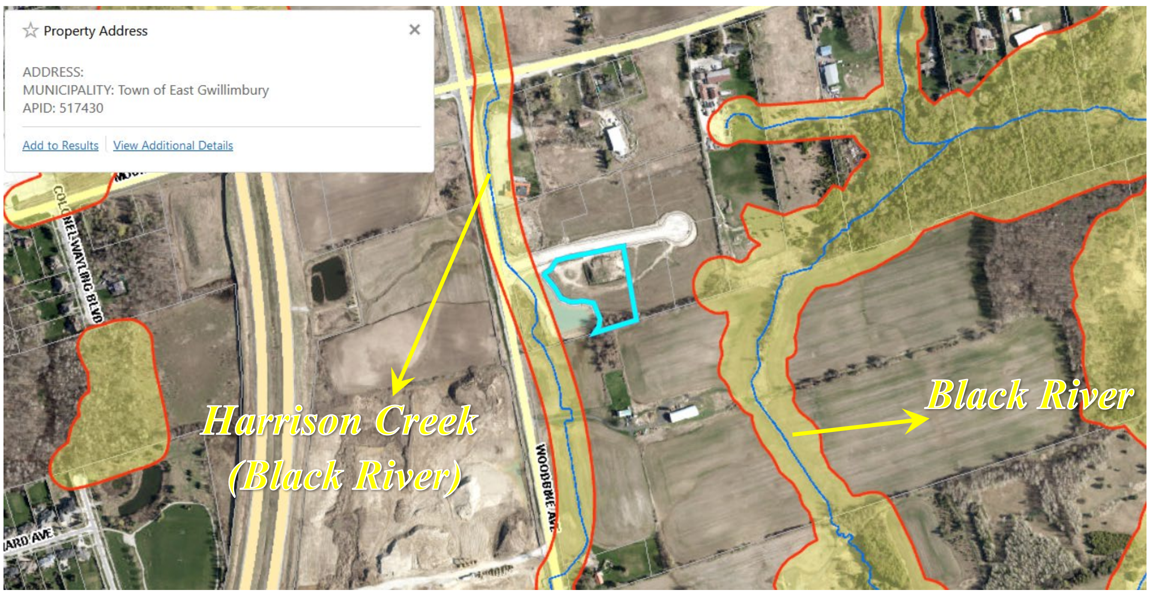

Figure 3 – LSRCA Regulated Area map, 25 Pagewood Crt., Town of East Gwillimbury, ON

2. Site Investigation & In-Situ Permeameter Testing

2.1. Monitoring Wells

Three (3) boreholes were drilled at the site property by King EPCM (O.Reg 903 License C-7691) and then developed into monitoring wells. Detailed borehole drill logs are in Appendix III, while Table 1 below shows the summary. Shallow groundwater has been observed in the boreholes since drilling in June 2023, March & April 2024 (Table 2). In general, the soil stratigraphy can be described in Table 3, which confirms the presence of sandy clay material layers under the topsoil.

Table 1 - Borehole Summary

Borehole #	Date	Northing (UTM)	Easting (UTM)	Surface Elev. (masl)	Hole Depth (m)	Screen Elevations (m)	Surface Soil type	Groundwater
101	28-June-2023	4,884,788	626,959	266.42	4.5	263.4-261.92	Backfill	yes
102	28-June-2023	4,884,748	626,938	265.58	3.5	263.57-262.08	Backfill soil and gravels	yes
103	29-June-2023	4,884,762	626,876	265.60	4.5	263.4-261.92	Backfill	yes

Table 2 –Water-level measurements(m) in monitoring wells, June, July & October 2023, March & April 2024

Borehole #	Date							Max GW (mgbl.)
	28-June, 2023	4-July, 2023	13-July, 2023	04-October, 2023	19-March, 2024	10-April, 2024	25-April, 2024	
101	262.4	264.36	263.95	Dry	264.52	265.44	265.12	0.98
102	263.48	264.35	264.0	Dry	264.25	265.06	264.87	0.52
103	261.6	263.57	263.27	Dry	263.82	264.50	264.41	1.10

Table 3 - Soil Stratigraphy Summary

	Top Layer	Middle Layer	Bottom Layer
From (m)	0 m	0.35	2.1
To (m)	0.35 m	0.35 – 2.1	2.1 – 4.45
Description	Top Soil	Sandy Clay	Silt
Primary Soil	-	Clay	Silt
Secondary Soil	-	Sand	-
Debris/Others	-	-	-
Cone Penetrometer Test	n/a	n/a	n/a
Shear Vane Test	n/a	n/a	n/a
Comments			Perched groundwater at around 2.1 – 4.0 m during drilling

2.2. In-Situ Permeameter Testing

Based on a field visit dated June 10, 2024, "field-saturated" hydraulic conductivity, K_{fs}, was achieved using the "Constant Head Well Permeameter" (CHWP) method. K_{fs} was conducted in the southern portion near the grass area or future septic bed, using ETC Standard Soils 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 bottom, 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 K_{fs} was calculated as:

K_{fs} = 9.4E-9 m/sec= 9.4E-7 cm/sec

Then using the temperature correction factor (for t=14$^{\circ c}$) from the manual:
K_a=6.9E-9 m/sec = 6.9E-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 = 49 min / cm (Infiltration Rate = 12.2 mm/hour)

The engineer's opinion is to trust the values obtained from the OMMAH (1997), with an unfactored surface infiltration rate of 12.2 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 4.9 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

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.
- This site is located within the Schomberg Clay Plain physiographic region and Black River Subwatershed, with a gentle slope to the southwest at a 2 - 4% gradient. This plain is described as having low permeability.
- Site survey and topographic information in Appendix I, Grading and Servicing Plans in Appendix II.
- There is no stream within the site property, however, there is a sideyard ditch along the lot boundary with the pond block in the west, which flows west and drains into a tributary connected to Harrison Creek.
- Base yearly precipitation is 895mm/year (LSRCA Climate Data, Appendix V)
- The site is within the Black River Subwatershed, with Sandy Clay Hydrologic Soil Group C/D.

3.2. Vegetation & Evapotranspiration

- The site property is considered as two separate trapezoids which are connected from the smaller bases. The southern trapezoid is completely permeable consisting of grass and trees, but the northern part will be developed and is located south of Pagewood Crt., east of Woodbine Avenue, between Mt. Albert Rd. and Herald Rd., within the Town of East Gwillimbury. The site was previously used for agricultural purposes, while the southern portion was vacant with existing trees and meadow, TIMP = 0%.

- The subject land is mostly agricultural fields (>85%) with several tree covers along the southern boundary (13.5%).
- ~1585.4m^2 / 13.5% of the site is natural heritage (Meadow with trees), with evapotranspiration of 632mm/year.
- Averaged site evapotranspiration is 649mm/year (LSRCA Climate Data)

3.3.Precipitation Surplus (Recharge + Runoff)

- Net surplus (precipitation surplus) = 895 – 649 = 246 mm/year
- Recharge for pervious area based on MOEE Hydrogeological Tech Info, 1995, Table 2: Infiltration factors
  - Topography (Rolling land) = 0.3
  - Soil (Sandy Clay) = 0.2
  - Cover = 0.15 (Meadow with some trees) or 0.1 (cultivated)
  - MOE infiltration factor = 0.6 – 0.65
  - The Recharge rate for pervious area would be 0.6*243 + 0.65*263 = 150 mm/year
- Storm runoff from the pervious area = 246 – 150 = 96 mm/year
- Total area-weighted average storm runoff = 96mm/year
- Total area-weighted average recharge = 150mm/year
- Total area-weighted average precipitation surplus (recharge + runoff) = 246mm/year.

3.4.Stormwater Run-off

- There is a stormwater sewer system within this site to collect the runoff from each lot plus roadway catchbasins and convey it to the constructed wet pond in the southwest corner of the subdivision.
- There is a SWM Pond downstream of the Commercial/Industrial Subdivision which will provide water quantity control as long as the post-development runoff coefficient is equal to or less than 0.76. The Storm Drainage Plan from the commercial/industrial subdivision has been included in Appendix XIV with the site location emphasized.
- A small portion of the site which will remain undeveloped/meadow/landscaped will continue to drain to the southwest outfall.
- All stormwater in the existing creek/ditch is generally infiltrated or moved downstream, with no visual presence of any semi-aquatic vegetation or standing water in the ditch during multiple inspections. The historical well records and new monitoring boreholes drilled by the engineer within the Site, near the proposed development, confirm that the seasonal groundwater level in this property is more than 0.5 – 1.1m below ground.
- All minor and major storm site drainage is to be collected on-site and discharged into the existing wet pond. If there is any overflow during the less frequent rainfall event, it would be discharged into the existing creek, along Woodbine Ave.

- Based on an in-situ infiltration test using ETC Pask Permeameter Apparatus, the infiltration rate was 49 min/cm = 0.20 mm/min = 12.2 mm/hour in this site.
- Native permeable infiltration rate should use a non-factored infiltration rate of 12.2 mm/hour because the predominant soil material of this site is composed of sandy clay to a depth of around 3 meters, and a factored engineering design infiltration rate of 4.9 mm/hour with a safety factor of 2.5 for infiltration purposes (infiltration trench or Septic bed).
- Based on the Town of East Gwillimbury requirements, the Regional Storm is the Hurricane Hazel event, with data from Toronto City EC Station 6158355, with A, B, C values of 3-parameter Chicago distribution design storm as Intensity = $A \times (t+B)^{-C}$.
- See Table 4: Chicago Distribution Design Storm Parameters and Rainfall Amounts cited as a table on Page 43 of the Town of East Gwillimbury "Engineering Standards and Design Criteria, September 2012" showing the IDF values (Appendix VI).

Table 4 - Town of East Gwillimbury Chicago Distribution Storm Parameters

Return Period	2 Year	5 Year	10 Year	25 Year	50 Year	100 Year
A	648	930	1021	1100	1488	1770
B	4	4	3	2	3	4
C	0.784	0.798	0.787	0.776	0.803	0.820

	Return Period (year)					
IDF Parameters	2	5	10	25	50	100
a	648	930	1021	1100	1488	1770
b	4	4	3	2	3	4
c	0.784	0.798	0.787	0.776	0.803	0.82
Duration						
5 min	115.73	161.06	198.74	242.99	280.17	292.08
10 min	81.85	113.21	135.63	159.94	189.72	203.31
15 min	64.42	88.72	104.98	122.06	146.09	158.27
30 min	40.82	55.77	65.16	74.71	89.79	98.21
1 hr	24.86	33.66	39.17	44.72	53.42	58.47
2 hr	14.80	19.86	23.14	26.45	31.22	33.99
4 hr	8.71	11.57	13.54	15.54	18.07	19.51
6 hr	6.36	8.41	9.87	11.37	13.09	14.06
12 hr	3.71	4.86	5.74	6.66	7.53	8.00
24 hr	2.16	2.80	3.33	3.89	4.32	4.54

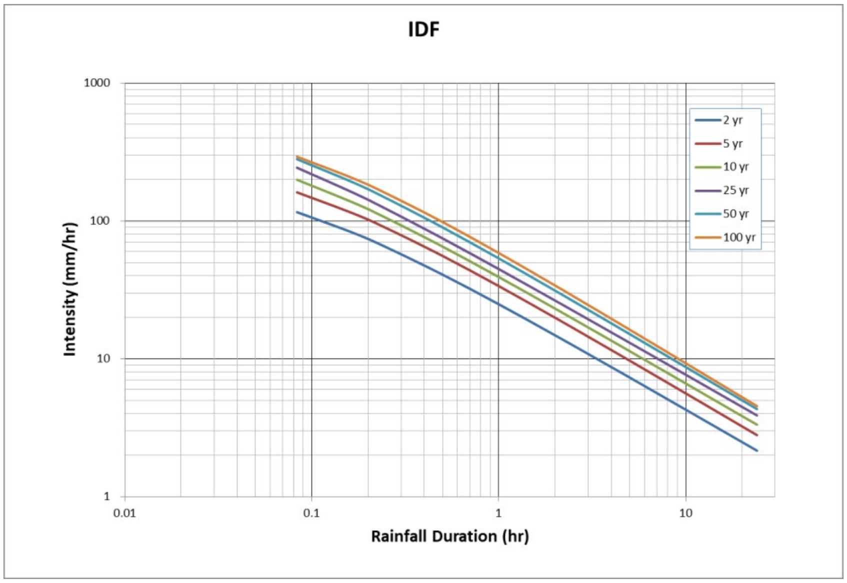

Figure 4- Intensity-Frequency-Duration (IDF) Curves calculated from Table 2 above

According to the LSRCA Technical Guidelines for Stormwater Management (Section 1.2), two methods are generally used to derive synthetic design storms. One method develops the storm hyetograph from the IDF curve or using uniform rainfall methods, named the Chicago design storm (Keifer and Chu, 1957) or the Rational Method. 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.

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 included the methods applying the Soil Conservation Service (SCS) design storm, and the Canadian Atmospheric Environment Service (AES) design storm.

<u>LSRCA recommended manual calculations such as the Rational or Modified Rational Method when there is a total development area of less than 5 hectares, and the Engineer has used the Modified Rational Method for this site.</u> These methods are 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 the 1 in 100-year design storm events, flow conditions are calculated as follows:
- Modified Rational Formula: Qp = (0.001/3600) * A * C * Ca * i
    - A = area in m^2 = 11,762.5m^2 (Total Site Area)**
        - A_{north} = 9740.5 m^2 (Main development)
        - A_{south} = 436.6 m^2 (Septic bed area)
        - $A_{undisturbed\ area}$ = 1585.4 m^2 (Natural Heritage)

North Catchment (A-B1) (Controlled)

South Catchment (B2-B3) (Uncontrolled)

** Note: The Engineer unselected conservatively the effect of Key Natural Heritage and/or undeveloped area (1585.4 m^2) 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.

- C = runoff coefficient = 0.1 (Entire site with 0% TIMP, rolling land, agriculture & meadow/tree, average runoff coefficient would be 0.1 based on MTO Drainage Management Manual & SCS Consulting Group Ltd Stormwater Management Report for Woodbine and Mount Albert Commercial / Industrial Subdivision. See Appendix IX for more details). **According to the pre-development subdivision agreement and design criteria for the downstream SWM pond, C = 0.76 for a 0.97 ha area (NORTH CATCHMENT), excluding the southern portion while the proposed septic bed area keeps C = 0.1 for a 436.6 m^2 area (SOUTH CATCHMENT).**
- 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 (4, 12 & 24 hour IDF for East Gwillimbury Table 4 above)
- Detailed calculations are contained within the attached Excel Spreadsheet.

- Time of Concentration:
  - Bransby Williams Method if $C > 0.4$, $S_w = 2\%$, $L = 135$ m, $A = 0.97$ ha*, C = 0.76: $\boldsymbol{T_c = 6.7}$ **min** $\Longrightarrow$ **(**$\boldsymbol{T_{c_{min}} = 8}$ **min**, based on township's guideline**)**
  - See Appendices IX, VIII & XIII for the drainage flow path and details of calculations.

* Controlled/developed area which will be discharged into the wet pond.

Table 5– Flow Rate Calculations using Modified Rational Method

(Pre-Development Scenario – Controlled)

Design Storm	2 yr Return Period (l/s)	5 yr Return Period (l/s)	10 yr Return Period (l/s)	25 yr Return Period (l/s)	50 yr Return Period (l/s)	100 yr Return Period (l/s)	Catchment
Intensity (mm/hr)	92.4	128.0	154.7	184.2	217.0	230.7	
Total Runoff Qp (l/s) (NORTH)	189.9	263.3	318.1	416.8	535.3	593.0	**A & B1**
Total Runoff Qp (l/s) (SOUTH)	1.1	1.6	1.9	2.5	3.2	3.5	**B2**

4. Stormwater Management Plan

The SWM plan has been prepared in accordance with the MOE Stormwater Management Planning and Design Manual, Town of East Gwillimbury Standards, and LSRCA Stormwater Management Policies and Guidelines as detailed in Section 12. The SWM plan is subject to review and approval by the Township, LSRCA, 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 both infiltration trench facility and wet pond (permanent pool and extended detention) 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.Proposed SWM Plan

The Client has proposed to construct four industrial units for the manufacturing and sale of architectural stones with a new driveway to access the buildings plus walkway and parking lots. Since the runoff coefficients for post-dev of 0.86 (NORTH catchment or Catchment A-B1) and 0.15 (SOUTH catchment or Catchment B2) are greater than the pre-dev subdivision site plan agreement of 0.76 (NORTH catchment) and 0.1 (SOUTH catchment), we need SWM control measures of BOTH peak flow rates as well as total volume discharged. A proposed Drainage Plan illustrating the proposed drainage conditions for the development is enclosed in Appendix XIII.

Due to the poorly draining sandy clay soil surrounding the site and high seasonal groundwater table observed during the snowmelt season, April to June, the Engineer proposed that parts of Timp area, i.e., southern driveway and parking lots (4747.2 m^2) plus the southern half of rooftops (1559 m^2) are to be connected into two buried storage tanks, below the parking lots, and outlets by a pump into the proposed underground sewer system to discharge to the existing swm wet pond at the west of the site.

To achieve this purpose, two separate holding tanks (Appendix XV) will be installed within the southern portion of the driveway and parking lots to reduce the 2 through 100-year post-development peak flows to pre-development values, with a total static volume capacity of 93.7 m^3.

Catchment A: The northern portion of the parking lots and driveway (1875.3 m^2) plus the northern half of rooftops (1559 m^2) are collected by several catchbasins directly and enter the existing subdivision storm sewer system to convey to the pond.

Catchment B1: The Engineer proposes that rooftop downspouts (southern half of the buildings) plus the southern asphalt area (parking lots and driveways) be captured by several catchbasins directly and then directed to the holding tanks through a network of storm sewer pipes. A 250 mm diameter orifice pipe (invert elevation = 260.9m, 100mm above the bottom of the tank) is proposed to reduce the 2 through 100-year post-development peak flows to pre-development values. In case of blockage to the outlet structure, a second overflow orifice (400 mm dia.) will act as an emergency overflow spillway (invert elevation = 262.8m) which is proposed at the east wall of the second tank to safely convey the peak flow rate from 100-year event, which is the greatest flow rate. The bottom of the tank is 260.8 m (Interior) and/or 260.65 m (Exterior), while the top of the tank is set at 263.70 m, providing 1.8 m of top cover. Furthermore, both outlets will be connected to a manhole and then a single pipe along with a pump will be installed to discharge the water towards the north, where runoff will enter the subdivision storm sewer system and discharge into the wet pond. The following Table 7 contains a summary of the outflow from the tank and the associated storage volume and water level inside the tank.

Moreover, additional storage (140 m^3) is also can be provided on the driveway/parking areas (above-ground storage @ 250mm maximum depth in the southern portion) to achieve the required quantity control for the 1:100-year design storms or other less frequent storm events. Furthermore, there is a shallow grading towards the southwest to overflow to the pond in cases where the depth of the ponding water exceeds the maximum permissible depth of 250 mm.

Below is a summary of the proposed site conditions:

- Total site surface area = 11,762.5m^2 (Developed = 10177.1 m^2, Undeveloped= 1585.4 m^2)
  - A_{north} = 9740.5 m^2 (Main development) — North Catchment (A-B1) (Controlled)
  - A_{south} = 436.6 m^2 (Septic bed area)
  - $A_{undisturbed\ area}$ = 1585.4 m^2 (Natural Heritage)

  South Catchment (B2-B3) (Uncontrolled)
- Total pervious surface area (tree+ grass) = 2581m^2 (21.9%)
- Total impervious surface area (TIMP) = 9181.5m^2 (78.1%)
  - 3118m^2 – new building
  - 136.2m^2 – new walkway
  - 4818.8m^2 – new driveway
  - 1108.5m^2 – new parking lot

- The northern half of the rooftops discharge directly into the storm sewer system through downspouts and are conveyed into the wet pond.
- The runoff captured by the northern driveway and parking lots is also discharged into the wet pond through the storm sewer system while the southern driveway and parking lots plus the southern half of the building rooftops are discharged directly into the storage tank through the on-site catchbasins and sewer system.
- All minor & major storm site drainage from the southern portion is to be collected on-site and outlet by storm sewer system to a proposed underground series stormwater concrete tank at the south boundary, below the parking lots. These tanks shall be controlled with an outlet control device (orifice) to restrict the stormwater flowing into the sewer system and then directed to the wet pond with a pump.
- Post-development flow conditions are calculated as follows:
- Modified Rational Formula: Qp = (0.001/3600) * A * C * Ca * i
  - A = area in m^2 = 11,762.5 m^2
  - C = runoff coefficient = 0.73 (Entire site with 78.1% TIMP, rolling land, agriculture & meadow/tree, average runoff coefficient would be 0.73 based on MTO Drainage Management Manual. See Appendix IX for more details). **According to the pre-development subdivision agreement and design criteria for the downstream SWM pond, C = 0.76 for a 0.97 ha area (CATCHMENT A-B1), excluding the southern portion while the proposed septic bed area has C = 0.15 for a 436.6 m^2 area (CATCHMENT B2). However, for the proposed controlled/developed area (0.97 ha), C = 0.86 (13% increase compared to the subdivision agreement).**
  - 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 (4, 12 & 24 hour IDF for East Gwillimbury Table 4 above)
  - Detailed calculations are contained within the attached Excel Spreadsheet.

- Time of Concentration:
  - Bransby Williams Method if $C > 0.4$, $S_w = 2\%$, $L = 108$ m, $A = 0.97$ ha*, C = 0.86: **Tc = 5.4 min** ⟹ **(Tc_{min} = 8 min**, based on township's guideline**)**

- See Appendices IX, VIII & XIII for the drainage flow path and details of calculations.

* Controlled/developed area which will be discharged into the wet pond.

- See Table 6: Flow rates for the 1:2-year through 1:100-year design storms based on the Modified Rational Method for both pre and post-development scenarios.

Table 6– Flow Rate Calculations using Modified Rational Method

(Post-Development Scenario - Controlled)

Design Storm	*2 yr Return Period (l/s)*	*5 yr Return Period (l/s)*	*10 yr Return Period (l/s)*	*25 yr Return Period (l/s)*	*50 yr Return Period (l/s)*	*100 yr Return Period (l/s)*
Intensity (mm/hr)	92.4	128.0	154.7	184.2	217.0	230.7
Total Runoff Qp (l/s) (NORTH)	214.9	297.9	359.9	471.6	605.8	671.0
Total Runoff Qp (l/s) (SOUTH)	1.7	2.3	2.8	3.7	4.7	5.2

See Appendix II & XIII for detailed design information and SWM flowchart (Figure 5 below).

Table 7 – SWM Tank Release Rates and Water Levels

	2 year	**5 year**	**10 year**	**25 year**	**50 year**	**100 year**
Peak Flow Discharge (l/s)	122.4	169.7	205.0	268.6	345.0	382.2
Storage Volume (m^3)	46.0	64.0	79.2	107.5	134.1	145.2
Water Elevation (m)	261.91	262.72	263.03	263.11	263.21	263.3
Water Depth in Tanks (m)	1.11	1.92	2.23	2.31	2.41	2.5

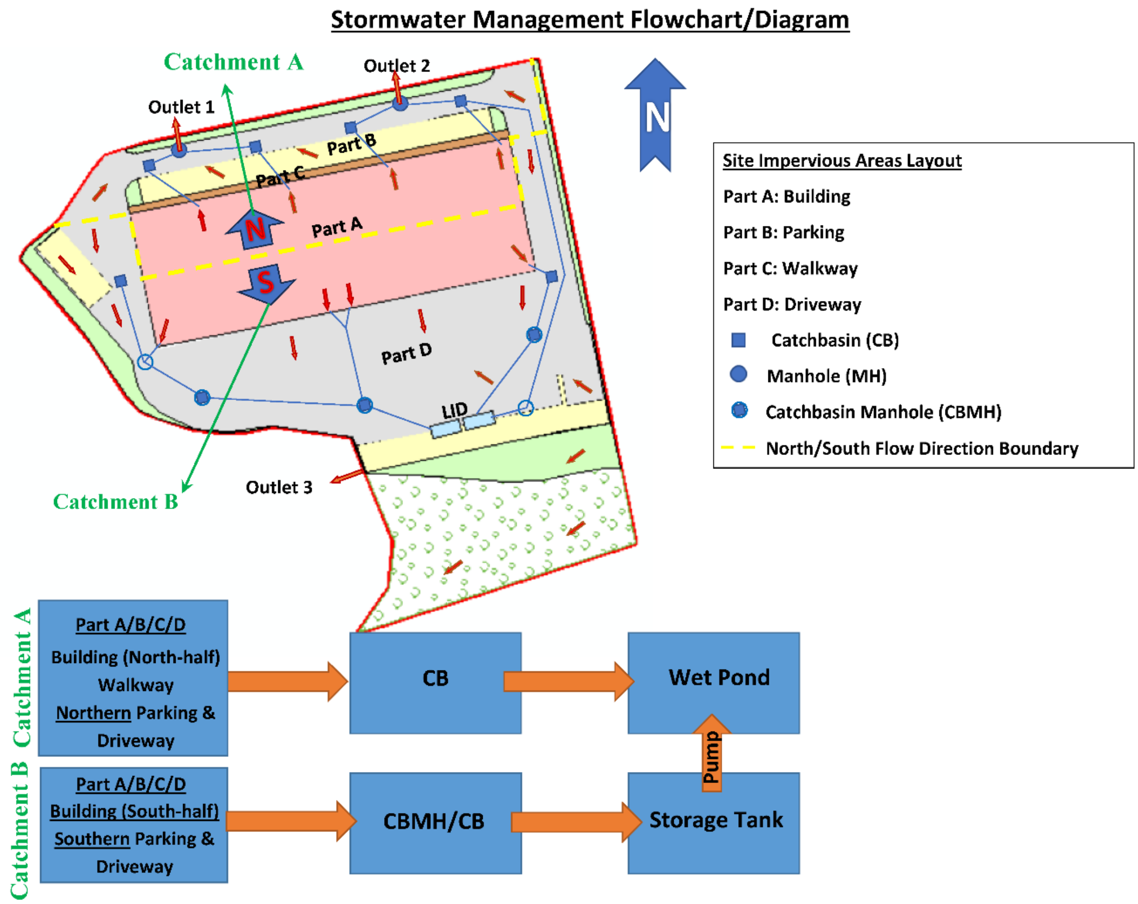

Figure 5- Stormwater Management Flowchart/Diagram

4.3.Water Holding Tank

Two underground stormwater holding tanks are installed in series along the south boundary of the site, below the parking lots that intercept runoff from impervious areas of the southern portion. The precast concrete underground holding tank is the superior choice for stormwater capture and management runoff and ensuring that the accumulated stormwater does not cause any environmental and flooding hazards in locations where different limitations such as: insufficient capacity in the City's sewer facility, very low permeability material and high groundwater table within the Site.

Underground storage tanks were provided to detain the southern half of rooftop runoffs along with southern parking lots/driveway, while parking lot ponding and storage within the storm sewers and manholes are providing quantity control and volume control for the site. Runoff captured by the northern half of rooftops along with northern parking lots and driveway discharges directly into the proposed catchbasins (storm sewer) and directs to the wet pond. The runoff captured by the other portions (southern half of the rooftops + southern driveway/parking lots) discharges into the holding tank through several catchbasins and underground sewer pipe network and then it will be controlled with an outlet control device (orifice) to restrict the stormwater flowing into the subdivision sewer system and direct to the wet pond with a pump.

Based on the 25mm volume control target (230 m^3), a series of two water holding tanks (49,851 litre each) will be connected through pipe (D= 450mm) along the south boundary which will be discharged into the existing wet pond through an orifice tube (250 mm dia.). An extra storage would be through storm sewer pipe storage (9.45 m^3) and parking lot ponding (140 m^3). Any overflow from the SWM tanks would be managed by an overflow orifice (400 mm dia.) which conveys an excess runoff to the storm sewer system through an outlet structure (manhole with outlet pipe dia. = 450 mm). Using this type of BMP (Water Holding Tank) along with an existing wet pond, both water quantity control and water quality control criteria were achieved. The tanks are designed to receive 1:2-year through 1:100-year, storms using the Town of East Gwillimbury IDF curve values. See Appendices VIII to XV for more details.

- Dimensions:
  - Depth (Inside) = 2.5 m
  - Height = 3054 mm
  - Width = 3040 mm
  - Length = 7315 mm

- Tank bottom elevation (m) = 260.65 m
- Tank top elevation (m) = 263.70 m
- Detailed diagrams and design maps are shown in Appendices II & XIII & XV.
- An amount of 140 m^3 of the volume control target would be managed via the ponding on the southern parking lot through catch basin restrictors or orifices in the storm sewer.
- For the remaining volume control target (**V = 90m^3**), the draw-down time of tank will be equal to 13.2 min based on the falling head orifice equation for total drawdown, above the sill of the orifice.
- A duplex pump system is proposed for discharging the tanks into the storm sewer system towards the wet pond. One pump can be used to handle the minor flow requirements, and if the peak flow is ever experienced, both pumps can be used to reach that pumping rate.
- The total tank height is 3.05 m plus a 1.8 m top cover along with 100 mm thick granular material bedding between the tank bottom and ground.
- The on-site controlled catchment area (Appendix XIII, Southern half of building rooftops plus southern parking lots and driveway = Catchments B1) is considered to be discharging to the storage tanks located south, below the parking, through several catchbasins and get into the underground storm sewer system.
- The southern on-site storm sewer network is sized to convey minor (1:5 years) and major (1:100 years) flows. The sub-catchment area (Appendix X) is divided into two portions, the northern half of rooftops plus northern driveway/parking lots (Catchment A) is considered to be discharging to the municipality's storm sewer system and directs to the pond while the southern half of the rooftops plus southern driveway/parking lots (Catchment B1) discharge to the holding tank through catchbasins and storm sewer system, located along the southern

driveway. This portion shall be controlled with an outlet control device (orifice) to restrict the stormwater flowing into the sewer system and then directed to the wet pond with a pump.

- The major storm events from the northern portion of the development's contributing drainage area will be conveyed along the internal roadways to the stormwater block while the southern portions will be directed to the pond from the southwest corner of the parking.

See Appendix XIII for a detailed SWM flowchart and Appendix XV for tank details.

5. Water Quantity

5.1. Peak flow

Prior to development, the property was considered vacant (without building) but the majority of the site was agricultural land with a small portion of natural heritage containing trees and grassy lawn in the south corner (0.2ha), with a Total Impervious Surface Area of 0%. The property is grading 2 - 4% towards the southwest, with sandy clay-primary soil having low infiltration rates.

Based on the Township's requirement related to the subdivision SWM report, the discharge from the development should be restricted to the allowable discharge rate, which is equivalent to the site's peak runoff rate using a runoff coefficient of 0.76 during the 2-year to 100-year design storm events. The proposed flow control measures comprise using the existing SWM pond located downstream of the site, in the west block. As discussed above, it is required to retain the site with only 0.11 (11%) additional runoff due to the new impervious area and related runoff coefficient increase. The proposed storage tank is an artificial structure designed to temporarily hold stormwater runoff.

Post-development has a TIMP = 94.3% (Catchment A & B1) and combined with the proposed construction of two holding tanks, there does not expect to be any changes in peak runoff flow rates or total volumes. With the proposed buried storage tanks for partially captured runoff from the southern half of rooftops plus southern parking lots and driveway, total peak flow control is achieved through precipitation volume storage using tank storage. The Commercial/Industrial SWM pond is already sized to receive peak flow from a runoff of 76% subdivision agreement, similar to the pre-development mode.

Stormwater quantity control will be provided on-site via a series of SWM holding tank storage and surface storage in proposed parking areas. An engineered (primary orifice at the bottom + overflow orifice at the top) outlet has been designed at the outlet of the storage system to control peak flows from the site to pre-development levels for the 1:2-year through 1:100-year design storms. The tank storage system with a footprint of approximately 37.5 m^2 will provide 93.7 m^3 of total volume capacity @ 2.5m depth (below overflow orifice) with an additional 9.5 m^3 capacity (equal to 0.25 depth) above the emergency outlet, plus an additional 9.5 m^3 of pipe storage, which is sufficient to provide quantity control for the 1:2-year through 1:25-year design storms (93.7+9.5+9.5 = 112.7 m^3).

Additional storage (140 m^3) is also provided in the parking lot areas (above-ground storage) to achieve additional quantity control for the 1:100-year design storms or other less frequent storm events when the water level is at the emergency outlet elevation (Appendix VIII). Calculations in Appendix VIII demonstrate that 145.2 m^3 is required to control the 100-year storm event to pre-development values while the proposed tanks along with ponding storage on entire southern parking areas plus pipe storage provide a total of 243 m^3 quantity storage.

The proposed primary outlet from the tank consists of a 250 mm diameter orifice which directs to a manhole and then discharges into the sewer system through a 450 mm pipe and pump. In the event of an obstruction to the primary outlet, an overflow orifice (400 mm dia.) has been included in the outlet structure @ 262.8 m.

Post-development peak flows for each catchment were calculated above using the Modified Rational Method (Table 6). The calculations are included in Appendix VIII. Detailed stage-storage-discharge calculations are included in Appendix VIII for reference.

5.2. Volume control

Based on LSRCA SWM guidelines, any works that meet the major development definition outlined in the Lake Simcoe Protection Plan or that results in site disturbance that creates 0.5 hectares of new impervious surface or fully reconstructs 0.5 hectares or more of impervious surface, should meet the volume control requirements from a 25mm rainfall event from the total impervious area.

In Table 8 the total runoff volume control target for the post-development scenario (25mm storm event) and the volume capacity of the proposed tanks 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 study site is 230m^3, which is less than the provided storage capacity within the proposed storage tanks/pipe storage (103 m^3) plus ponding storage (140 m^3). Given the presence of a downstream pond, the gradual discharge of this volume into the pond, followed by its infiltration into the ground, will ensure compliance with the volume control criterion.

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

Parameter	Building	Parking lot	Walkway	Driveway	Total
TIMP Area (m^2)	3118	1108.5	136.2	4818.8	9181.5
25 mm runoff volume control (m^3)	78	27.7	3.4	120.5	230.0

- **Active volume capacity through storage tank @ max. depth of 2.5m ~ 93.7m^3 plus ponding/parking storage ~ 140 m^3 plus pipe storage ~ 9.5 m^3 are equal to 243 m^3 versus a site-wide volume control target of 230m^3. It should be noted that the downstream pond is designed to control the runoff volume generated by an impermeability percentage of 78.6%.**

- **The additional volume control target due to an increase of the total impervious area (Timp) from 78.6% (subdivision study) to 94.3% (proposed) or runoff coefficient increase from 0.76 to 0.86, for 1530 m^2 impervious area, will be equal to 38.3 m^3 which is much less than the capacity of the proposed tanks (243 m^3).**

Assuming a conservative approach, according to Section 3.2.6 of the LSRCA SWM Guidelines (2022) and considering the high groundwater level constraint at the site, it is feasible to retain runoff from a 12.5

mm event from all impervious surfaces or at least 5 mm using Alternatives 1 and 2, rather than controlling the full 25 mm volume. As a result, the required controlled runoff volume is reduced to 115 m^3 or 46 m^3, respectively, instead of 230 m^3 for the entire site area. Given the presence of the existing wet pond, the necessary volume control will be effectively achieved.

5.3. Major-minor system conveyance

This site is classified as a low infiltration area due to the presence of a thick layer of sandy clay soil in most of the developed areas along with a shallow groundwater table, and a 12.2mm/hr unfactored infiltration rate.

The proposed development shall control both minor (i.e., 8.71mm/hr precipitation in 2yr storm event) and major (i.e., 19.51mm/hr precipitation in 100yr storm event) flows to the downstream storm drainage capacity, i.e., Black River. The proposed subdivision is considered to be a commercial/industrial development with a calculated percentage of impervious area of 78.6 % for the area draining to the pond. This impervious area includes all anticipated hardened surfaces, such as driveways, roads, walkways, and rooftops.

The existing wet stormwater management pond located in the west block will collect both minor and major flows. The majority of the site will be graded in order to drain to the south portion of the site, except the northern driveway/parking lot and the northern half of the rooftops.

There is around 140 m^3 surface storage capacity within the southern parking lots, between curbs (250 mm @ surface area = 1123 m^2 in the lowest area at the south portion). The wet pond along with the storage tank plus parking ponding will provide the requisite peak flow attenuation in the form of post to pre-development peak flow matching for the development. When holding tank capacity is completed and water depth rises on the driveway to more than 250mm, the overflow is discharged to the wet pond through sheet flow.

A piped stormwater system (minor system) has been designed to convey storm events up to and including the 5-year storm to the stormwater management pond and storage tanks.

The major storm events from the development's contributing drainage area will be conveyed along the internal roadways to the stormwater pond. The paved surface will provide sufficient erosion protection. The internal overland flow route will discharge to a 62 m long, 2.0 m wide, 0.20 m deep trapezoidal inlet channel. The bottom width is 2.0 m wide, with 10-20% side slopes and a 3.3% slope to the stormwater pond. This section will convey the peak flow of 0.95 m^3/s at a depth of 0.19 m, and a velocity of 1.66 m/s. A turfstone-lined flow route will provide sufficient erosion protection. At the crossing of the northwest entrance with this channel, a new concrete box culvert is proposed with 0.3m Height, 1.1m Width at 2.2% slope, 13.5 meters long, which will provide more than 1.05 m^3/s flow capacity along with 50mm freeboard.

The emergency overflow weir is required to safely convey the 100-year post-development uncontrolled flows of 2.33 m^3/s for the entire subdivision. With a 10 m wide, 0.30 m deep trapezoidal overflow weir with 10% side slopes, the peak flow depth will provide for a depth of 0.24 m and a velocity of 0.80 m^3/s.

5.4. Regulatory storm conveyance

The site property is serviced by a storm sewer system which leads to a wet pond in the west block, and at the same time, a tributary of Harrison Creek is also flowing along Woodbine Ave. to the north which collects the runoff from an uncontrolled permeable area of this site. There is no stream or regulatory storm conveyance on the site property, with neighbors to the north, south, and east being higher than the proposed Site plan.

In general, the existing creek between Woodbine Ave. and the wet pond area is not disturbed, and stormwater from the subject property DOES NOT enter into this creek or ditch, except a small portion of the southern property boundary (natural heritage and grassy lawn, Catchments B2-B3), which has a natural slope towards the creek, same as the pre-development condition.

6. Water Quality

6.1. Total Suspended solids

Stormwater quality controls will be required to implement in accordance with requirements of the Lake Simcoe Region Conservation Authority (LSRCA) with a targeted Total Suspended Solids (TSS) removal of 80% in accordance with provincial policy, while an infiltration facility can achieve enhanced 80% S.S. Removal by a wet pond with a range of 250m^3/Ha for 85% levels of imperviousness (MOE 2003 SWM PDM Table 3.2). With TIMP equal to 94.3% for controlled catchment, required storage volumes may be obtained by <u>extrapolating the values provided in Table 3.2, which is about 255.5m^3/Ha for wet ponds.</u> 80% TSS removal can be achieved through the use of a forebay, or through an integrated treatment train approach including LID practices (Wet Pond with forebay). In this project, the minimum storage volume required is 248m^3 (wet pond), for a 0.97 ha development in the controlled catchment area while the proposed site property has a total storage volume of more than 3050 m^3 with a proposed SWM wet pond for the entire subdivision with a total of 5.26 ha development. A sediment forebay is also designed at the SWM inlet to settle out the majority of the sediment load within an area that can be conveniently accessed for maintenance.

Thus, the efficiency would be:

Initial TSS Load = 1.0
TSS Load Removed by Wet Pond = 1.0 X 80% Removal Rate = 0.80
Final TSS Load Downstream of Wet Pond which drained to downstream creek = 1.0 – 0.80 = 0.20
TSS Removal Rate by Wet Pond = 1.0 – 0.20 = 0.8 or 80%

<u>The existing SWM wet pond will treat the post-development flows to the required MOE quality standard, with a TSS removal rate of approximately 80%.</u>

6.2. Total Phosphorus Budget

The high phosphorus levels in Lake Simcoe have led to excessive growth of plants and algae. In this regard, LSRCA policies require the removal of 80% of the annual total phosphorus (TP) load from impervious areas of Major Development projects. Thus, this project with a 0.92ha new TIMP and 3118 m^2 new building is classified as a major development, and TP budget evaluation is required.

Furthermore, in addition to the above-noted requirements, the removal of 100% (zero export target) of the annual TP load from **all new or redevelopment** as per the Phosphorus Offsetting Policy (P.O.P.) is required.

The proposed development site has been modeled using the MECP Hutchinson Tool (Appendix IX) for the Lake Simcoe Watershed, developed by Hutchinson Environmental Sciences Ltd., Greenland International Consulting Ltd., and Stoneleigh Associates Inc.

MECP Hutchinson Tool was modeled for the following scenarios:
1. Pre-development
2. Post-development without mitigation
3. Post-development with mitigation (using SWM pond)

Pre-development with TIMP = 0%, outgoing TP of 0.26kg/year.
Post-development with TIMP = 78.1%, outgoing TP of 1.71kg/year.
Post-development with TIMP = 78.1% using LID's, outgoing TP of 0.66kg/year in which 0.63 kg discharged by controlled catchment area and 0.03 kg by uncontrolled catchment area.

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

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

100 x [(Controlled – Existing)/Existing]
100 x [(0.66 – 0.26)/0.26]
100 x [1.54]
154% increase in Total Phosphorus concentrations versus existing condition

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

100 x [(Controlled – Uncontrolled)/Uncontrolled].
100 x [(0.66 – 1.71)/1.71].
100 x [– 0.61].
61% reduction in Total Phosphorus concentrations versus Post-development

Without any mitigation, average annual runoffs would release a total of 1.71kg 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 0.63kg plus the uncontrolled catchment's phosphorus, i.e., 0.03kg. In summary, there is a 154% increase in total phosphorus discharge when comparing post-development with mitigation and pre-development (existing) scenarios while 61% reduction in total phosphorus discharge when comparing post-development with mitigation measures and post-development scenarios using the MECP Hutchinson Tool.

To minimize the amount of phosphorus discharged from the site, a treatment train approach is to be utilized. The northern half of rooftops plus northern paved area runoffs will be conveyed to the pond directly through the sewer system, which will provide phosphorous reduction. Moreover, runoff captured by the southern half of the rooftops plus the southern parking lot and driveway will be conveyed to the storage tanks below the parking lot on the south portion, which has been discharged to the SWM Pond at the end. Ultimately, all storm runoffs will be conveyed to an SWM wet Pond, which will further reduce phosphorous levels.

According to the LSRCA Standards, the typical phosphorus reduction is 63% for a wet pond.

Detailed calculations can be found in Appendix XII.

The following Table 9 details the anticipated phosphorous loadings for the pre and post-development conditions.

Table 9- Phosphorus Loadings

	Total P (kg/year)
Pre-Development	0.26
Uncontrolled Post-Development	1.71
Controlled Post-Development	0.66

When the effects of the wet pond treatment are considered, the actual reduction rate of TP is equal to 61%. The proposed development will incur a phosphorus load of 0.66 kg/year which is more than double the current phosphorus loading of 0.26 kg/year while the LSRCA requires zero export target of the annual TP load from all new or redevelopment as per the Phosphorus Offsetting Policy.

The LSRCA implemented a Phosphorous Offsetting Policy in May 2023 which has a goal that all new development must reduce 100% of the phosphorous leaving the property. A fee of $89,425 per annual kg is required for anything above a net 0 kg of phosphorous running off the site. Therefore, the required fee for the proposed development is as follows:

LSPOP Fee = $35,770 x 2.5 x 0.40 kg = $35,770 + 15% Administration Fee

6.3. Winter salt

Post-development winter salt use is anticipated to be higher than pre-development. This is primarily a concern for the driveways. Design practices that help reduce winter salt use include:

- Minimal trees are considered for landscaping near driveways, to reduce winter shading
- Recommend the client to use sand & gravel for traction instead of salt
- Special consideration should also be given to the color selection of the pavement. Dark-colored pavers will increase the absorption of solar radiation and lead to higher ice-melting potential. For additional information on permeable pavers and turf and grass block pavers refer to CVC and TRCA, 2010.

7. Water Balance

Based on Black River Subwatershed Climate Data, precipitation is 895mm/year, and averaged evapotranspiration rate is 652mm/year which is mostly intensive agriculture in hydrologic soil group D (Appendix V).

Pre-development site conditions have low groundwater infiltration/recharge, as the site is predominantly sandy clay soil with slopes <2 - 4% grade and therefore a small portion of the runoff flows into the downstream creek through the sheet flow. On a 1.02Ha with 0% TIMP, there is an estimated 1484m^3 of groundwater recharge per year. Runoff is 97mm/year= 989 m^3/yr.

As seen in Table 10 below, pre-development conditions are discussed in detail in Section 3. The site is within the Black River Subwatershed, with Sandy Clay Hydrologic Soil Group D, with 0% TIMP.

The proposed post-development TIMP = 0.92Ha / 94.3%, and without any LID treatments, the site recharge is estimated at an equivalent of 18mm/year for the site, or 179m^3 (88% decrease as compared to pre-development).

Post-development, TIMP = 0.92Ha / 94.3%, with all of the impermeable surfaces from a reduction of agriculture or grass fields. This causes a reduction of evapotranspiration (-78.7%), while all of the precipitation within the TIMP area is managed for 100% recharge and infiltration (+410.5%). The results show that the amount of external outflow has also decreased compared to the pre-development state (97mm to 12mm).

Post-development with mitigation, the total rate of infiltration has increased from 146 to 744mm/year or 1484m^3 to 7574m^3 (+410.5%). Detailed calculations of water balance for each scenario are presented in Appendix XI.

Table 10 –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)	9,109	9,109	0.0%	9,109	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)	**9,109**	9,109	0.0%	9,109	0.0%
Outputs (Volumes)					
Precipitation Surplus (m^3/yr)	2,473	7,693	211.1%	7,693	211.1%
Net Surplus (m^3/yr)	2,473	7,693	211.1%	7,693	211.1%
Evapotranspiratin (m^3/yr)	6,635	1,415	-78.7%	1,415	-78.7%
Infiltration (m^3/yr)	1,484	179	-88.0%	5,063	241.2%
Rooftop Infiltration (m^3/yr)	0	0	0.0%	2,512	0.0%
Total Infiltration (m^3/yr)	1,484	179	-88.0%	7,574	410.5%
Runoff Pervious Area (m^3/yr)	989	119	-88.0%	119	-88.0%
Runoff Impervious Area (m^3/yr)	0	7,393	0.0%	0	0.0%
Total Runoff (m^3/yr)	989	7,512	659.4%	119	-88.0%
Total Outputs (m^3/yr)	9,109	9,109	0.0%	9,109	0.0%

8. Erosion and Sediment Control

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

1. Minimizing soil erosion at the source;
2. Containing sediment on site;
3. Treat sediment-laden water in a location away from the work area.
4. Being proactive, not reactive.

During construction, there is potential for short-term sediment wash-off from the site. To protect the downstream receiving watercourse, on-site erosion and sediment control (ESC) measures are necessary during construction and it will be submitted after plan approval. The ESC measures focus on minimizing adverse environmental impacts by restricting the mobilization and transport of sediment, the following general practices are recommended:

1.1. Minimizing soil erosion at the source:
Based on the proposed developments and the shallow seasonal groundwater, the main area of concern for ESC is the construction of a septic bed and installation of storage tanks in the southern portion, near the natural heritage which has the lower topography; and foundation works as well. All temporary grading will have slopes directed into the excavated area.

1.2. Containing sediment on site:

Where sediments are not directed into the excavation area, it is a requirement to install Silt Fences as per LSRCA design charts at the site boundaries.

1.3. Treat sediment-laden water in a location away from the work area:
The proposed cut and fill plan requires careful management to prevent sediment-laden storm runoff from being conveyed off-site and into the downstream pond or creek. Based on the low-infiltration rate of the site property (sandy soil & infiltration rate = 49 min/cm), it is not a requirement for the site to produce any runoff discharge locations and silt fences are adequately installed and maintained, any temporary ponding of sediment-laden runoff will be quickly infiltrated into the natural soils.

1.4. Being proactive, not reactive
It is acknowledged that the project proposes cut and fill operations. Topsoil stripping of the fill location must be prepared prior to the actual cut operations. This would allow the fill portion to occur as soon as feasible, with minimal stockpiling. It is also a requirement to seed the backfilled area as soon as possible.

This project can be separated into three (3) separate phases as follows:

Phase 1 – Site Preparations: Proposed driveway construction and the preparation for future cut & fill operations require topsoil stripping. Actual heavy-duty silt fences are also installed in this phase. All topsoil shall be stockpiled at the southeastern corner of the driveway, behind the silt fences, to act as a loose permeable filtration mound.

Phase 2 –Cut & Fill Operations, Construction & Backfill: Once site preparation is completed, the actual cut & fill operation for the building, storage tank, and septic can begin. It is a requirement that all excavated materials (sandy clay soils) be used 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 along the eastern property boundary, 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.

When initial excavation is completed, the concrete foundation and walls will be formed and poured. The proposed buildings will be constructed in this phase. The storage tanks are also installed in this phase. Septic holding tanks are also 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, topsoil-covered, and reseeded as soon as possible.

Phase 3 – Final Grading, Paving & Auxiliary Structures: After the overall backfill is completed, the physical construction may be completed, as well as keeping a very small amount of soil for final grading. Auxiliary structures will be constructed at this time, like paving the driveway, planting, and landscaping. Previously stockpiled topsoil would be mechanically processed and then covered on all exposed soils.

Routine inspection and maintenance of the ESC measures are required to ensure these measures function properly and effectively. Details of Erosion and Sediment Control Measures and Cut & Fill Details are provided in a separate ESC plan report.

9. Reliance & Signature

This report is the intellectual property of King EPCM, and has been prepared for the sole use of Jonathan Benczkowski (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 Town of East Gwillimbury, Lake Simcoe Region Conservation Authority (LSRCA), and Regional Municipality of York (York Region) in regards to the Client's rural industrial development project at 25 Pagewood Crt., Town of East Gwillimbury, ON.

Respectfully,

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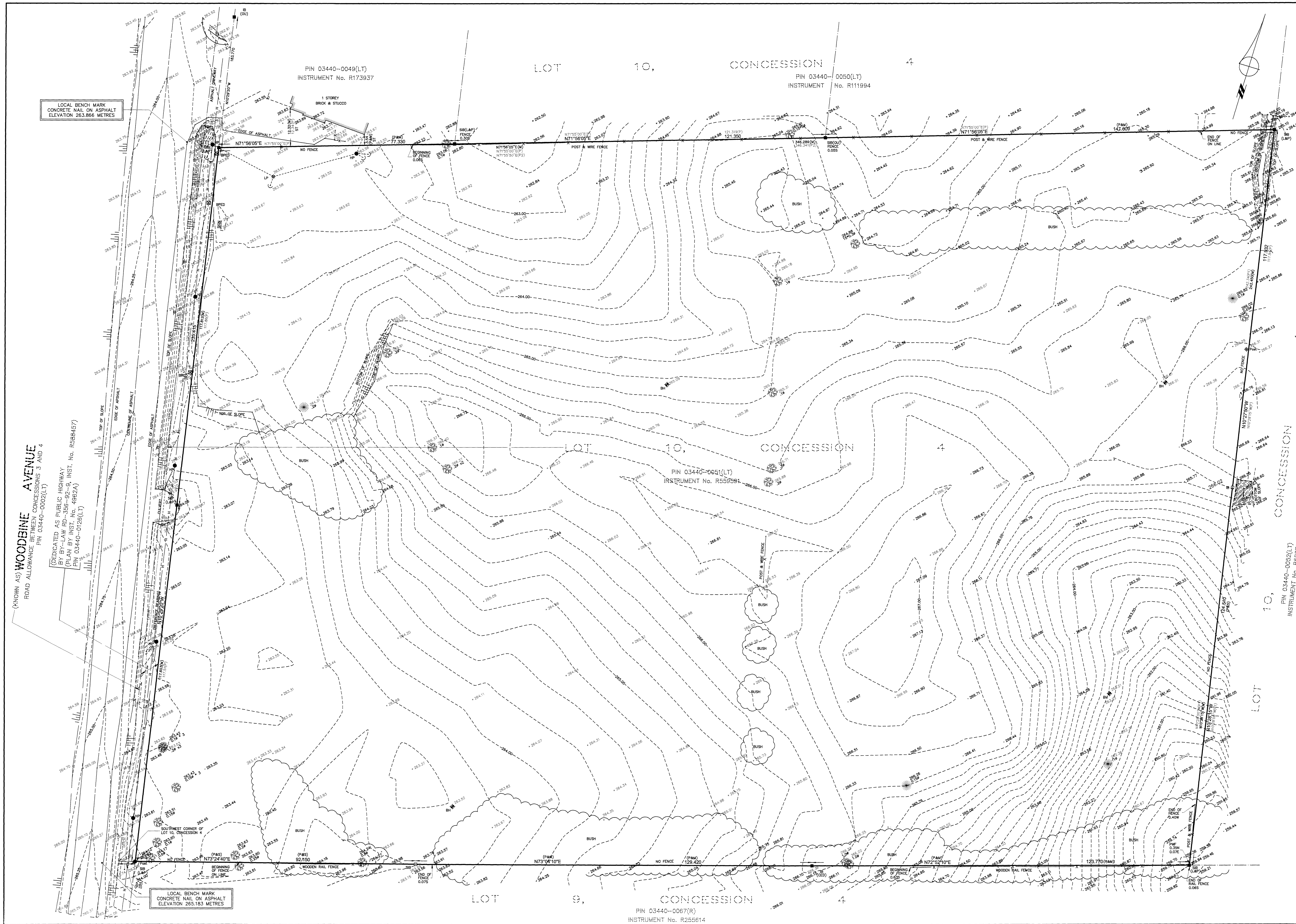
Wisconsin Department of Natural Resources. (2004). Conservation Practice Standards. Site Evaluation for Stormwater Infiltration (1002). Madison, WI.

APPENDIX I – SITE SURVEY PLAN



BOUNDARY AND TOPOGRAPHIC SURVEY OF

PART OF LOT 10 CONCESSION 4 TOWN OF EAST GWILLIMBURY

REGIONAL MUNICIPALITY OF YORK

SCALE 1:500

10 0 10 20 30 40m

KRCMAR SURVEYORS LTD.

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

BEARING

BEARINGS SHOWN HEREON ARE ASTRONOMIC AND ARE REFERRED TO THE EASTERLY LIMIT OF ROAD ALLOWANCE BETWEEN CONCESSIONS 3 AND 4 AS WIDENED BY INST. 4962A HAVING A BEARING OF N10°29'20"W

ELEVATION

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48-100 HAVING AN ELEVATION OF 271.230 METRES.

LEGEND

■	DENOTES SURVEY MONUMENT FOUND
□	DENOTES SURVEY MONUMENT PLANTED
SIB	DENOTES STANDARD IRON BAR
SSIB	DENOTES SHORT STANDARD IRON BAR
IB	DENOTES IRON BAR
(M)	DENOTES MEASURED
(S)	DENOTES SET
(OU)	DENOTES ORIGIN UNKNOWN
(WIT)	DENOTES WITNESS
(P)	DENOTES PLAN OF SURVEY BY C.E. DOTTERILL LTD., O.L.S. DATED SEPTEMBER 6, 1988
(P1)	DENOTES SURVEYORS REAL PROPERTY REPORT BY E.R. GARDEN LIMITED, O.L.S. DATED OCTOBER 10, 2000
(P2)	DENOTES PLAN OF SURVEY BY LLOYD & PURCELL, O.L.S. DATED ON JUNE 10, 1964 (ATTACHED TO INST. No. R111994)
(P3)	DENOTES LAND PLAN BY MARSHALL, MACKLIN & MONAGHAN, O.L.S. BY INST. No. 4962A, DATED AUGUST 3, 1956
(L&P)	DENOTES LLOYD & PURCELL, O.L.S.
(CED)	DENOTES C.E. DOTTERILL, O.L.S.
(1370)	DENOTES KRCMAR SURVEYORS LTD. O.L.S.
PWF	DENOTES POST & WIRE FENCE
ST	DENOTES TIES TO STUCCO
123.45	DENOTES EXISTING GRADE ELEVATION
BH	DENOTES BOREHOLE
0.1ø	DENOTES CONIFEROUS TREE WITH TRUNK DIAMETER
0.1ø	DENOTES DECIDUOUS TREE WITH TRUNK DIAMETER
	DENOTES DOWN GUY ANCHOR
HP	DENOTES HYDRO POLE

SITE AREA = 8.2118 ha

ASSOCIATION OF ONTARIO LAND SURVEYORS
PLAN SUBMISSION FORM
1730719

THIS PLAN IS NOT VALID UNLESS IT IS AN EMBOSSED ORIGINAL COPY ISSUED BY THE SURVEYOR
In accordance with Regulation 1026, Section 29(3).

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. THE SURVEY WAS COMPLETED ON THE 13th DAY OF JULY, 2009

DATE JULY 21, 2009

ROBERT WIEGENBROKER
ONTARIO LAND SURVEYOR

FIELD:	S.M.	DRAWN:	S.D./K.B.	CHECKED:	R.W.	JOB NO:	09-080
DWG NAME:	09-080BT01	PLOT INFO:	07:51 20/Jul/2009	WORK ORDER NO:	10158		

1137 Centre Street Thornhill ON L4J 3M6 905.738.0053 F 905.738.9221 www.krcmar.ca

KRCMAR

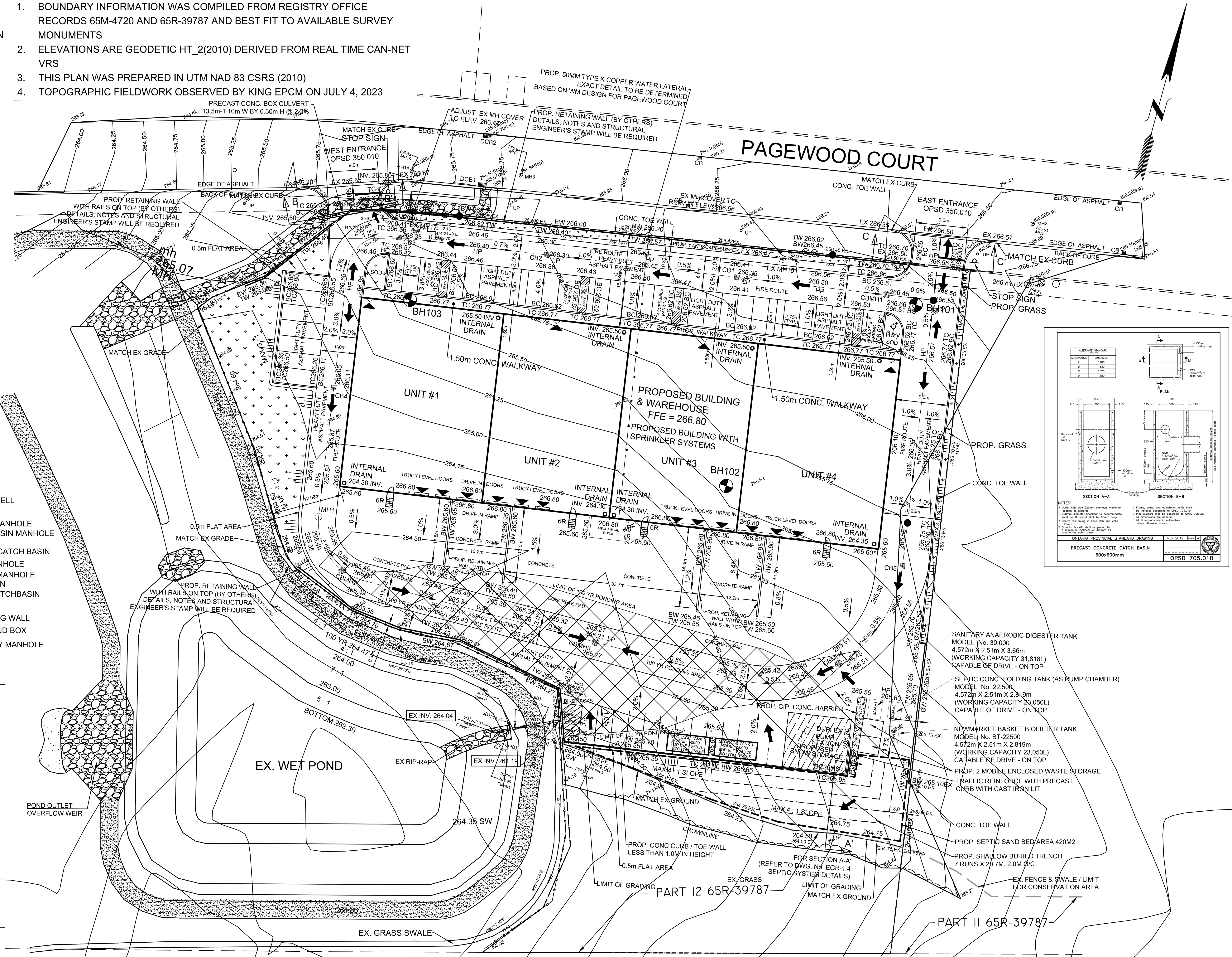
APPENDIX II – SITE GRADING PLAN & SWM PLAN

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. CONTRACTOR SHALL CHECK AND VERIFY ALL GIVEN GRADE ELEVATIONS AND DRAINAGE PRIOR TO CONSTRUCTION
5. DRIVEWAY CONSTRUCTION
   COMMERCIAL - LIGHT DUTY:
   - ASPHALT: 40mm HL3 SURFACE COURSE & 50mm HL8 BASE COURSE
   - GRANULAR: 150mm GRANULAR "A" & 300mm GRANULAR "B"
   - GRANULAR SHALL BE 98% COMPACTION
   - PAVEMENT MATERIAL TO BE REFERRED BY BELOW TABLE
   - HEAVY DUTY FIRE ROUTE:
   - ASPHALT: 50mm HL3 SURFACE COURSE & 100mm HL8 BASE COURSE
   - GRANULAR: 150mm GRANULAR "A" & 450mm GRANULAR "B"
   - GRANULAR SHALL BE 98% COMPACTION
   PAVEMENT MATERIAL TO BE REFERRED BY BELOW TABLE
6. SETBACK
   - FRONT SETBACK: 16.48M
   - BACK SETBACK: 71.80M
   - WEST SETBACK: 12.56M
   - EAST SETBACK: 10.26M

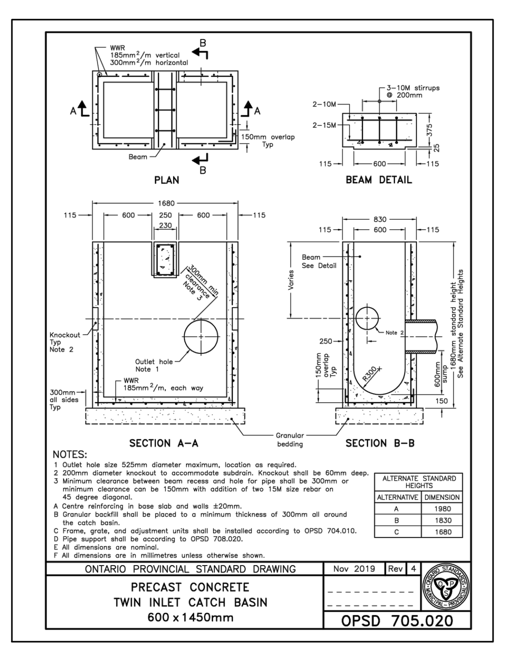
TOPOGRAPHIC SURVEY

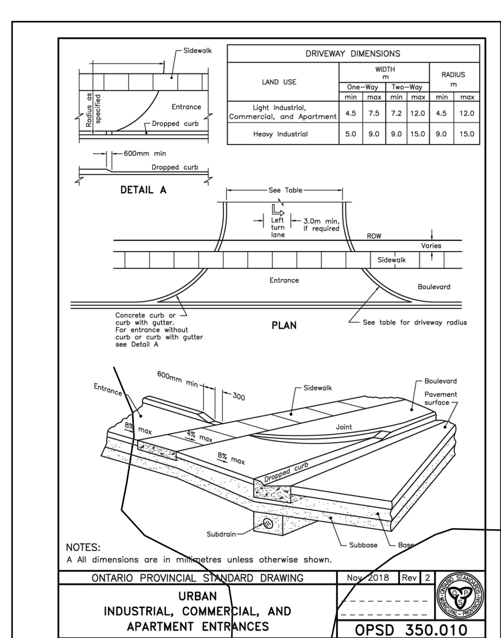
1. BOUNDARY INFORMATION WAS COMPILED FROM REGISTRY OFFICE RECORDS 65M-4720 AND 65R-39787 AND BEST FIT TO AVAILABLE SURVEY MONUMENTS
2. ELEVATIONS ARE GEODETIC HT_2(2010) DERIVED FROM REAL TIME CAN-NET VRS
3. THIS PLAN WAS PREPARED IN UTM NAD 83 CSRS (2010)
4. TOPOGRAPHIC FIELDWORK OBSERVED BY KING EPCM ON JULY 4, 2023



LEGEND:

.266.50 EX.	EXISTING ELEVATION
.266.90	PROPOSED ELEVATION
	EXISTING SURFACE FLOW ROUTE
	PROPOSED SURFACE FLOW ROUTE
	ROOF WATER INTERNAL DOWNSPOUT
.TW 315.58	TOP OF PROPOSED RETAINING WALL
.BW 315.58	BOTTOM OF PROPOSED RETAINING WALL
.TC 315.58	TOP OF PROPOSED CURB
.BC 315.58	BOTTOM OF PROPOSED CURB
	PROPOSED CONCRETE CURB
	PROPOSED PROTECTIVE BOLLARD
	100 YEAR PONDING AREA
FH	FIRE HYDRANT
	BOREHOLE
	PROPOSED WATER WELL
HP	HIGH POINT
	PROPOSED STORM MANHOLE
	PROPOSED CATCHBASIN MANHOLE CATCH BASIN
	PROPOSED DOUBLE CATCH BASIN
	EXISTING STORM MANHOLE
	EXISTING SANITARY MANHOLE
EX.CB.	EXISTING CATCHBASIN
EX.DCB.	EXISTING DOUBLE CATCHBASIN
	PROPERTY LINE
	PROPOSED RETAINING WALL
	PROPOSED VALVE AND BOX
	PROPOSED SANITARY MANHOLE





ib1(1) 263.89

ib(1) 264.04

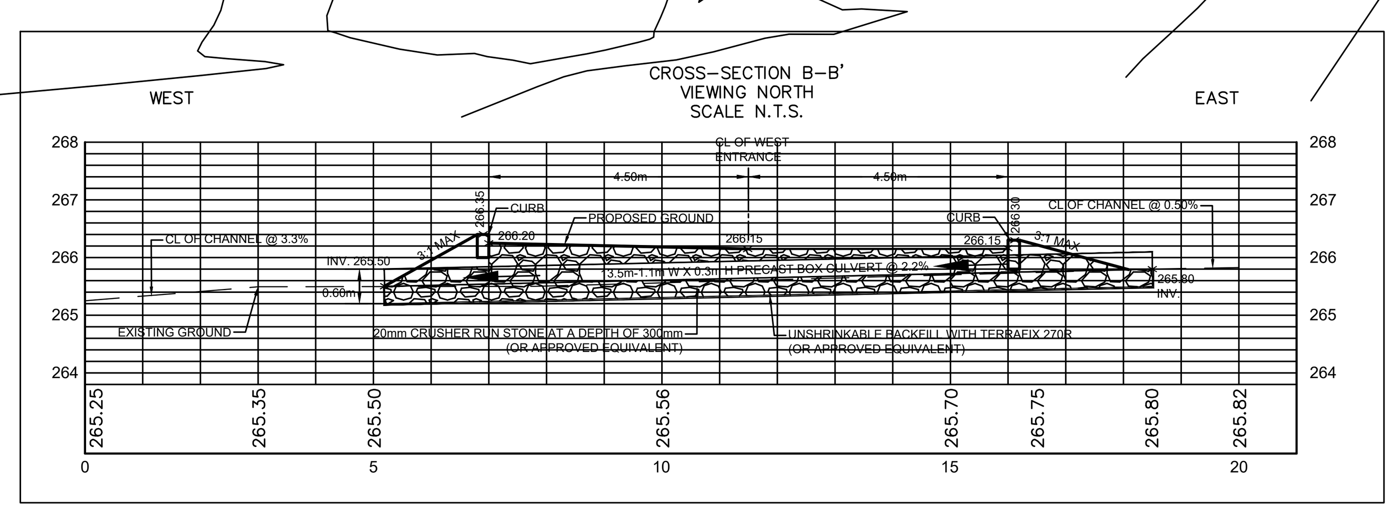
BENCH MARK

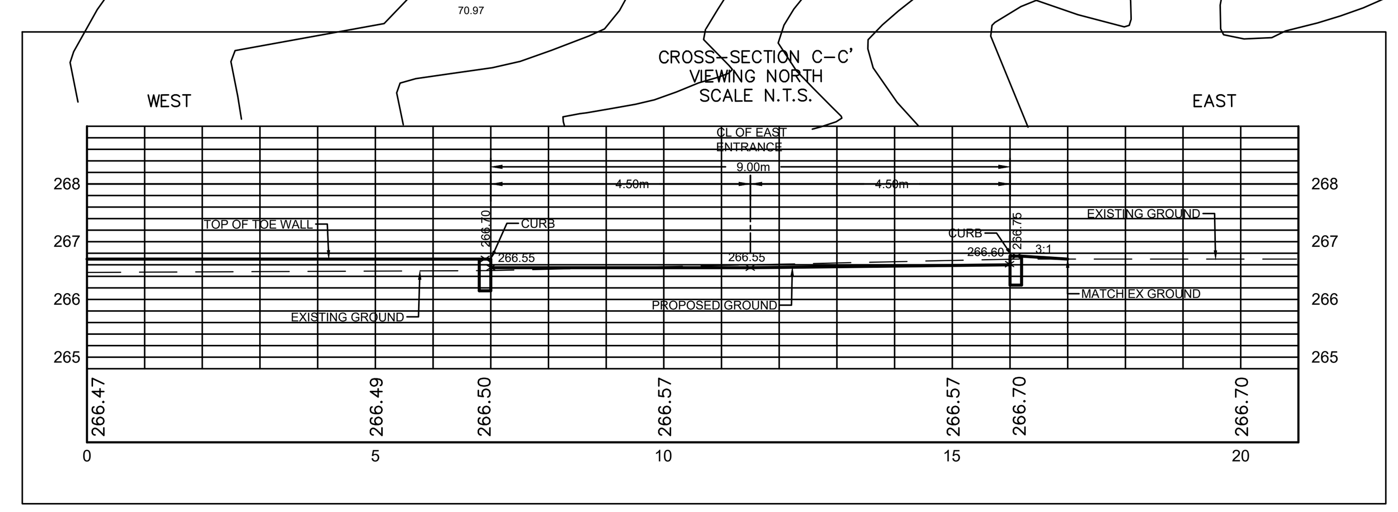
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48-100 HAVING AN ELEVATION OF 271.230 METRES.

SCALE 1:400

1cm = 4m

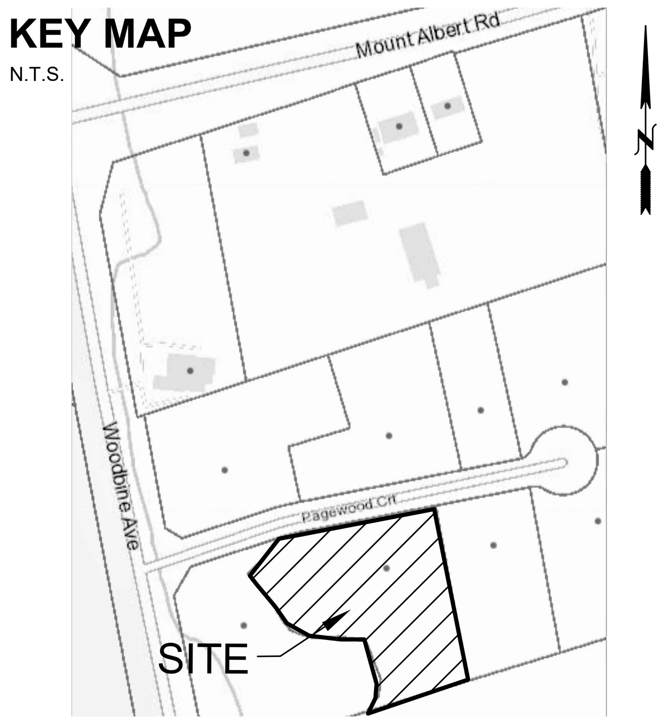
0m 20m 40m





KEY MAP

N.T.S.



DRAWN: TW

DATE: MAR 07, 2025

LICENSED PROFESSIONAL ENGINEER Y. T. WANG 100228476 PROVINCE OF ONTARIO 03/07/2025

KING EPCM

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

CLIENT: WELEAP INVESTMENTS LTD.

PROJECT NAME: CONSTRUCTION OF SELF-STORAGE WAREHOUSE UNITS

PROJECT LOCATION: 25 PAGEWOOD COURT EAST GWILLIMBURY

PRINT TITLE: GRADING & DRAINAGE PLAN

FILE No.: EGR - 1.1

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V5	ISSUED FOR PERMITS	DEC 07, 2023	KL	TW
V15	ISSUED FOR PERMITS	OCT 10, 2024	KL	TW
V17	ISSUED FOR PERMITS	MAR 07, 2025	TW	TW

STORMWATER MANAGEMENT

1. TWO STORMWATER SEWER PIPE SYSTEMS ARE PREPARED FOR COLLECTING, CONVEYING AND TREATMENT OF THE RUNOFF GENERATED ON THE SITE
2. PROPOSED BUILDING & WAREHOUSE ROOFTOP STORMWATER WILL FLOW TO CATCH BASINS AT NORTH SIDE OF THE SITE THROUGH INTERNAL DRAIN, COMBINED WITH RUNOFF FROM THE FRONT YARD WALKWAY, DRIVEWAY AND PARKING LOTS COLLECTED BY THE CATCH BASINS, AND THEN CONVEYED TO THE EXISTING WET POND ON THE WEST SIDE OF THE PROPERTY
3. RUNOFF FROM BACKYARD DRIVEWAY AND PARKING LOTS IS COLLECTED BY CATCH BASINS AT SOUTH SIDE OF THE SITE THROUGH PIPES AND CONVEYED TO INFILTRATION CHAMBER AT SOUTH SIDE OF THE SITE FOR TREATMENT AND INFILTRATION AND THEN OVERFLOW TO THE EXISTING WET POND ON THE WEST SIDE OF THE PROPERTY
4. CATCH BASIN

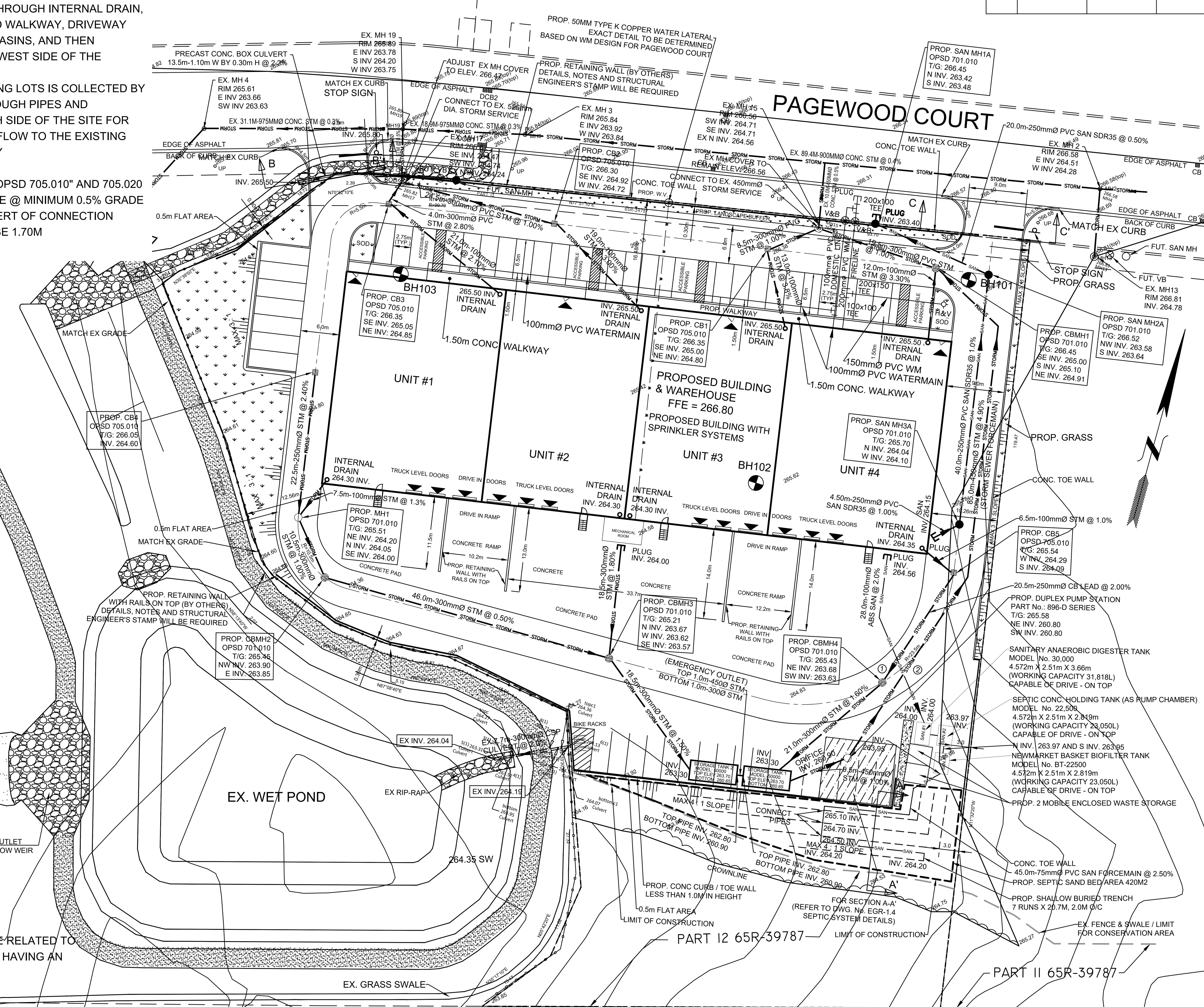
- CATCH BASIN SHALL BE CONSTRUCTED AS PER "OPSD 705.010" AND 705.020
- CONNECTION PIPE TO BE 12"/ 305MM DIA. PVC PIPE @ MINIMUM 0.5% GRADE
- MINIMUM COVER FROM FINISHED GRADE TO OBVERT OF CONNECTION PIPE SHALL BE 1.40M, TO INVERT OF PIPE SHALL BE 1.70M

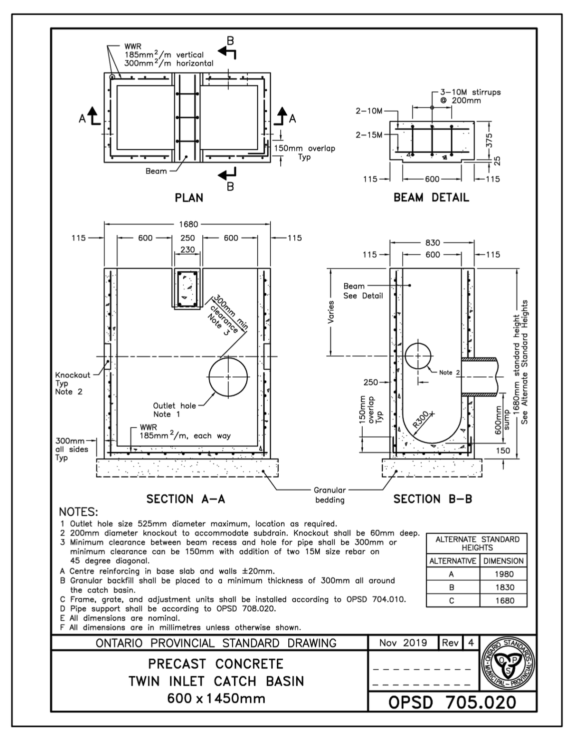
SERVICE PIPE CROSSING

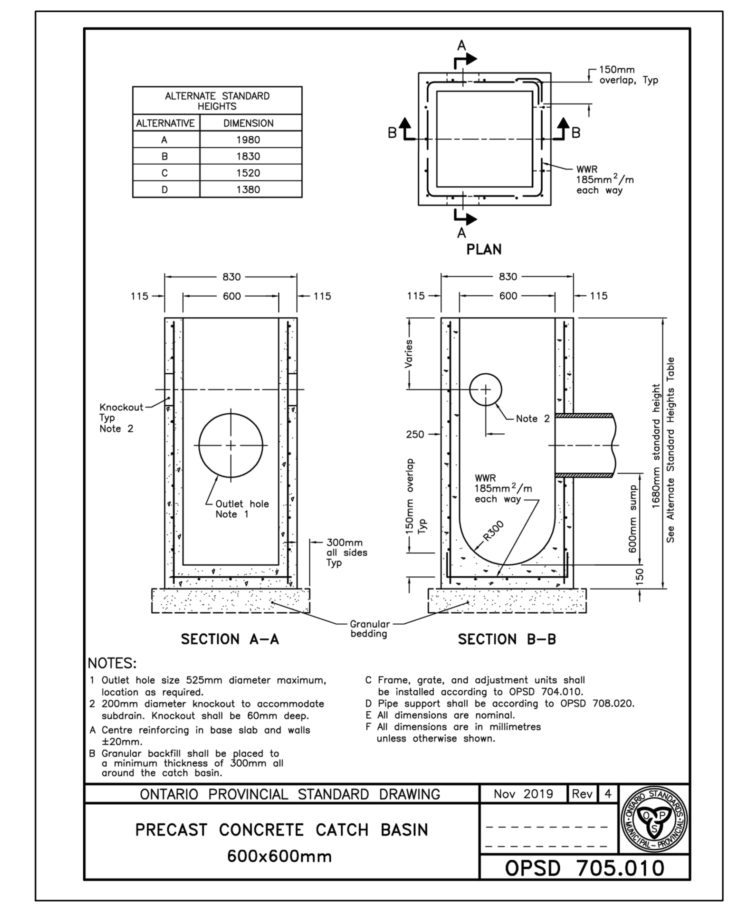
No.	STM	SAN	CLEARANCE
1	OBV. 264.03	INV. 264.43	0.40m
2	OBV. 261.98	INV. 264.38	2.40m

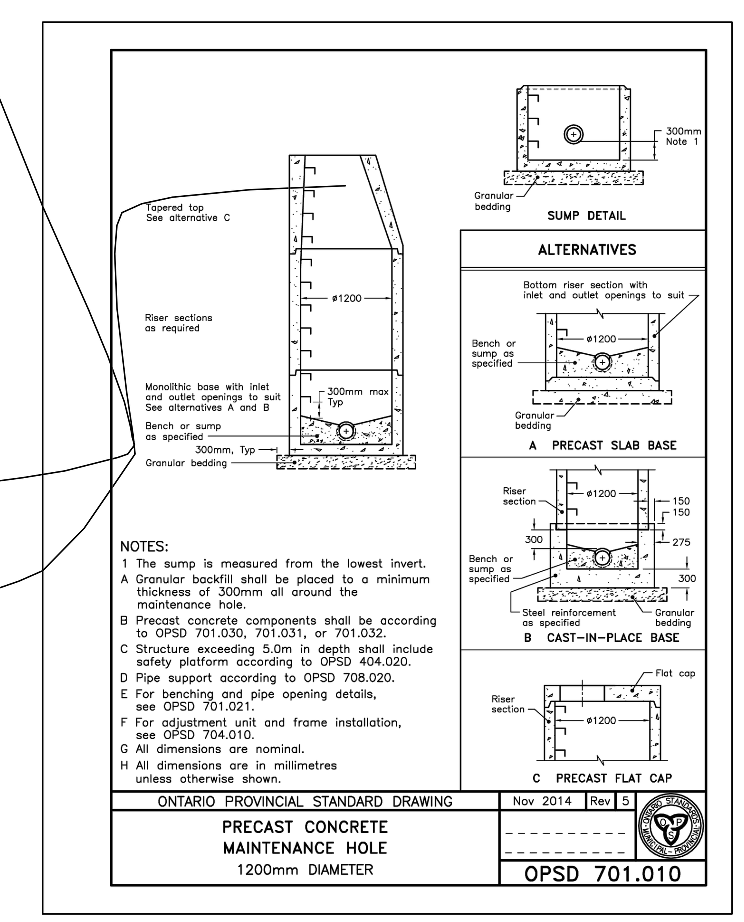
LEGEND:

- PROPOSED WATER WELL
- ROOF WATER DOWNSPOUT
- FIRE HYDRANT
- PROPOSED VALVE AND BOX
- BOREHOLE
- PROPOSED STORM MANHOLE
- PROPOSED SANITARY MANHOLE
- PROPOSED CATCHBASIN MANHOLE
- CATCH BASIN
- PROPOSED DOUBLE CATCH BASIN
- EXISTING STORM MANHOLE
- EXISTING SANITARY MANHOLE
- EXISTING CATCHBASIN
- EXISTING DOUBLE CATCHBASIN
- PROPOSED STORM SEWER
- PROPOSED SANITARY SEWER
- EXISTING STORM SEWER
- PROPERTY LINE
- PROPOSED RETAINING WALL
- 100 YEAR PONDING AREA



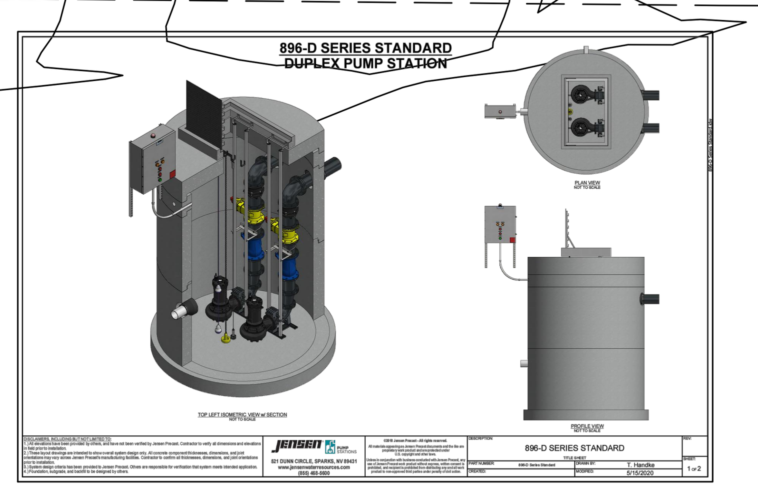


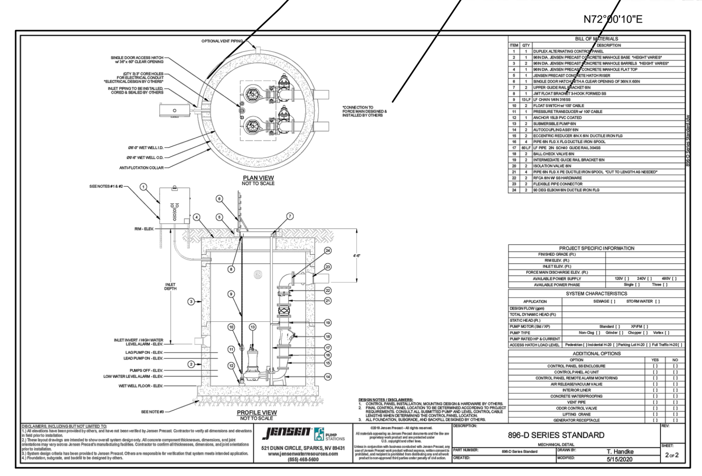


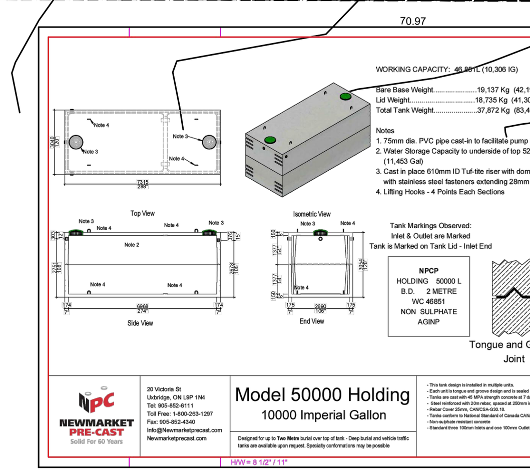


BENCH MARK
ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48-100 HAVING AN ELEVATION OF 271.230 METRES.

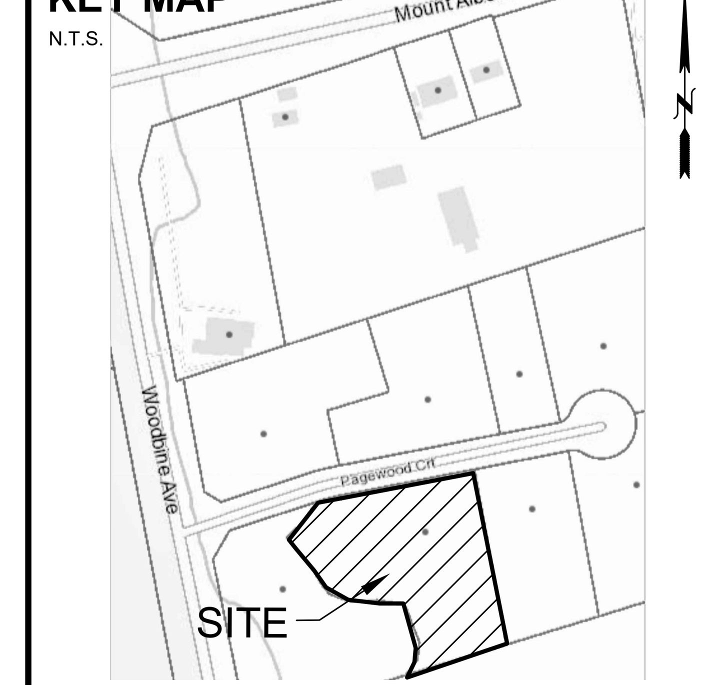
SCALE 1:400
1cm = 4m
0m 20m 40m







KEY MAP
N.T.S.



DRAWN
TW

DATE
MAR 07, 2025

03/07/2025

KING EPCM

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

CLIENT
WELEAP INVESTMENTS LTD.

PROJECT NAME
CONSTRUCTION OF SELF-STORAGE WAREHOUSE UNITS

PROJECT LOCATION
25 PAGEWOOD COURT
EAST GWILLIMBURY

PRINT TITLE
SITE SERVICING PLAN

FILE No.
EGR - 1.2

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V5	ISSUED FOR PERMITS	DEC 07, 2023	KL	TW
V15	ISSUED FOR PERMITS	OCT 10, 2024	KL	TW
V17	ISSUED FOR PERMITS	MAR 07, 2025	TW	TW

GENERAL NOTES:

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2. SEE "EROSION AND SEDIMENT CONTROL PLAN" FOR CONTROL INSTALLATION PRIOR TO CONSTRUCTION AND STOCKPILE MANAGEMENT
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4. CONTRACTOR SHALL CHECK AND VERIFY ALL GIVEN GRADE ELEVATIONS AND DRAINAGE PRIOR TO CONSTRUCTION
5. DRIVEWAY CONSTRUCTION
   COMMERCIAL:
   - ASPHALT: 40mm HL3 SURFACE COURSE & 50mm HL8 BASE COURSE
   - GRANULAR: 150mm GRANULAR "A" & 225mm GRANULAR "B"
   - GRANULAR SHALL BE 98% COMPACTION
   - PAVEMENT MATERIAL TO BE REFERRED BY BELOW TABLE
   - HEAVY DUTY FIRE ROUTE:
   - ASPHALT: 40mm HL3 SURFACE COURSE & 75mm HL8 BASE COURSE
   - GRANULAR: 150mm GRANULAR "A" & 300mm GRANULAR "B"
   - GRANULAR SHALL BE 98% COMPACTION
     PAVEMENT MATERIAL TO BE REFERRED BY BELOW TABLE
6. SETBACK
   - FRONT SETBACK: 16.48M
   - BACK SETBACK: 71.80M
   - WEST SETBACK: 12.56M
   - EAST SETBACK: 10.26M

TOPOGRAPHIC SURVEY

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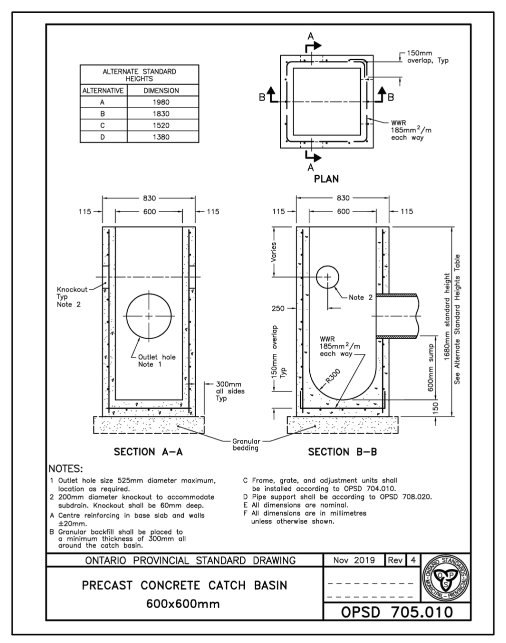
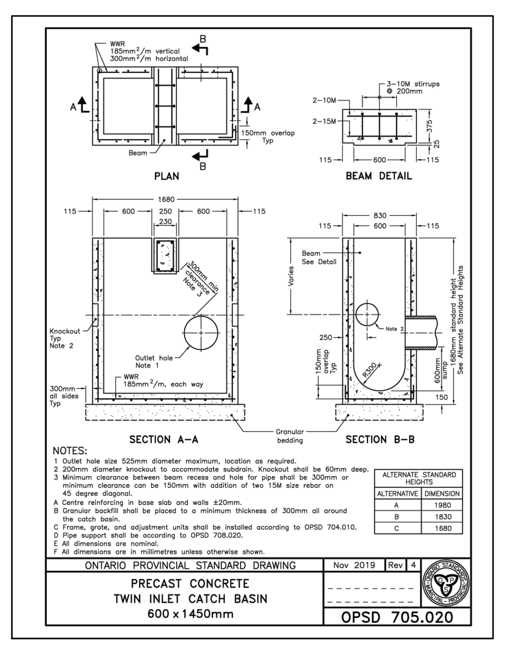
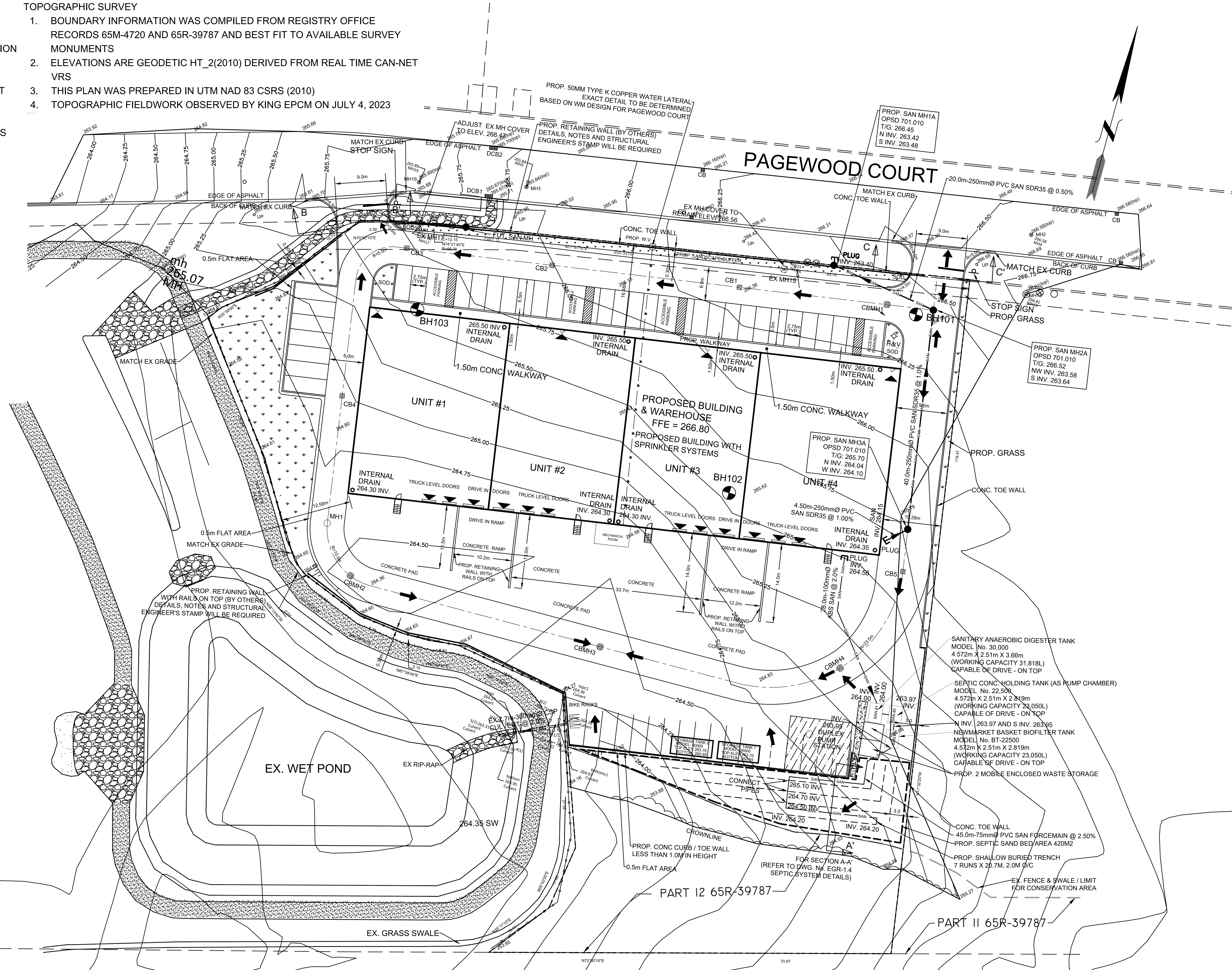
2. ELEVATIONS ARE GEODETIC HT_2(2010) DERIVED FROM REAL TIME CAN-NET VRS
3. THIS PLAN WAS PREPARED IN UTM NAD 83 CSRS (2010)
4. TOPOGRAPHIC FIELDWORK OBSERVED BY KING EPCM ON JULY 4, 2023



PRECAST CONCRETE CATCH BASIN 600x600mm — OPSD 705.010



PRECAST CONCRETE TWIN INLET CATCH BASIN 600x1450mm — OPSD 705.020



BENCH MARK

ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48-100 HAVING AN ELEVATION OF 271.230 METRES.

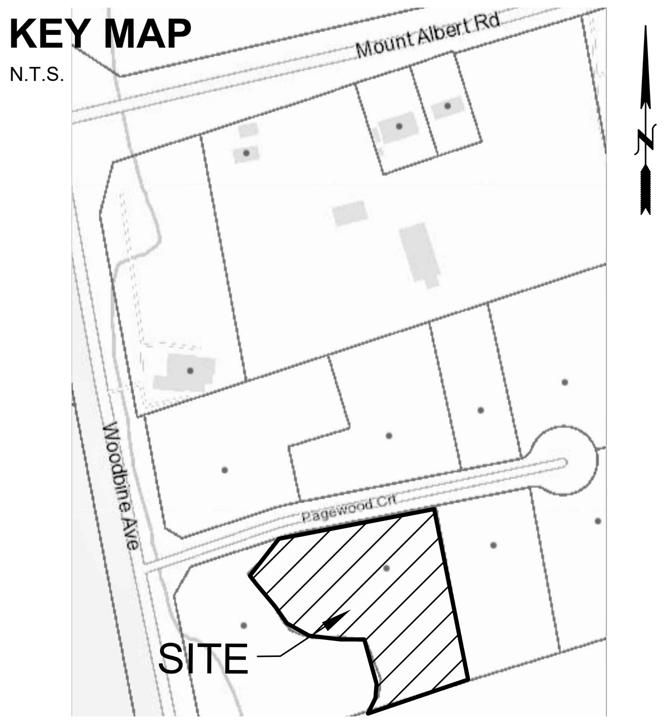
SCALE 1:400
1cm = 4m
0m 20m 40m

LEGEND:

266.50 EX.	EXISTING ELEVATION
266.90	PROPOSED ELEVATION
	EXISTING SURFACE FLOW ROUTE
	PROPOSED SURFACE FLOW ROUTE
	ROOF WATER INTERNAL DOWNSPOUT
TW 315.58	TOP OF PROPOSED RETAINING WALL
XB	BOTTOM OF PROPOSED RETAINING WALL
CP 315.58	TOP OF PROPOSED CURB
C 315.58	BOTTOM OF PROPOSED CURB
	PROPOSED CONCRETE CURB
*	PROPOSED PROTECTIVE BOLLARD
W	PROPOSED WATER WELL
HP	HIGH POINT
	PROPOSED STORM MANHOLE
	PROPOSED CATCHBASIN MANHOLE
	CATCH BASIN
	PROPOSED DOUBLE CATCH BASIN
	EXISTING STORM MANHOLE
	EXISTING SANITARY MANHOLE
EX.CB	EXISTING CATCHBASIN
EX.DCB	EXISTING DOUBLE CATCHBASIN
	PROPERTY LINE
	PROPOSED RETAINING WALL

KEY MAP

N.T.S.



DRAWN: TW

DATE: MAR 07, 2025

LICENSED PROFESSIONAL ENGINEER Y. T. KING 100228476 03/07/2025 PROVINCE OF ONTARIO

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647-459-5647
General@KingEPCM.com

CLIENT

WELEAP INVESTMENTS LTD.

PROJECT NAME

CONSTRUCTION OF SELF-STORAGE WAREHOUSE UNITS

PROJECT LOCATION

25 PAGEWOOD COURT
EAST GWILLIMBURY

PRINT TITLE

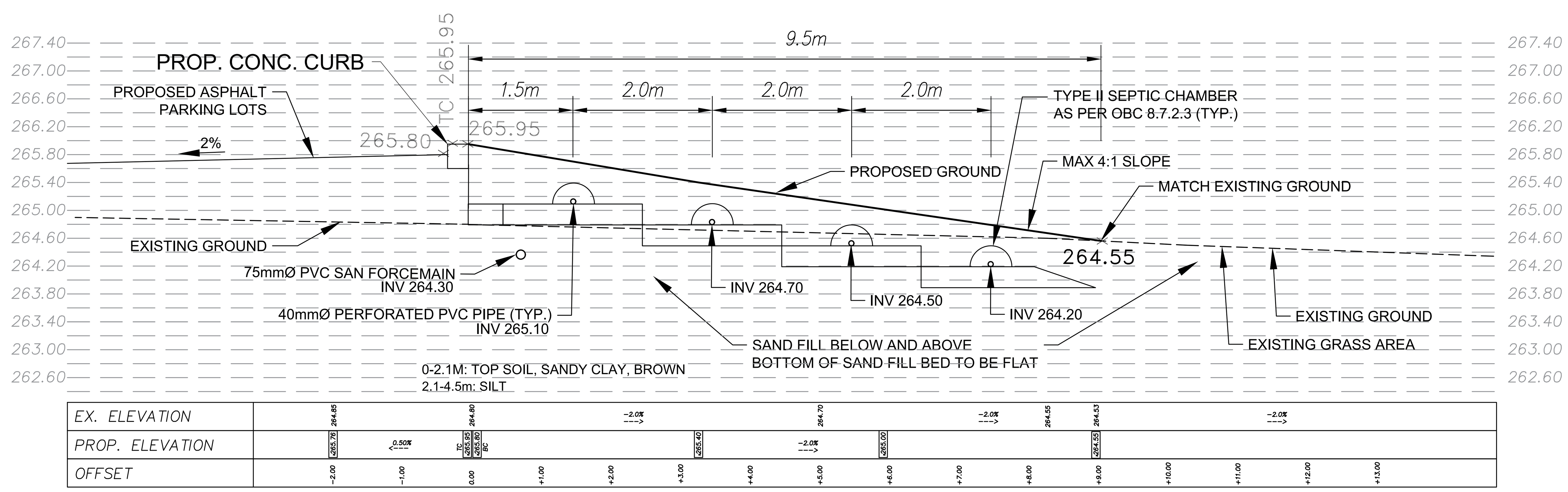
SEPTIC DESIGN

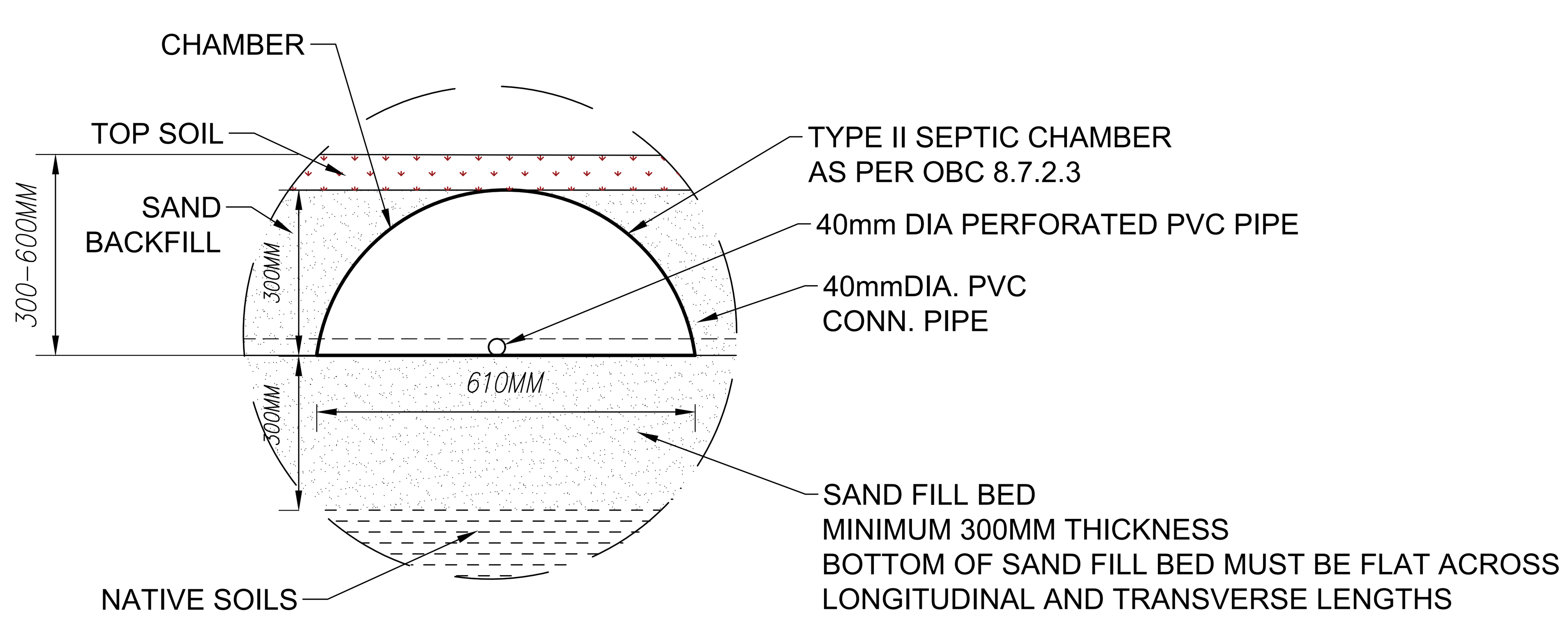
FILE No.

EGR - 1.3

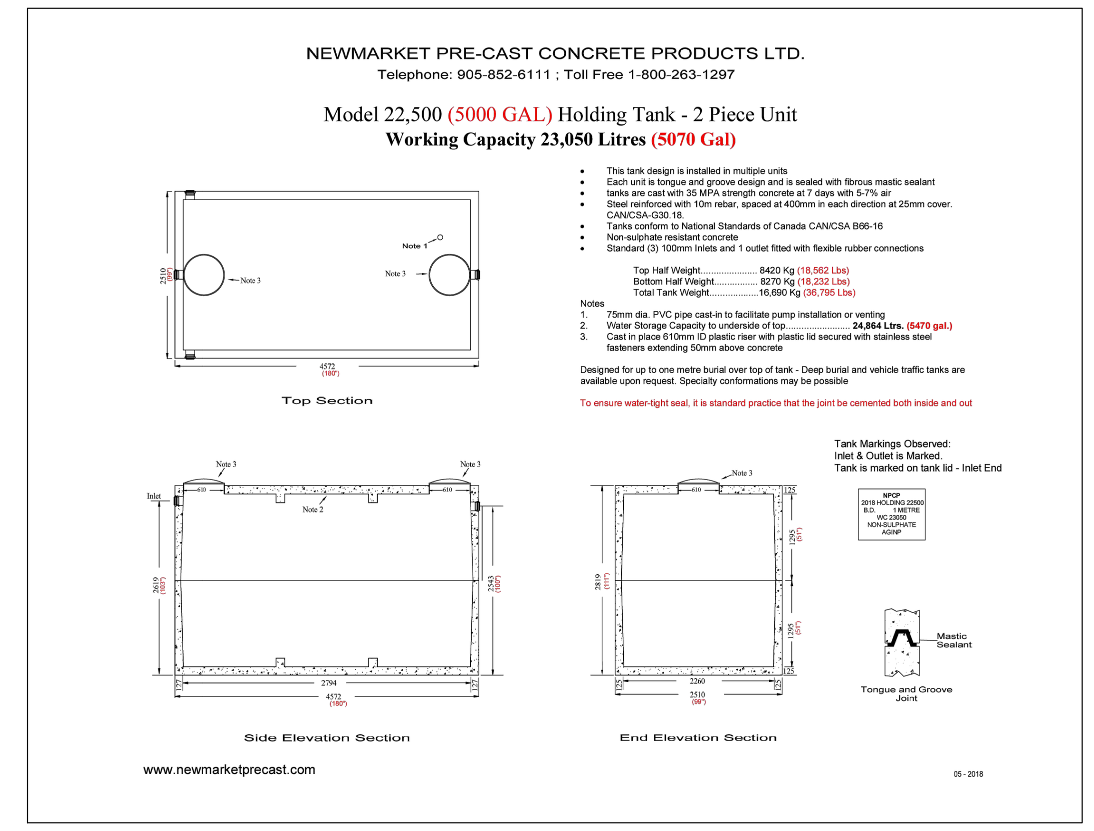
No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V5	ISSUED FOR PERMITS	DEC 07, 2023	KL	TW
V15	ISSUED FOR PERMITS	OCT 10, 2024	KL	TW
V17	ISSUED FOR PERMITS	MAR 07, 2025	TW	TW

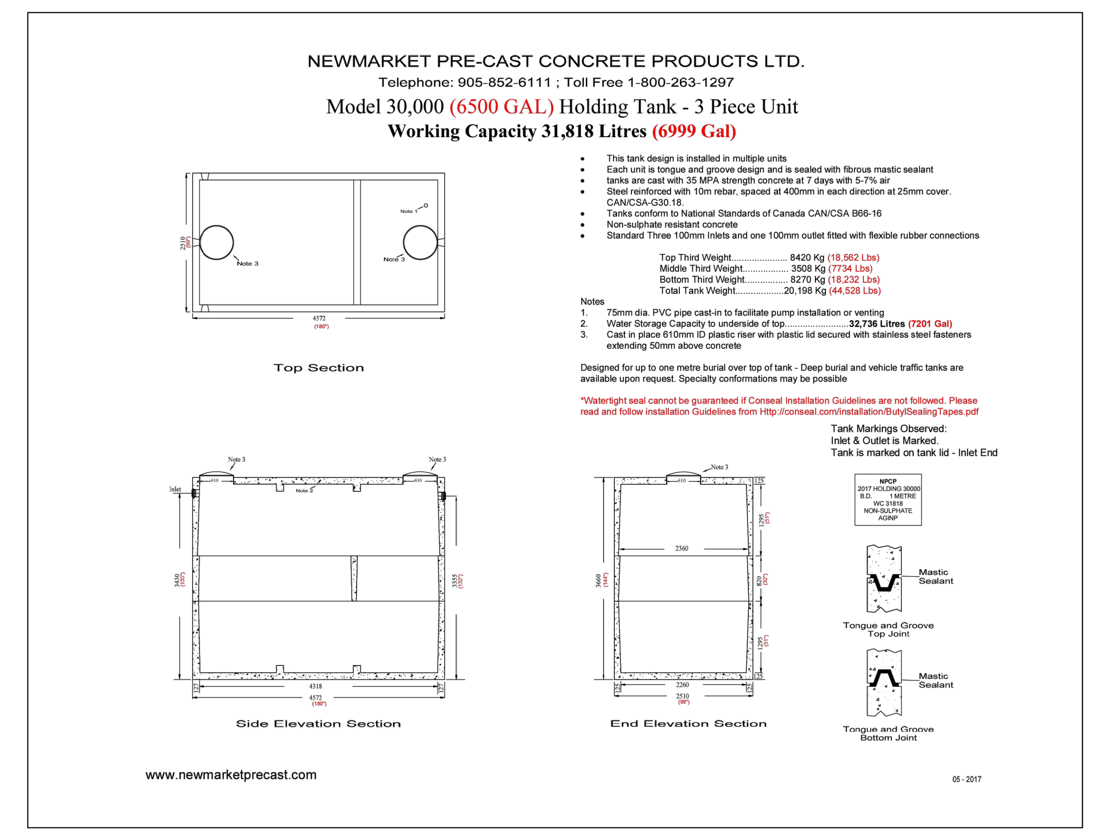
SECTION A–A' (N.T.S.)
SANITARY SEPTIC SYSTEM
CROSS–SECTION VIEW
VIEWING EAST

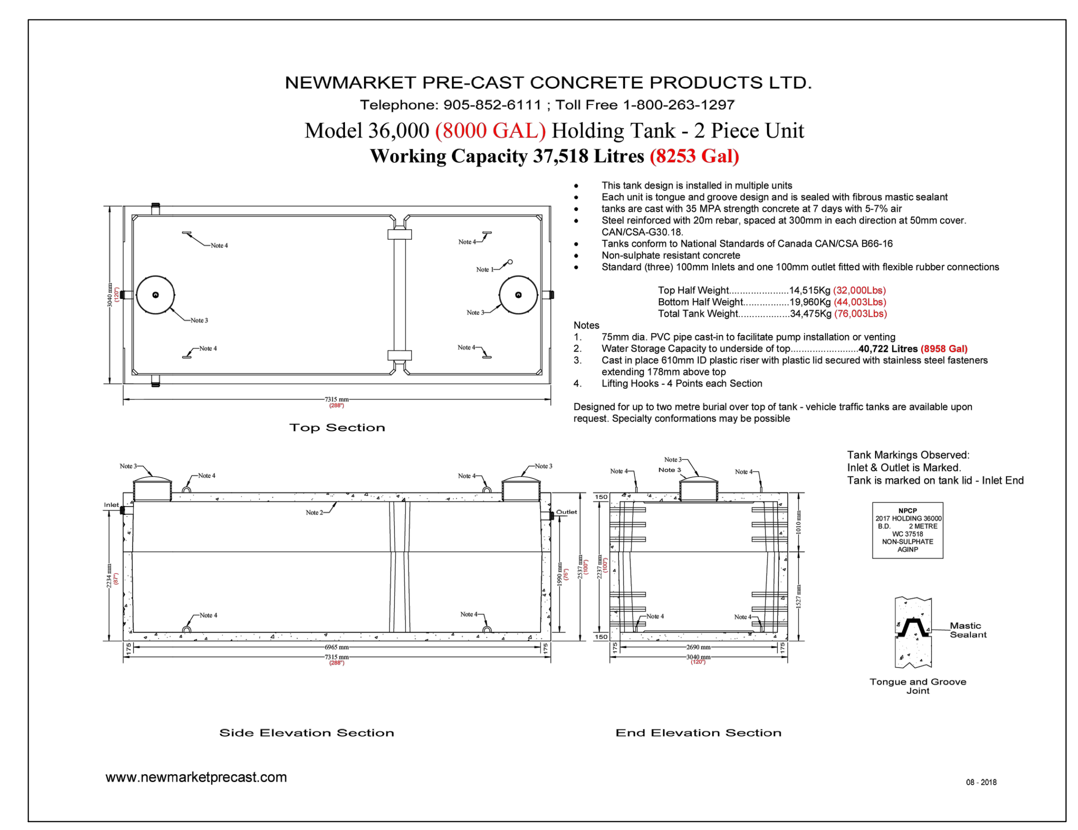




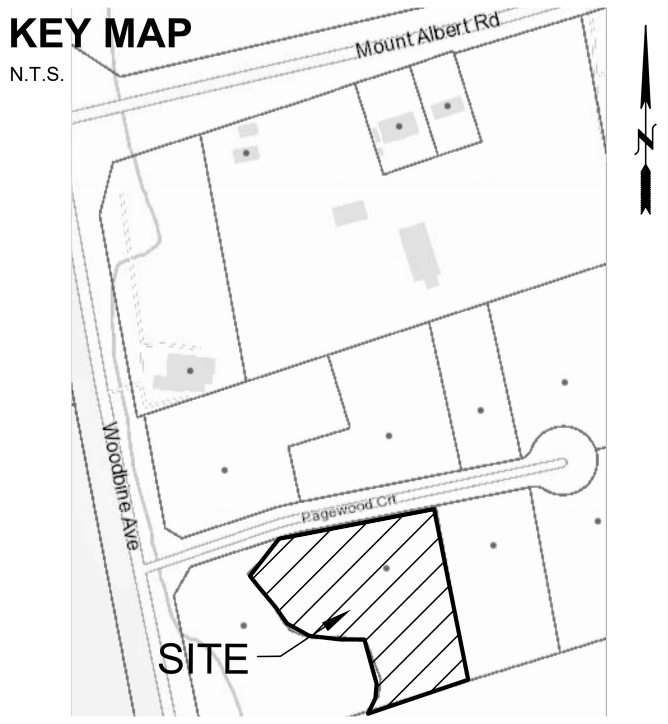
DETAIL E (N.T.S)
SHALLOW BURIED TRENCH
DETAIL VIEW


NEWMARKET PRE-CAST CONCRETE PRODUCTS LTD.
Telephone: 905-852-6111 ; Toll Free 1-800-263-1297
Model 22,500 (5000 GAL) Holding Tank - 2 Piece Unit
Working Capacity 23,050 Litres (5070 Gal)
Top Section
Side Elevation Section
End Elevation Section
www.newmarketprecast.com


NEWMARKET PRE-CAST CONCRETE PRODUCTS LTD.
Telephone: 905-852-6111 ; Toll Free 1-800-263-1297
Model 30,000 (6500 GAL) Holding Tank - 3 Piece Unit
Working Capacity 31,818 Litres (6999 Gal)
Top Section
Side Elevation Section
End Elevation Section
www.newmarketprecast.com


NEWMARKET PRE-CAST CONCRETE PRODUCTS LTD.
Telephone: 905-852-6111 ; Toll Free 1-800-263-1297
Model 36,000 (8000 GAL) Holding Tank - 2 Piece Unit
Working Capacity 37,518 Litres (8253 Gal)
Top Section
Side Elevation Section
End Elevation Section
www.newmarketprecast.com

KEY MAP
N.T.S.



DRAWN
TW

DATE
MAR 07, 2025

LICENSED PROFESSIONAL ENGINEER
Y.T. WANG
100228476
03/07/2025
PROVINCE OF ONTARIO

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CLIENT
WELEAP INVESTMENTS LTD.

PROJECT NAME
CONSTRUCTION OF SELF-STORAGE WAREHOUSE UNITS

PROJECT LOCATION
25 PAGEWOOD COURT EAST GWILLIMBURY

PRINT TITLE
SEPTIC SYSTEM DETAILS

FILE No.
EGR - 1.4

No.	ISSUED FOR:	DATE	DRAW BY	CHECK
V5	ISSUED FOR PERMITS	DEC 07, 2023	KL	TW
V15	ISSUED FOR PERMITS	OCT 10, 2024	KL	TW
V17	ISSUED FOR PERMITS	MAR 07, 2025	TW	TW

APPENDIX III – BOREHOLE DRILL LOG

GROUNDWATER MONITORING Well MH101

KING EPCM
Flexible. Dependable. On-site Engineering.

PROJECT NUMBER	**DRILLING COMPANY** King EPCM	**COORDINATES** 4884788.815, 626959.725
PROJECT NAME 25 Pagewood Court	**DRILLER** Chris, Leng	**COORD SYS** UTM-17
CLIENT	**DRILL RIG** Little Beaver	**SURFACE ELEVATION** 266.416
ADDRESS 25 Pagewood Court	**DRILLING METHOD** Solid Auger	**WELL TOC** None
DRILLING DATE 06/28/2023	**TOTAL DEPTH** 4.5m	**LOGGED BY** Leng
LICENCE NO. C-7691	**DIAMETER** 2.5 in	**CHECKED BY** Tony Wang
COMPLETION	**CASING** 2 inch	**SCREEN** 2 inch

COMMENTS

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Well Installation	Elevation (m)
0.0 – 0.3			Backfill		266.4 – 266.2
0.3 – 0.9		USCS: SP	Brown sand, moist		266 – 265.6
0.9 – 2.1		USCS:CL	Brown sandy clay, moist, low plasticity		265.4 – 264.4
2.1 – 4.0		USCS:CL	Brown sandy clay, wet		264.2 – 262.4
4.0 – 4.5		USCS:MH	Brown silt		262.4 – 262
4.5			Termination Depth at: 4.5 m		261.8

Depth scale (m): 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.4, 2.6, 2.8, 3, 3.2, 3.4, 3.6, 3.8, 4, 4.2, 4.4, 4.6

Elevation scale (m): 266.4, 266.2, 266, 265.8, 265.6, 265.4, 265.2, 265, 264.8, 264.6, 264.4, 264.2, 264, 263.8, 263.6, 263.4, 263.2, 263, 262.8, 262.6, 262.4, 262.2, 262, 261.8

Disclaimer

KING EPCM

Flexible. Dependable. On-site Engineering.

GROUNDWATER MONITORING Well MH102

PROJECT NUMBER	**DRILLING COMPANY** King EPCM	**COORDINATES** 4884748.618, 626938.235
PROJECT NAME 25 Pagewood Court	**DRILLER** Chris, Leng	**COORD SYS** UTM-17
CLIENT	**DRILL RIG** Little Beaver	**SURFACE ELEVATION** 265.58
ADDRESS 25 Pagewood Court	**DRILLING METHOD** Solid Auger	**WELL TOC** None
DRILLING DATE 06/28/2023	**TOTAL DEPTH** 3.5m	**LOGGED BY** Leng
LICENCE NO. C-7691	**DIAMETER** 2.5 in	**CHECKED BY** Tony Wang
COMPLETION	**CASING** 2 inch	**SCREEN** 2 inch

COMMENTS

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Well Installation	Elevation (m)
0.2			Backfill soil and gravels		265.4
0.4					265.2
0.6					265
0.8					264.8
1		USCS: CL	Brown sandy clay, wet, low plasticity		264.6
1.2					264.4
1.4					264.2
1.6					264
1.8					263.8
2					263.6
2.2		USCS:MH	Brown silt		263.4
2.4					263.2
2.6					263
2.8					262.8
3					262.6
3.2					262.4
3.4					262.2
3.6			Termination Depth at: 3.5 m		262

Disclaimer

GROUNDWATER MONITORING Well MH103

PROJECT NUMBER
PROJECT NAME 25 Pagewood Court
CLIENT
ADDRESS 25 Pagewood Court
DRILLING DATE 06/29/2023
LICENCE NO. C-7691

DRILLING COMPANY King EPCM
DRILLER Chris, Leng
DRILL RIG Little Beaver
DRILLING METHOD Solid Auger
TOTAL DEPTH 4.5m
DIAMETER 2.5 in

COORDINATES 4884762.162, 626875.877
COORD SYS UTM-17
SURFACE ELEVATION 265.598
WELL TOC None
LOGGED BY Leng
CHECKED BY Tony Wang

COMPLETION
CASING 2 inch
SCREEN 2 inch

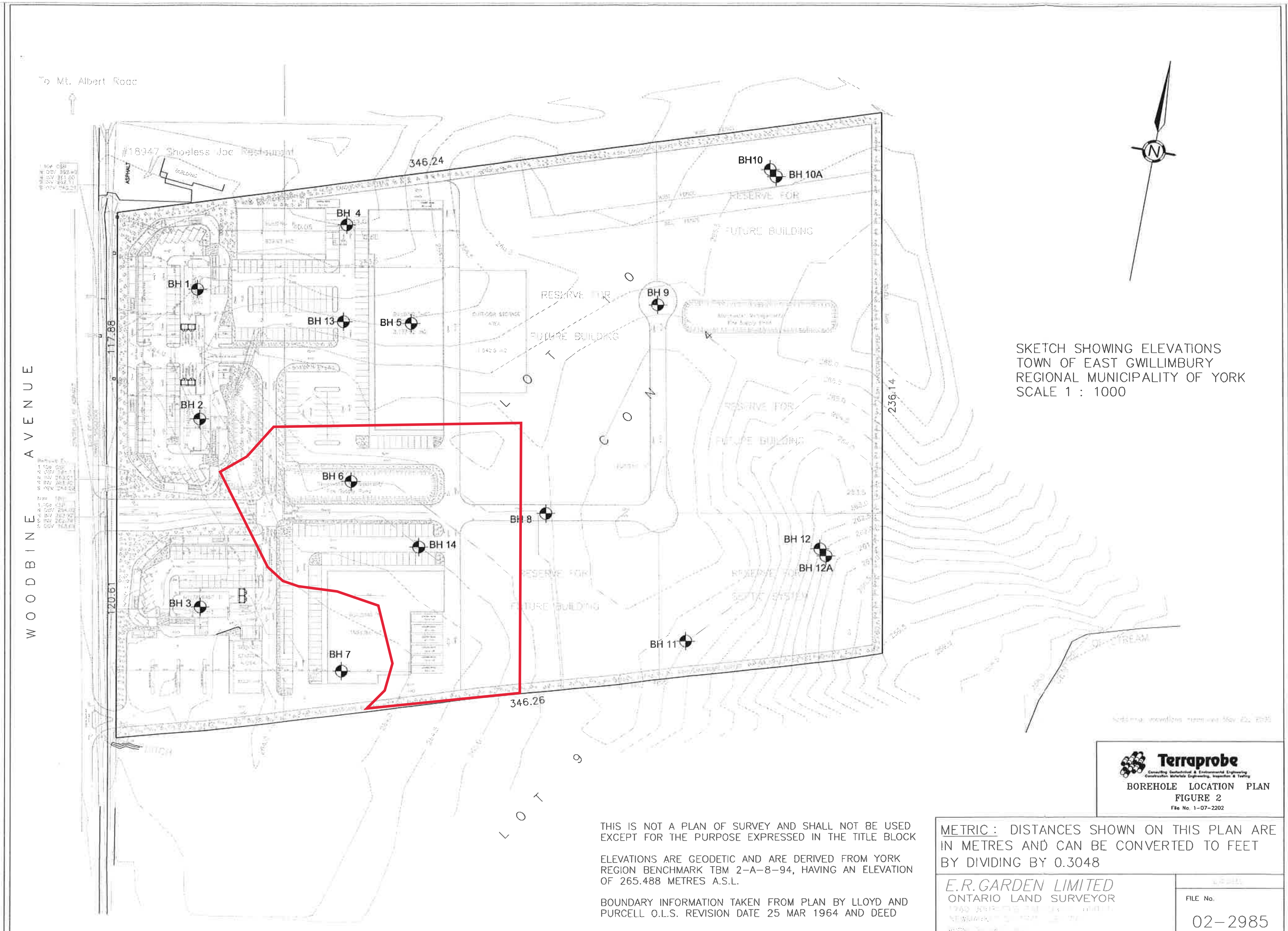
COMMENTS

Depth (m)	Graphic Log	USCS SAMPLES	Material Description	Well Installation	Elevation (m)
0.0–0.6			Backfill		266.4
0.6–2.1		USCS: CL	Brown sandy clay, moist		
2.1–4.0		USCS:CL	Brown sandy clay, wet, low plasticity		
4.0–4.5		USCS:MH	Brown silt		
			Termination Depth at: 4.5 m		

Depth scale (m): 0.2, 0.4, 0.6, 0.8, 1, 1.2, 1.4, 1.6, 1.8, 2, 2.2, 2.4, 2.6, 2.8, 3, 3.2, 3.4, 3.6, 3.8, 4, 4.2, 4.4, 4.6

Elevation scale (m): 266.4, 266.2, 266, 265.8, 265.6, 265.4, 265.2, 265, 264.8, 264.6, 264.4, 264.2, 264, 263.8, 263.6, 263.4, 263.2, 263, 262.8, 262.6, 262.4, 262.2, 262, 261.8

Disclaimer



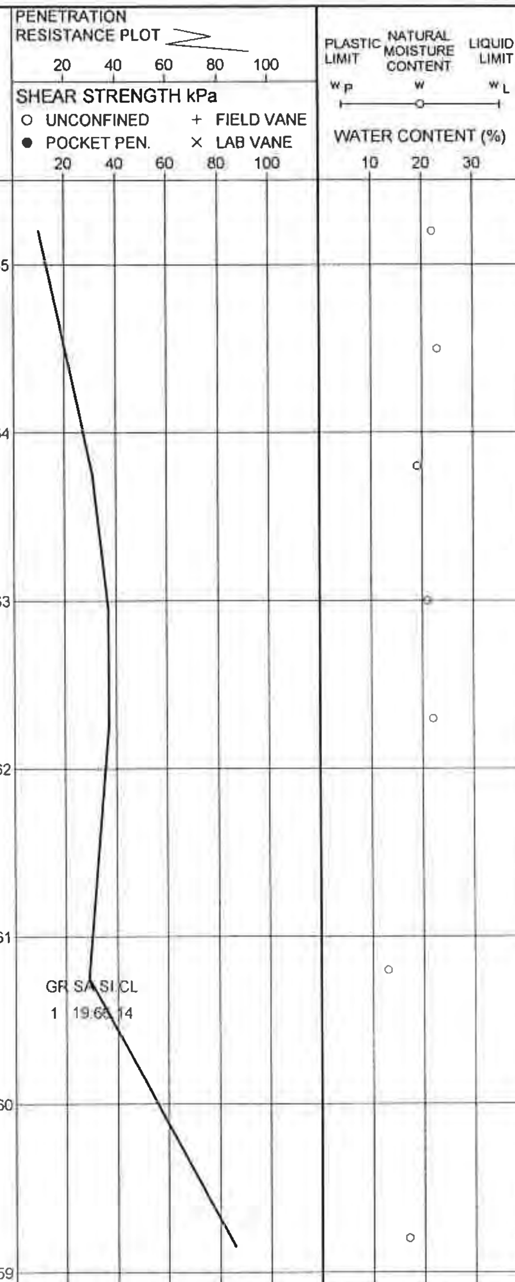
To Mt. Albert Road
#18947 Shoeless Joe Restaurant
WOODBINE AVENUE
117.88
120.61
346.24
346.26
236.14
BH 1
BH 2
BH 3
BH 4
BH 5
BH 6
BH 7
BH 8
BH 9
BH10
BH 10A
BH 11
BH 12
BH 12A
BH 13
BH 14
RESERVE FOR
FUTURE BUILDING
RESERVE FOR
FUTURE BUILDING
RESERVE FOR
FUTURE BUILDING
RESERVE FOR
FUTURE BUILDING
RESERVE FOR
SEPTIC SYSTEM
LOT 10
CON 4
LOT 9
N
SKETCH SHOWING ELEVATIONS
TOWN OF EAST GWILLIMBURY
REGIONAL MUNICIPALITY OF YORK
SCALE 1 : 1000
Terraprobe
BOREHOLE LOCATION PLAN
FIGURE 2
File No. 1-07-2202
THIS IS NOT A PLAN OF SURVEY AND SHALL NOT BE USED EXCEPT FOR THE PURPOSE EXPRESSED IN THE TITLE BLOCK
ELEVATIONS ARE GEODETIC AND ARE DERIVED FROM YORK REGION BENCHMARK TBM 2-A-8-94, HAVING AN ELEVATION OF 265.488 METRES A.S.L.
BOUNDARY INFORMATION TAKEN FROM PLAN BY LLOYD AND PURCELL O.L.S. REVISION DATE 25 MAR 1964 AND DEED
METRIC : DISTANCES SHOWN ON THIS PLAN ARE IN METRES AND CAN BE CONVERTED TO FEET BY DIVIDING BY 0.3048
E.R.GARDEN LIMITED
ONTARIO LAND SURVEYOR
FILE No.
02-2985

Terraprobe

LOG OF BOREHOLE 6

PROJECT: 18879 & 18917 Woodbine Avenue
LOCATION: East Gwillimbury, ON
CLIENT: Mi-Ko Urban Consulting
DATE: 11 July 2007
EQUIPMENT: Bombardier / Solid Stem Auger
ELEVATION DATUM: Geodetic
FILE: 1-07-2202

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES
265.5	Ground Surface			
0.0 / 265.2	350mm TOPSOIL	1	SS	10
0.4 / 264.7	Weathered / Disturbed, trace organics, rusted brown			
0.8	SILT some sand TO SAND AND SILT trace to some clay, trace gravel, compact to dense, brown, very moist to wet (GLACIAL TILL)	2	SS	21
		3	SS	31
		4	SS	37
		5	SS	37
	--- grey			
		6	SS	29
	--- very dense	7	SS	85
258.9 / 6.6	End of Borehole			



ORGANIC VAPOUR (ppm)

STANDPIPE INSTALLATION OR REMARKS

NOTES:
Borehole was open and dry upon completion of drilling

Terraprobe

LOG OF BOREHOLE 14

PROJECT: 18879 & 18917 Woodbine Avenue
LOCATION: East Gwillimbury, ON
CLIENT: Mi-Ko Urban Consulting
DATE: 11 July 2007
EQUIPMENT: Bombardier / Solid Stem Auger
ELEVATION DATUM: Geodetic
FILE: 1-07-2202

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	ORGANIC VAPOUR (ppm)	STANDPIPE INSTALLATION OR REMARKS
265.0	Ground Surface				265			
0.0	350mm TOPSOIL	1	SS	9				
264.7 / 0.4	Weathered / Disturbed, trace organics, rusted brown							
264.2 / 0.8	SILT - some sand TO SAND AND SILT trace to some clay, trace gravel, compact, brown, moist (GLACIAL TILL)	2	SS	19	264			
		3	SS	20				
263.0 / 2.0	End of Borehole				263			

Penetration Resistance Plot: 20 40 60 80 100
Shear Strength kPa: O Unconfined; + Field Vane; ● Pocket Pen; × Lab Vane — 20 40 60 80 100
Plastic Limit w_P — Natural Moisture Content w — Liquid Limit w_L; Water Content (%) 10 20 30

NOTES:
Borehole was open and dry upon completion of drilling.

APPENDIX IV – IN-SITU INFILTRATION

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 partial blocking of soil pores by air bubbles and it is preferred over Ks 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 Standard Soils Pask Permeameter, is an extended single-head analysis method and calculations procedure used here are 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 Irate (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} = the calculated permeability from the field test

Table 1. Parameters used

Parameter	Description	BH
Soil Texture Factor (**α***) in cm^{-1}	Porous materials that are both fine textured and massive; including unstructured clayey and silty soils, as well as very fine to fine structureless sandy materials.	0.04
R in cm/min	Quasi steady state (constant) rate of fall of water in permeameter reservoir (Measured in the site)	0.006
μ_k/μ_a	Temperature Correction Factor (t= 21$^{°c}$)	0.625
C	Shape factor	1.36
X in cm^2	Cross-sectional area of permeameter reservoir	12.8
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 Slow Soils, the K_{fs} was calculated as:

K_{fs} = 1.9E-8 m/sec= 1.9E-6 cm/sec

And then the temperature corrected permeability would be calculated using equation 2 for the rest of the site, as follows:

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

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

K_a=1.2E-8 m/sec = 1.2E-6 cm/sec

The field permeability data sheet is in the following.

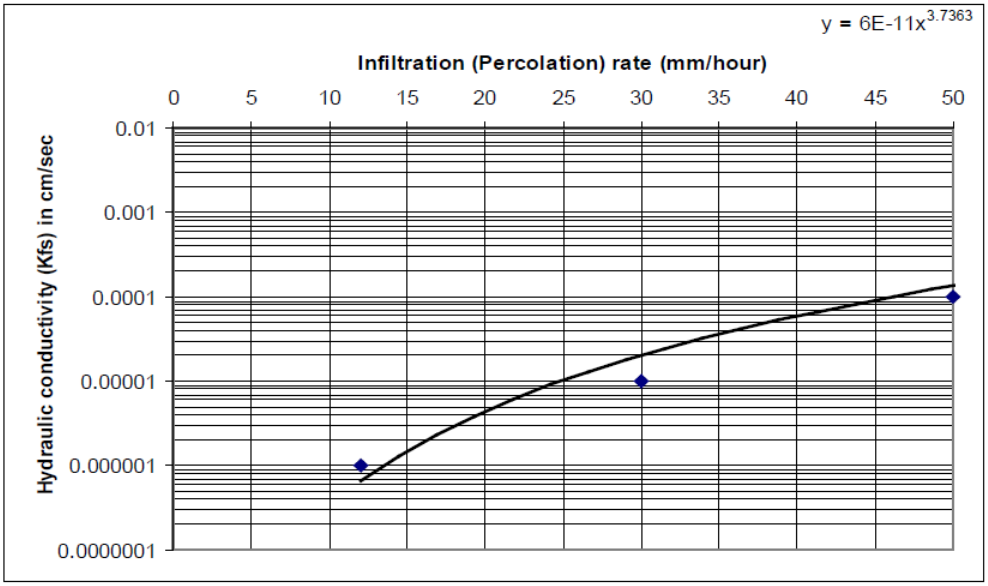
2. Percolation time/infiltration rate for design (OMMAH, 1997)

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 = 43 min / cm   (Infiltration Rate = 14 mm/hour)

As per the TRCA Stormwater Management Criteria guideline, the engineer's opinion is to trust the values obtained from this method (OMMAH, 1997), with an unfactored infiltration rate = 14 mm/hour.

3. 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 massive soil formation is sandy clay below a backfill layer (30-90cm) followed by brown silt material to depth, with an unfactored infiltration rate = 14 mm/hour at the top layer. 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 in Geotechnical Report, the predominant soil material is composed of sandy clay with different moisture content to a depth of more than 3 meters and then gradually transfers to a silt soil, which has a low permeability.

Based on Borehole datasets, the soil layer remains consistent of sandy clay material, including the soil layers 1.5 meters below the proposed bottom of the probable BMP. This means that based on the 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 14 mm/hour, and the mean infiltration rate measured in the slowest underlying soil horizon is 5.6 mm/hour which is really negligible, 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: 25 Pagewood Crt

PID #:

ST PIT#:

TECHNICIAN: Leng

DATE: June-29-2023

WEATHER/TEMPERATURE: Rainy/21c

FIELD PERMEABILITY TEST #:

D - reservoir diameter (cm)	Standard	Soil Texture	
d - well hole diameter (cm)		Soil Structure	
height of water in well (cm)		α*(cm-1)	
pth 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		41.9		
10	10	41	0.9	0.090
20	10	40.9	0.1	0.010
30		40.8		
40		40.8		
50		40.7		
60		40.7		
70		40.6		
80		40.5		
90		40.5		
100	80	40.4	0.5	0.006

uasi Steady-State Rate of Fall(R) = 0.006 cm/min

APPENDIX V –CLIMATE DATA TABLE

Black River Subwatershed

	Hydrologic Soil Group	Subwatershed Area (km^2)	Mean Annual Precipitation (mm/yr.)	Actual Evapotranspiration (mm/yr.)	Precipitation Surplus (mm/yr.)
Urban Lawns/Golf Courses					
Fine Sand	A	3.13	895	564	331
Fine Sandy Loam	B		895	579	316
Silt Loam	C		895	569	326
Clay	D		895	596	299
Forest					
Fine Sand	A	73.90	895	578	317
Fine Sandy Loam	B		895	605	290
Silt Loam	C		895	589	306
Clay	D		895	632	263
Pasture & Shrubs					
Fine Sand	A	14.32	895	581	314
Fine Sandy Loam	B		895	605	290
Silt Loam	C		895	591	304
Clay	D		895	607	288
Non-Intensive Agriculture (e.g. Hay)					
Fine Sand	A	57.67	895	581	314
Fine Sandy Loam	B		895	603	292
Silt Loam	C		895	624	271
Clay	D		895	601	294
Intensive Agriculture (e.g. Row crop)					
Fine Sand	A	86.23	895	585	310
Fine Sandy Loam	B		895	615	280
Silt Loam	C		895	620	275
Clay	D		895	652	243
Open Alvar					
Fine Sand	A	-	-	-	-
Fine Sandy Loam	B		-	-	-
Silt Loam	C		-	-	-
Clay	D		-	-	-
Aggregates					
Fine Sand	A	3.53	895	486	409
Fine Sandy Loam	B		895	509	386
Silt Loam	C		895	485	410
Clay	D		-	-	-
Mean Annual			**895**	**574**	**320**

Notes: Precipitation and Actual Evapotranspiration values are the AVERAGE ANNUAL estimates obtained from the Lake Simcoe PRMS model (Earthfx, 2010).

APPENDIX VI –IDF DATA

- 24 hour Chicago distribution (where requested).

The Town or LSRCA may request other design storm lengths and distributions for evaluation during the pre-consultation process.

Rainfall IDF curves to be used as defined by the equation:

$$I = a \times (b+t)^{-c}$$

Where:

I = Rainfall intensity (mm/hr)
t = Time of Concentration (minutes)

The coefficients for a, b and c values are shown below:

Return Period	a	b	c
Pond Bypass	160	4	0.800
1:2 year	648	4	0.784
1:5 year	930	4	0.798
1:10 year	1021	3	0.787
1:25 year	1100	2	0.776
1:50 year	1488	3	0.803
1:100 year	1770	4	0.820

33.0 STORMWATER MANAGEMENT POND DESIGN

Stormwater management ponds are required to meet provincial SWM prerequisites as set out by MNR, MOE or LSRCA.

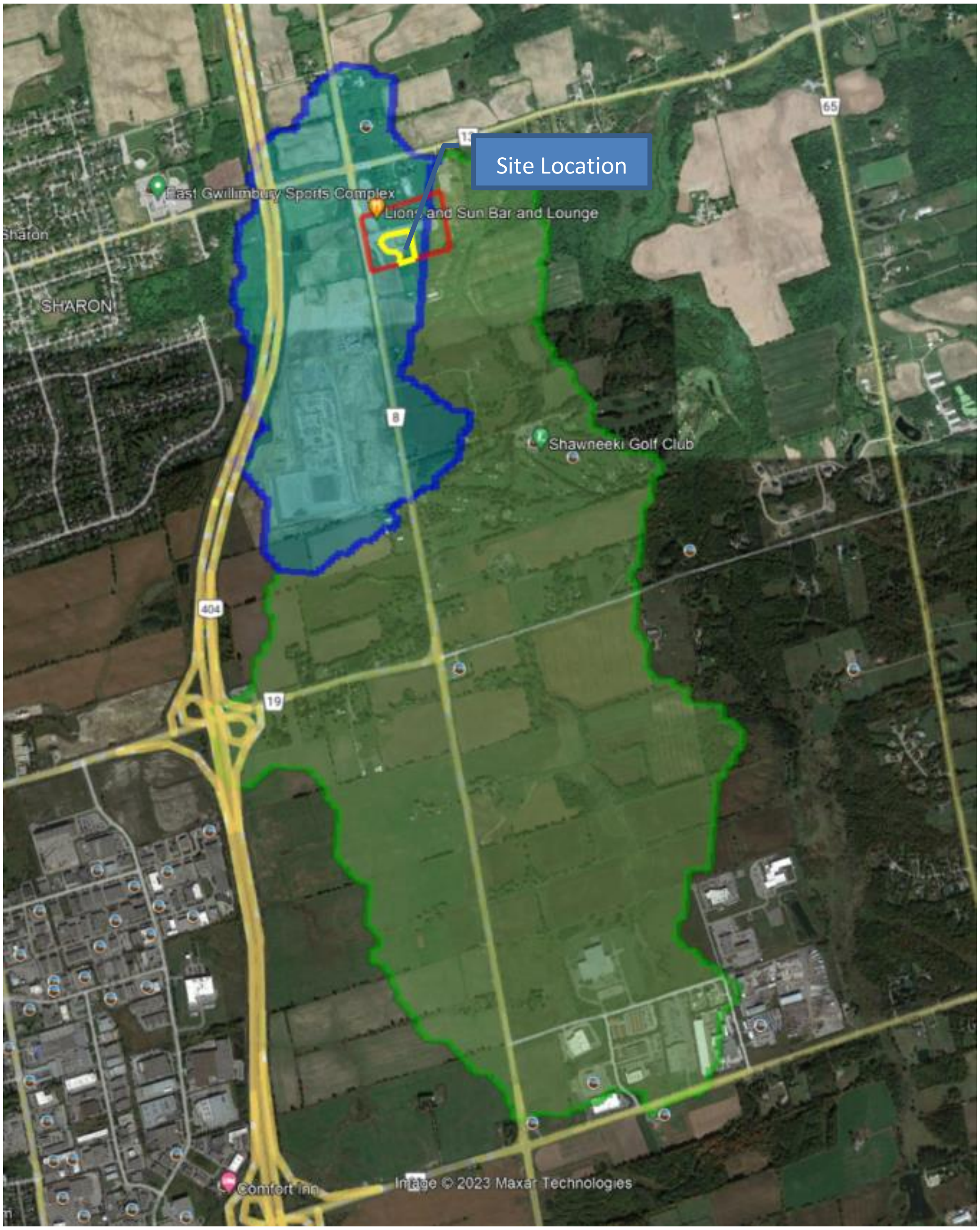
SWM pond locations, functions and design criteria shall be confirmed through consultation with the Conservation Authority and the Town. Where Stormwater Master Plans have been completed, the design criteria shall follow the approved Master Plan. End-of-pipe facilities are acceptable to the Town when the designs are safe, maintainable, integrated with the surrounding landscape, and aesthetically pleasing.

The Town concedes the overall design requirements to the most recent provincial direction, as is acceptable to the LSRCA. Exceptions to this are in circumstances that involve:

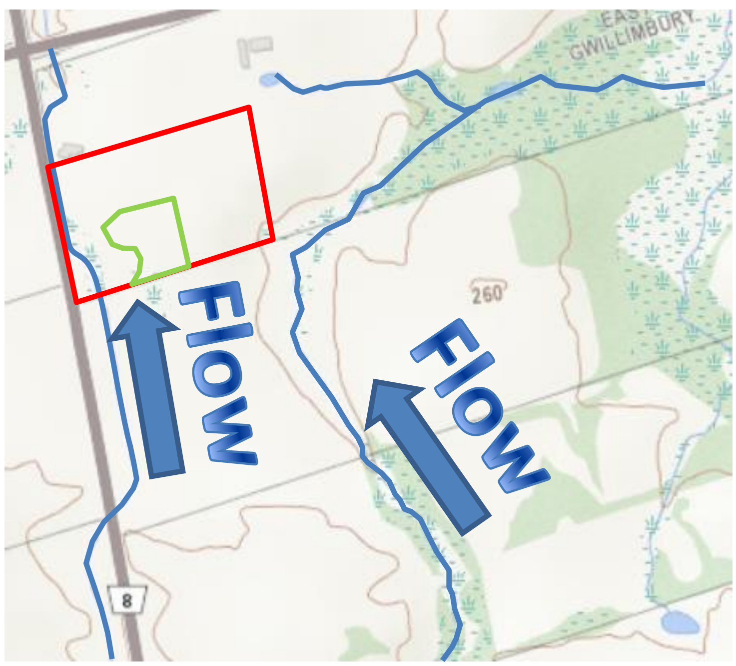
- matters of public safety and aesthetics
- operation and maintenance requirements
- protecting the riparian rights of private Landowners
- protection of municipal infrastructure
- conflicts with land use.

APPENDIX VII –HYDROLOGIC SUBWATERSHED

Watershed Calculations



Overview of Watersheds and Property Boundary



Identified Water courses in the site

APPENDIX VIII –STORMWATER MANAGEMENT CALCULATIONS

STORMWATER MANAGEMENT CALCULATIONS

Pre-Development Peak Flows

Storm (yrs)	Township of East Gwillimbury Coeff A	Coeff B	Coeff C	Modified Rational Method Q= CaCIA/360	
2	648	4	0.784	Where:	
5	930	4	0.798	Q-	Flow Rate (m3/s)
10	1021	3	0.787	Ca-	Peaking Coefficient
25	1100	2	0.776	C-	Rational Method Runoff Coefficient
50	1488	3	0.803	I-	Storm Intensity (mm/hr)
100	1770	4	0.82	A-	Area (ha)

Area #	(A + B1) North catchment		(B2) South catchment	
Area	0.97	ha	0.04	ha
Runoff Coefficient	0.76		0.1	
Time of Concentration	8	min	8	min
Return Rate	2	year	2	year
Peaking Coefficient	1.0		1.0	
Rainfall Intensity	92.36	mm/hr	92.36	mm/hr
Pre-Development Peak Flow	189.9	L/s	1.1	L/s
Return Rate	5	year	5	year
Peaking Coefficient	1.0		1.0	
Rainfall Intensity	128.03	mm/hr	128.03	mm/hr
Pre-Development Peak Flow	263.3	L/s	1.6	L/s
Return Rate	10	year	10	year
Peaking Coefficient	1.0		1.0	
Rainfall Intensity	154.69	mm/hr	154.69	mm/hr
Pre-Development Peak Flow	318.1	L/s	1.9	L/s
Return Rate	25	year	25	year
Peaking Coefficient	1.1		1.1	
Rainfall Intensity	184.24	mm/hr	184.24	mm/hr
Pre-Development Peak Flow	416.8	L/s	2.5	L/s
Return Rate	50	year	50	year
Peaking Coefficient	1.2		1.2	
Rainfall Intensity	216.95	mm/hr	216.95	mm/hr
Pre-Development Peak Flow	535.3	L/s	3.2	L/s
Return Rate	100	year	100	year
Peaking Coefficient	1.25		1.25	
Rainfall Intensity	230.70	mm/hr	230.70	mm/hr
Pre-Development Peak Flow	593.0	L/s	3.5	L/s

Post-Development Peak Flows

Storm (yrs)	Township of East Gwillimbury Coeff A	Coeff B	Coeff C	Modified Rational Method Q= CaCIA/360	
2	648	4	0.784	Where:	
5	930	4	0.798	Q-	Flow Rate (m3/s)
10	1021	3	0.787	Ca-	Peaking Coefficient
25	1100	2	0.776	C-	Rational Method Runoff Coefficient
50	1488	3	0.803	I-	Storm Intensity (mm/hr)
100	1770	4	0.82	A-	Area (ha)

Area #	(A + B1) North catchment		(B2) South catchment	
Area	0.97	ha	0.04	ha
Runoff Coefficient	0.86		0.15	
Time of Concentration	8	min	8	min
Return Rate	2	year	2	year
Peaking Coefficient	1.0		1.0	
Rainfall Intensity	92.36	mm/hr	92.36	mm/hr
Post-Development Peak Flow	214.9	L/s	1.7	L/s
Return Rate	5	year	5	year
Peaking Coefficient	1.0		1.0	
Rainfall Intensity	128.03	mm/hr	128.03	mm/hr
Post-Development Peak Flow	297.9	L/s	2.3	L/s
Return Rate	10	year	10	year
Peaking Coefficient	1.0		1.0	
Rainfall Intensity	154.69	mm/hr	154.69	mm/hr
Post-Development Peak Flow	359.9	L/s	2.8	L/s
Return Rate	25	year	25	year
Peaking Coefficient	1.1		1.1	
Rainfall Intensity	184.24	mm/hr	184.24	mm/hr
Post-Development Peak Flow	471.6	L/s	3.7	L/s
Return Rate	50	year	50	year
Peaking Coefficient	1.2		1.2	
Rainfall Intensity	216.95	mm/hr	216.95	mm/hr
Post-Development Peak Flow	605.8	L/s	4.7	L/s
Return Rate	100	year	100	year
Peaking Coefficient	1.25		1.25	
Rainfall Intensity	230.70	mm/hr	230.70	mm/hr
Post-Development Peak Flow	671.0	L/s	5.2	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)	Qpre (l/s)	Q_U (l/s)	Q (l/s)
2	124.1	1.7	122.4
5	172.0	2.3	169.7
10	207.8	2.8	205.0
25	272.3	3.7	268.6
50	349.8	4.7	345.0
100	387.4	5.2	382.2

- ☑ The above calculation (Pre-development Flow) is for the southern portion of the driveway/parking lots + Southern half of the building rooftops which will be discharged into the storage tank, Timp = 6306 m^2 = Catchments B1

- ☑ Post-Development Uncontrolled Flow is for the Septic Bed Area = Catchment B2

Quantity Control Volume Calculations

Drainage Area (ha)	**0.97**
Runoff Coeff. (C)	**0.86**
Time of Concentration	8 min
Time Step	1 min
Controlled Release Rate (Q_C)	122.4 - 382.2
Max. Storage Required	**145.2**

Results

Storm Event (yr)	Storage (m3)	Time (min)
2	46	7
5	64	6
10	79.2	6
25	107.5	6
50	134.1	6
100	145.2	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	183.48	0.430	25.8	7.3	18.5
2	159.04	0.373	44.8	14.7	30.1
3	140.93	0.331	59.5	22.0	37.5
4	126.93	0.298	71.4	29.4	42.1
5	115.73	0.271	81.4	36.7	44.7
6	106.56	0.250	90.0	44.1	45.9
7	98.88	0.232	97.4	51.4	46.0
8	92.36	0.217	104.0	58.8	45.2
9	86.74	0.203	109.9	66.1	43.8
10	81.85	0.192	115.2	73.4	41.7

: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	257.46	0.604	36.2	10.2	26.1
2	222.60	0.522	62.7	20.4	42.3
3	196.83	0.462	83.1	30.5	52.6
4	176.94	0.415	99.6	40.7	58.9
5	161.06	0.378	113.3	50.9	62.4
6	148.08	0.347	125.0	61.1	64.0
7	137.23	0.322	135.2	71.3	63.9
8	128.03	0.300	144.1	81.4	62.7
9	120.10	0.282	152.1	91.6	60.5
10	113.21	0.266	159.3	101.8	57.5

: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	342.93	0.804	48.3	12.3	36.0
2	287.70	0.675	81.0	24.6	56.4
3	249.24	0.585	105.2	36.9	68.3
4	220.77	0.518	124.3	49.2	75.1
5	198.74	0.466	139.8	61.5	78.3
6	181.15	0.425	153.0	73.8	79.2
7	166.73	0.391	164.3	86.1	78.2
8	154.69	0.363	174.2	98.4	75.8
9	144.45	0.339	183.0	110.7	72.3
10	135.63	0.318	190.9	123.0	67.9

: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	468.97	1.210	72.6	16.1	56.5
2	375.14	0.968	116.1	32.2	83.9
3	315.49	0.814	146.5	48.3	98.2
4	273.87	0.707	169.6	64.5	105.1
5	242.99	0.627	188.1	80.6	107.5
6	219.08	0.565	203.5	96.7	106.8
7	199.94	0.516	216.7	112.8	103.9
8	184.24	0.475	228.2	128.9	99.3
9	171.11	0.441	238.4	145.0	93.4
10	159.94	0.413	247.6	161.2	86.4

:Maximum Storage Volume

Required Storage Volumes (50yr)

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	488.82	1.376	82.6	20.7	61.8
2	408.63	1.150	138.0	41.4	96.6
3	352.98	0.993	178.8	62.1	116.7
4	311.88	0.878	210.7	82.8	127.9
5	280.17	0.789	236.6	103.5	133.1
6	254.89	0.717	258.3	124.2	134.1
7	234.21	0.659	276.9	144.9	132.0
8	216.95	0.611	293.1	165.6	127.5
9	202.31	0.569	307.5	186.3	121.2
10	189.72	0.534	320.4	207.0	113.4

: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	472.95	1.387	83.2	22.9	60.3
2	407.28	1.194	143.3	45.9	97.4
3	358.92	1.052	189.4	68.8	120.6
4	321.69	0.943	226.4	91.7	134.6
5	292.08	0.856	256.9	114.7	142.2
6	267.90	0.785	282.8	137.6	145.2
7	247.76	0.726	305.1	160.5	144.6
8	230.70	0.676	324.7	183.4	141.2
9	216.04	0.633	342.0	206.4	135.7
10	203.31	0.596	357.6	229.3	128.3

:Maximum Storage Volume

Stage-Storage-Discharge Table

Water level in Tank	Orifice 1	Orifice 2	Total	Volume		
	Outlet	Overflow				
	Discharge	Discharge	Discharge	Tank	Pipe	Total
(m)	(l/s)	(l/s)	(l/s)	(m3)	(m3)	(m3)
260.80	0.0	0.0	0.0	0.00	9.45	9.45
261.00	0.0	0.0	0.0	7.50	9.45	16.95
261.20	54.5	0.0	54.5	14.99	9.45	24.44
261.40	79.8	0.0	79.8	22.49	9.45	31.94
261.60	98.9	0.0	98.9	29.98	9.45	39.44
261.80	114.8	0.0	114.8	37.48	9.45	46.93
262.00	128.8	0.0	128.8	44.98	9.45	54.43
262.20	141.3	0.0	141.3	52.47	9.45	61.92
262.40	152.9	0.0	152.9	59.97	9.45	69.42
262.60	163.6	0.0	163.6	67.46	9.45	76.92
262.80	173.7	0.0	173.7	74.96	9.45	84.41
263.00	183.2	0.0	183.2	82.46	9.45	91.91
263.20	192.3	149.3	341.6	89.95	9.45	99.40
263.30	196.7	182.8	379.5	93.70	9.45	103.15

Orifice 1		
Orifice Diameter (mm)		250
High Water Elev. (m)		263.3
Orifice Invert Elev.(m)		260.90
Orifice Center Elev. (m)		261.03
Head (m)		2.28
C		0.6
Orifice Area (m2)		0.0491
Flow Rate (L/s)	Required	382.2
	Provided	196.7

Orifice 2 (Emergency Exit)		
Orifice Diameter (mm)		400
High Water Elev. (m)		263.3
Orifice Invert Elev.(m)		262.8
Orifice Center Elev. (m)		263.00
Head (m)		0.300
C		0.6
Orifice Area (m2)		0.1256
Flow Rate (L/s)		182.8

Parking Storage

(Parking lot/Driveway storage calculations using the Average End Area Method)

ge (m)	Increment (m)	Area (m2)	Average Area (m2)	Incremental Volume (m3)	Cumulative Volume (m3)
0.00		0			
0.05	0.05	225	112.50	5.6	5.6
0.15	0.10	674	449.50	45.0	50.6
0.25	0.10	1123	898.50	89.9	140.4

Drawdown Time

(Using the falling head orifice equation for total drawdown above the sill of orifice 1, before overflow condition)

$$t = \frac{2A_p}{CA_o(2g)^{0.5}}\,(h_1^{0.5} - h_2^{0.5})$$

Where:

t = drawdown time in second		
A_p = average surface area of tank (m^2)	37.5	
C = discharge coefficient (orifice)	0.6	
A_o = cross-sectional area of orifice (m^2)	0.0491	
g = gravitational acceleration constant (9.81 m/s^2)		
h_1 = starting water elevation above the orifice 1 before overflow condition (m)	262.8	1.9 m
h_2 = ending water elevation above the orifice 1 (m)	260.9	0 m

t = 792.42 seconds

t = 0.22 hours

Box Culvert Design

Manning Formula Uniform Trapezoidal Channel Flow at Given Slope and Depth

25 Pagewood Crt.

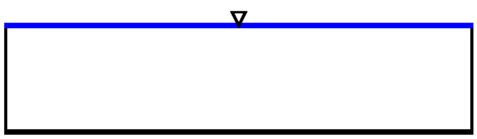
Box Culvert Design

Inputs

Bottom width, b	1.1	m
Side slope 1 (horiz./vert.)	0	
Side slope 2 (horiz./vert.)	0	
Manning roughness, n ? Strickler B/B (See notes)	0.012	
Channel slope, S	2.2	% rise/run
Flow depth, y	0.25	m
Bend Angle ? (for riprap sizing)	1	
Rock specific gravity (2.65)	2.65	

Results

Flow area, a	0.2750	m^2
Wetted perimeter, P_w	1.6000	m
Hydraulic radius, R_h	0.1719	m
Velocity, v	3.8210	m/s
Flow, Q	1050.7626	l/s
Velocity head, h_v	0.7444	m
Top width, T	1.1000	m
Froude number, F	2.44	
Average shear stress (tractive force), tau	37.0789	N/m^2
n for design rock size per Strickler	0.0256	
n for design rock size per Blodgett	0.0338	
n for design rock size per Bathurst	0.0113	
Required angular rock size, D50 (Maynord, Ruff, and Abt 1989)	1.1039	m
Required angular rock size, D50 (Searcy 1967)	0.3212	m



Storm Sewer Design

Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

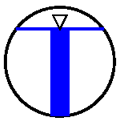
Sewer Design (Unit 1 to MH1)

Inputs

Pipe diameter, d_0	100	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1.3	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.0800	m
Flow area, a	0.0067	m^2
Pipe area, a0	0.0079	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.2214	m
Hydraulic radius, R_h	0.0304	m
Top width, T	0.0800	m
Velocity, v	0.9259	m/s
Velocity head, h_v	0.0437	m H2O
Froude number, F	1.02	
Average shear stress (tractive force), tau	3.8778	N/m^2
Flow, Q (See notes)	6.2365	l/s
Full flow, Q0	0.0064	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

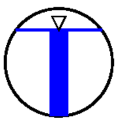
Sewer Design (CB4 to MH1)

Inputs

Pipe diameter, d_0	250	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	2.4	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2000	m
Flow area, a	0.0421	m^2
Pipe area, a0	0.0491	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.5536	m
Hydraulic radius, R_h	0.0760	m
Top width, T	0.2000	m
Velocity, v	2.3173	m/s
Velocity head, h_v	0.2738	m H2O
Froude number, F	1.61	
Average shear stress (tractive force), tau	17.8975	N/m^2
Flow, Q (See notes)	97.5550	l/s
Full flow, Q0	0.0998	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

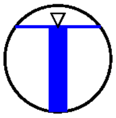
Sewer Design (CBMH2 to CBMH3)

Inputs

Pipe diameter, d_0	300	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	0.5	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2400	m
Flow area, a	0.0606	m^2
Pipe area, a0	0.0707	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.6643	m
Hydraulic radius, R_h	0.0913	m
Top width, T	0.2400	m
Velocity, v	1.1944	m/s
Velocity head, h_v	0.0727	m H2O
Froude number, F	0.76	
Average shear stress (tractive force), tau	4.4744	N/m^2
Flow, Q (See notes)	72.4067	l/s
Full flow, Q0	0.0741	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

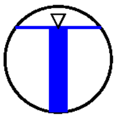
Sewer Design (Unit 2/3 to CBMH3)

Inputs

Pipe diameter, d_0	150	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	2.6	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.1200	m
Flow area, a	0.0152	m^2
Pipe area, a0	0.0177	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.3321	m
Hydraulic radius, R_h	0.0456	m
Top width, T	0.1200	m
Velocity, v	1.7158	m/s
Velocity head, h_v	0.1501	m H2O
Froude number, F	1.54	
Average shear stress (tractive force), tau	11.6334	N/m^2
Flow, Q (See notes)	26.0036	l/s
Full flow, Q0	0.0266	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

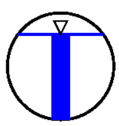
Sewer Design (CBMH3 to Tank)

Inputs

Pipe diameter, d_0	300	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1.5	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2400	m
Flow area, a	0.0606	m^2
Pipe area, a0	0.0707	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.6643	m
Hydraulic radius, R_h	0.0913	m
Top width, T	0.2400	m
Velocity, v	2.0688	m/s
Velocity head, h_v	0.2182	m H2O
Froude number, F	1.31	
Average shear stress (tractive force), tau	13.4231	N/m^2
Flow, Q (See notes)	125.4122	l/s
Full flow, Q0	0.1283	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

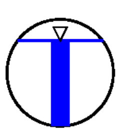
Sewer Design (Unit 4 to CB5)

Inputs

Pipe diameter, d_0	100	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.0800	m
Flow area, a	0.0067	m^2
Pipe area, a0	0.0079	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.2214	m
Hydraulic radius, R_h	0.0304	m
Top width, T	0.0800	m
Velocity, v	0.8121	m/s
Velocity head, h_v	0.0336	m H2O
Froude number, F	0.89	
Average shear stress (tractive force), tau	2.9829	N/m^2
Flow, Q (See notes)	5.4698	l/s
Full flow, Q0	0.0056	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

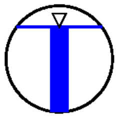
Sewer Design (CB5 to CBMH4)

Inputs

Pipe diameter, d_0	250	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	2	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2000	m
Flow area, a	0.0421	m^2
Pipe area, a0	0.0491	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.5536	m
Hydraulic radius, R_h	0.0760	m
Top width, T	0.2000	m
Velocity, v	2.1154	m/s
Velocity head, h_v	0.2282	m H2O
Froude number, F	1.47	
Average shear stress (tractive force), tau	14.9146	N/m^2
Flow, Q (See notes)	89.0552	l/s
Full flow, Q0	0.0911	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

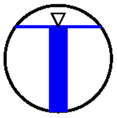
Sewer Design (CBMH4 to Tank)

Inputs

Pipe diameter, d_0	300	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1.7	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2400	m
Flow area, a	0.0606	m^2
Pipe area, a0	0.0707	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.6643	m
Hydraulic radius, R_h	0.0913	m
Top width, T	0.2400	m
Velocity, v	2.2024	m/s
Velocity head, h_v	0.2473	m H2O
Froude number, F	1.40	
Average shear stress (tractive force), tau	15.2129	N/m^2
Flow, Q (See notes)	133.5114	l/s
Full flow, Q0	0.1366	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

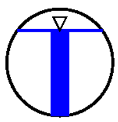
Sewer Design (Tank to MH2)

Inputs

Pipe diameter, d_0	450	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.3600	m
Flow area, a	0.1364	m^2
Pipe area, a0	0.1590	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.9964	m
Hydraulic radius, R_h	0.1369	m
Top width, T	0.3600	m
Velocity, v	2.2134	m/s
Velocity head, h_v	0.2498	m H2O
Froude number, F	1.15	
Average shear stress (tractive force), tau	13.4231	N/m^2
Flow, Q (See notes)	301.9053	l/s
Full flow, Q0	0.3089	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

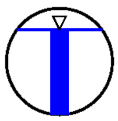
Sewer Design (MH2 to CBMH1)

Inputs

Pipe diameter, d_0	450	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	4.9	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.3600	m
Flow area, a	0.1364	m^2
Pipe area, a0	0.1590	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.9964	m
Hydraulic radius, R_h	0.1369	m
Top width, T	0.3600	m
Velocity, v	4.8996	m/s
Velocity head, h_v	1.2240	m H2O
Froude number, F	2.54	
Average shear stress (tractive force), tau	65.7734	N/m^2
Flow, Q (See notes)	668.2959	l/s
Full flow, Q0	0.6837	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

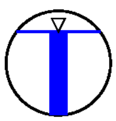
Sewer Design (Unit 3 to CB1)

Inputs

Pipe diameter, d_0	100	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	3.8	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.0800	m
Flow area, a	0.0067	m^2
Pipe area, a0	0.0079	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.2214	m
Hydraulic radius, R_h	0.0304	m
Top width, T	0.0800	m
Velocity, v	1.5830	m/s
Velocity head, h_v	0.1278	m H2O
Froude number, F	1.74	
Average shear stress (tractive force), tau	11.3351	N/m^2
Flow, Q (See notes)	10.6626	l/s
Full flow, Q0	0.0109	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

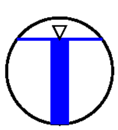
Sewer Design (CB2 to MH17) / (CB1 to MH15) / (MH1 to CBMH2) / (CBMH1 to MH15)

Inputs

Pipe diameter, d_0	300	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2400	m
Flow area, a	0.0606	m^2
Pipe area, a0	0.0707	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.6643	m
Hydraulic radius, R_h	0.0913	m
Top width, T	0.2400	m
Velocity, v	1.6891	m/s
Velocity head, h_v	0.1455	m H2O
Froude number, F	1.07	
Average shear stress (tractive force), tau	8.9488	N/m^2
Flow, Q (See notes)	102.3986	l/s
Full flow, Q0	0.1048	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

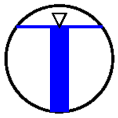
Sewer Design (Unit 4 to CBMH1)

Inputs

Pipe diameter, d_0	100	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	3.3	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.0800	m
Flow area, a	0.0067	m^2
Pipe area, a0	0.0079	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.2214	m
Hydraulic radius, R_h	0.0304	m
Top width, T	0.0800	m
Velocity, v	1.4752	m/s
Velocity head, h_v	0.1110	m H2O
Froude number, F	1.62	
Average shear stress (tractive force), tau	9.8436	N/m^2
Flow, Q (See notes)	9.9364	l/s
Full flow, Q0	0.0102	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

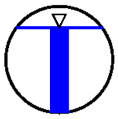
Sewer Design (CBMH1 to MH15)

Inputs

Pipe diameter, d_0	300	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	1	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2400	m
Flow area, a	0.0606	m^2
Pipe area, a0	0.0707	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.6643	m
Hydraulic radius, R_h	0.0913	m
Top width, T	0.2400	m
Velocity, v	1.6891	m/s
Velocity head, h_v	0.1455	m H2O
Froude number, F	1.07	
Average shear stress (tractive force), tau	8.9488	N/m^2
Flow, Q (See notes)	102.3986	l/s
Full flow, Q0	0.1048	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

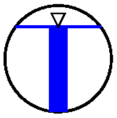
Sewer Design (Unit 1 to CB3)

Inputs

Pipe diameter, d_0	100	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	2.1	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.0800	m
Flow area, a	0.0067	m^2
Pipe area, a0	0.0079	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.2214	m
Hydraulic radius, R_h	0.0304	m
Top width, T	0.0800	m
Velocity, v	1.1768	m/s
Velocity head, h_v	0.0706	m H2O
Froude number, F	1.30	
Average shear stress (tractive force), tau	6.2641	N/m^2
Flow, Q (See notes)	7.9265	l/s
Full flow, Q0	0.0081	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

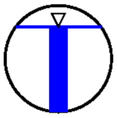
Sewer Design (CB3 to MH17)

Inputs

Pipe diameter, d_0	300	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	2.8	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.2400	m
Flow area, a	0.0606	m^2
Pipe area, a0	0.0707	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.6643	m
Hydraulic radius, R_h	0.0913	m
Top width, T	0.2400	m
Velocity, v	2.8265	m/s
Velocity head, h_v	0.4074	m H2O
Froude number, F	1.80	
Average shear stress (tractive force), tau	25.0565	N/m^2
Flow, Q (See notes)	171.3456	l/s
Full flow, Q0	0.1753	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



Manning Formula Uniform Pipe Flow at Given Slope and Depth

25 Pagewood Crt.

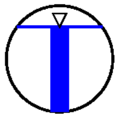
Sewer Design (Unit 2 to CB2)

Inputs

Pipe diameter, d_0	100	mm
Manning roughness, n	0.012	
Pressure slope (possibly ? equal to pipe slope), S_0	3	% rise/run
Relative flow depth, y/d_0	0.8	fraction

Results

Flow depth, y	0.0800	m
Flow area, a	0.0067	m^2
Pipe area, a0	0.0079	m^2
Relative area, a/a0	0.8576	fraction
Wetted perimeter, P_w	0.2214	m
Hydraulic radius, R_h	0.0304	m
Top width, T	0.0800	m
Velocity, v	1.4065	m/s
Velocity head, h_v	0.1009	m H2O
Froude number, F	1.55	
Average shear stress (tractive force), tau	8.9488	N/m^2
Flow, Q (See notes)	9.4740	l/s
Full flow, Q0	0.0097	m^3/s
Ratio to full flow, Q/Q0	0.9775	fraction



APPENDIX IX –RUNOFF COEFFICIENT CALCULATIONS

PRE-DEVELOPMENT (Runoff coefficient)

Ref.: Runoff Coefficients, Page 48, Section G- STORM DRAINAGE & STORMWATER MANAGEMENT, Engineering Standards and Design Criteria, Town of East Gwillimbury & Design Chart 1.07, MTO Drainage Management Manual & SCS Consulting Group Ltd Stormwater management Report for Woodbine and Mount Albert Commercial / Industrial Subdivision.

Entire Site

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m^2)	A × C	A × % Imp.
Meadow	0.1	0.0	10177.1	1017.7	0.0
Natural Heritage (Woodland)	0.08	0.0	1585.4	126.8	0.0
Weighted Average			**11762.5**	**0.1**	**0.0**

Developed Area (north) – Catchments A & B1

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m^2)	A × C	A × % Imp.
Meadow	0.1	0.0	9740.5	974.1	0.0
Weighted Average			**9740.5**	**0.1***	**0.0**

** C = 0.76, based on the sub-division site plan agreement*

Developed Area (south) – Catchment B2

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m^2)	A × C	A × % Imp.
Meadow	0.1	0.0	436.6	43.7	0.0
Weighted Average			**436.6**	**0.1**	**0.0**

POST-DEVELOPMENT (Runoff coefficient)

Ref.: Runoff Coefficients, Page 48, Section G- STORM DRAINAGE & STORMWATER MANAGEMENT, Engineering Standards and Design Criteria, Town of East Gwillimbury & Design Chart 1.07, MTO Drainage Management Manual & SCS Consulting Group Ltd Stormwater management Report for Woodbine and Mount Albert Commercial / Industrial Subdivision.

Entire Site

Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m^2)	A × C	A × % Imp.
Building	0.9	100	3118	2806.2	3118
Parking/Walkway/Driveway	0.9	100	6063.5	5457.2	6063.5
Lawn, Sand, <2% grade	0.15	0.0	995.6	149.3	0.0
Natural Heritage	0.08	0.0	1585.4	126.8	0.0
Weighted Average			**11762.5**	**0.73**	**78.1**

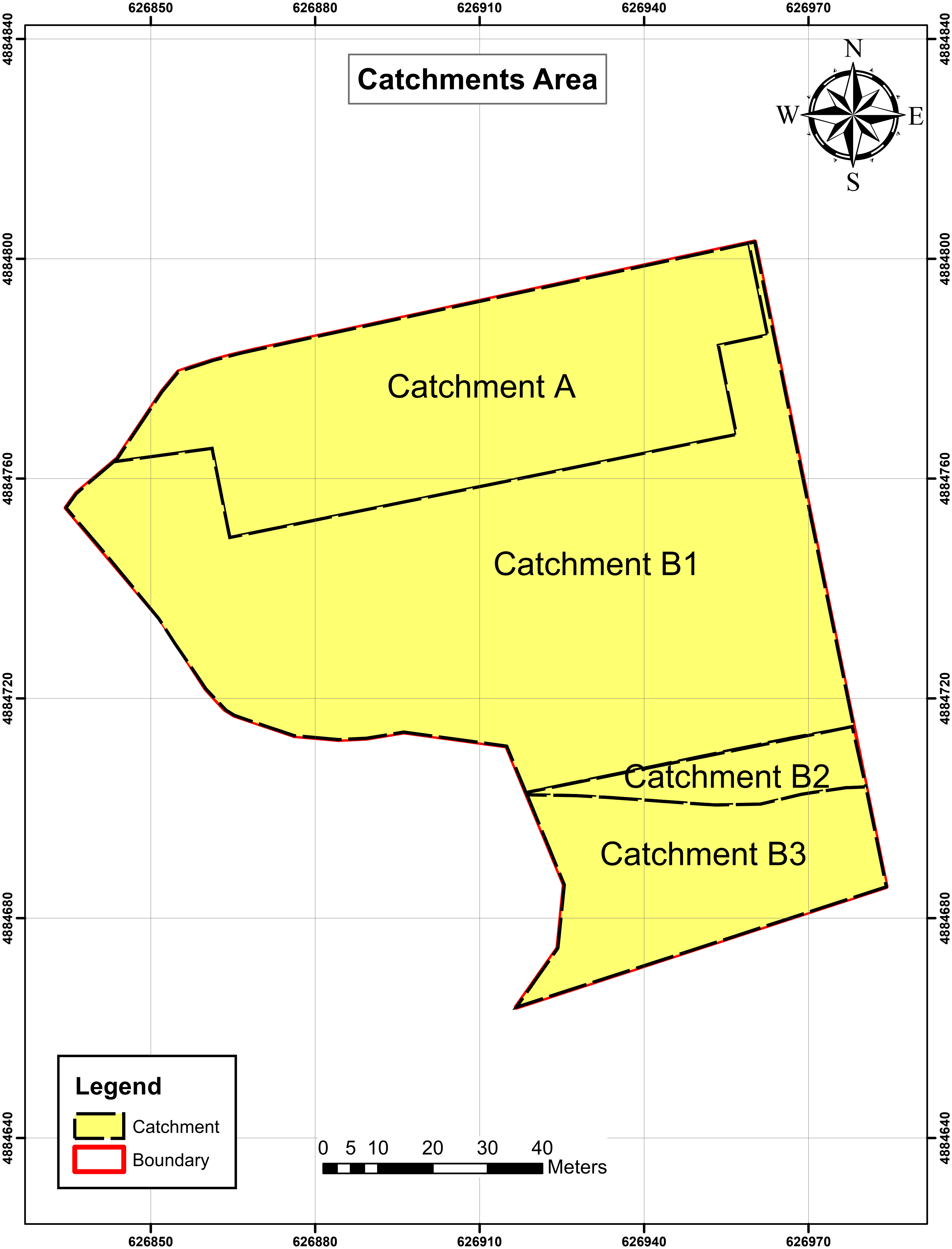
Developed Area (north) – Catchments A & B1

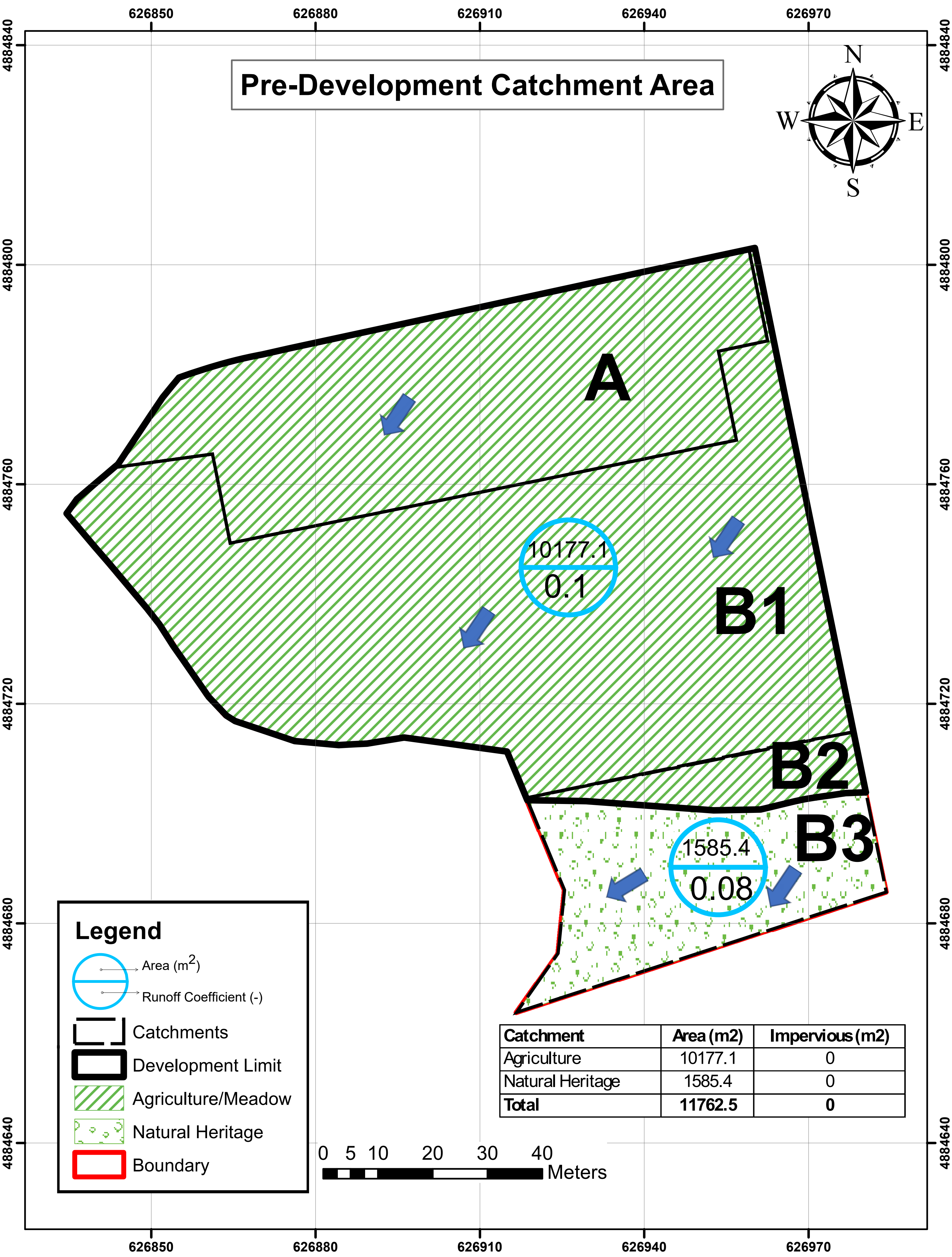
Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m^2)	A × C	A × % Imp.
Building	0.9	100	3118	2806.2	3118
Parking/Walkway/Driveway	0.9	100	6063.5	5457.2	6063.5
Lawn, Sand, <2% grade	0.15	0.0	559	83.9	0.0
Weighted Average			**9740.5**	**0.86**	**94.3**

Developed Area (south) – Catchment B2

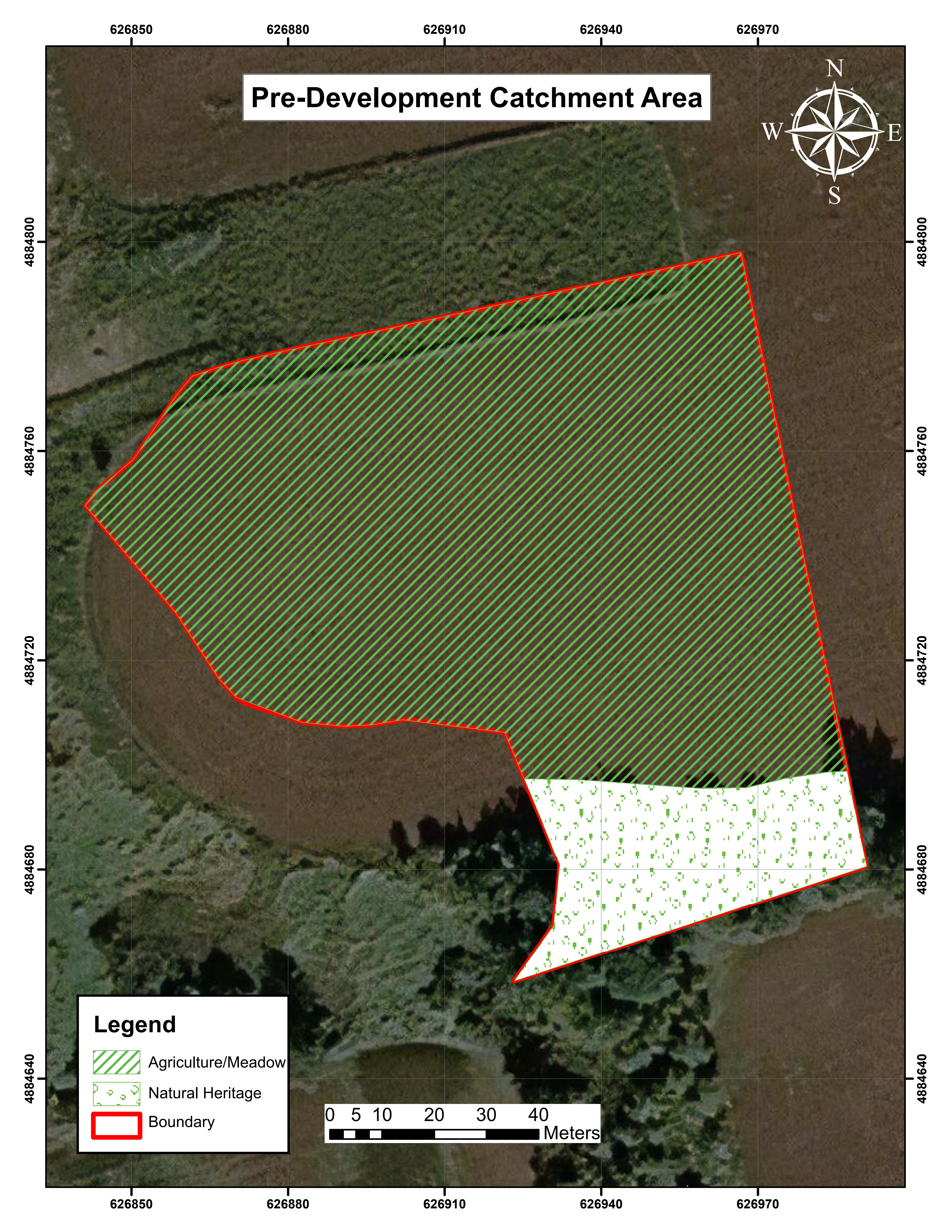
Land Use	Runoff Coefficient "C"	% Imperviousness	Total Area (m^2)	A × C	A × % Imp.
Lawn, Sand, <2% grade	0.15	0.0	436.6	65.5	0.0
Weighted Average			**436.6**	**0.15**	**0.0**

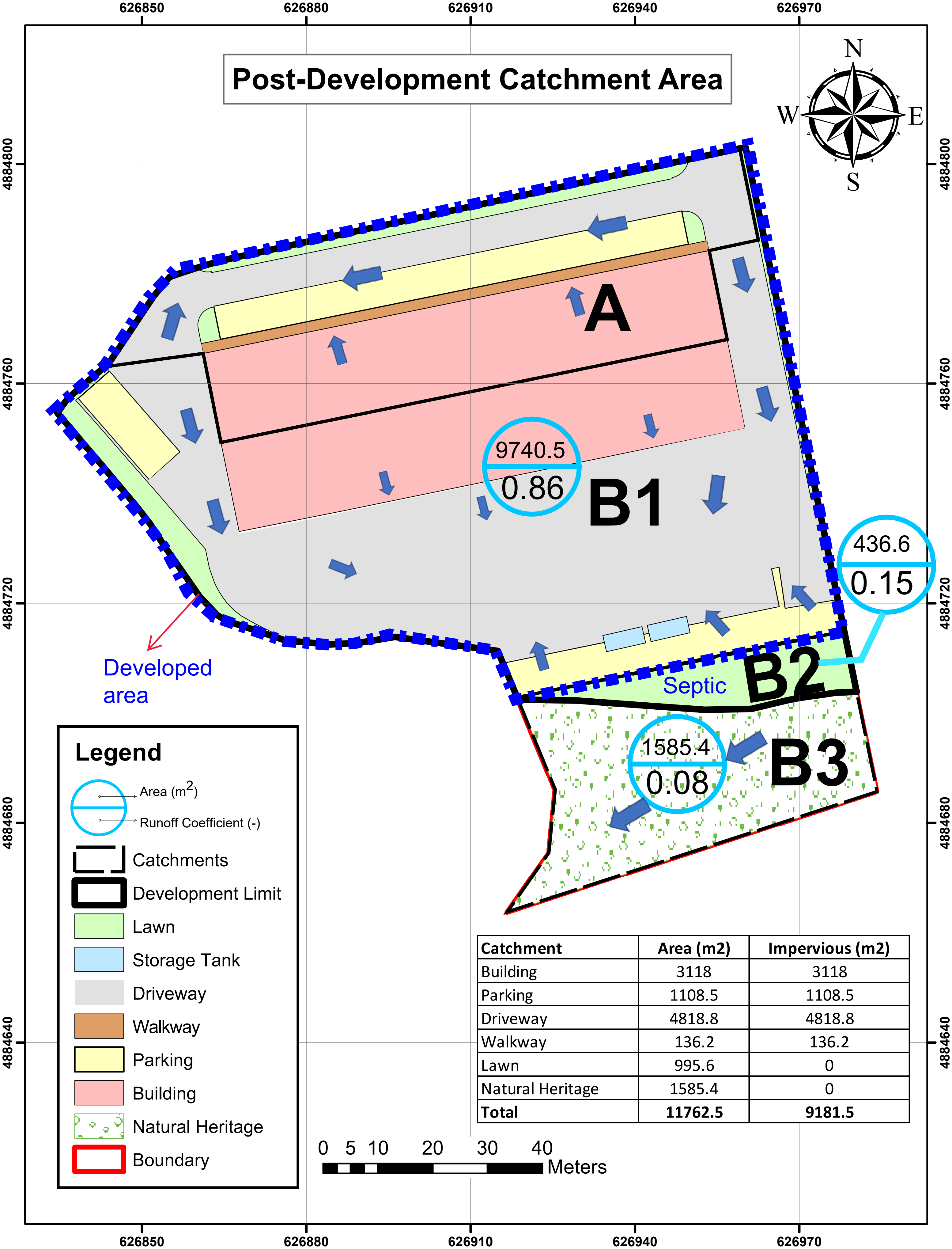
APPENDIX X –PRE- and POST-DEVELOPMENT CATCHMENT AREA


Catchments Area
N
W
E
S
Catchment A
Catchment B1
Catchment B2
Catchment B3
Legend
Catchment
Boundary
0 5 10 20 30 40
Meters
626850
626880
626910
626940
626970
4884840
4884800
4884760
4884720
4884680
4884640

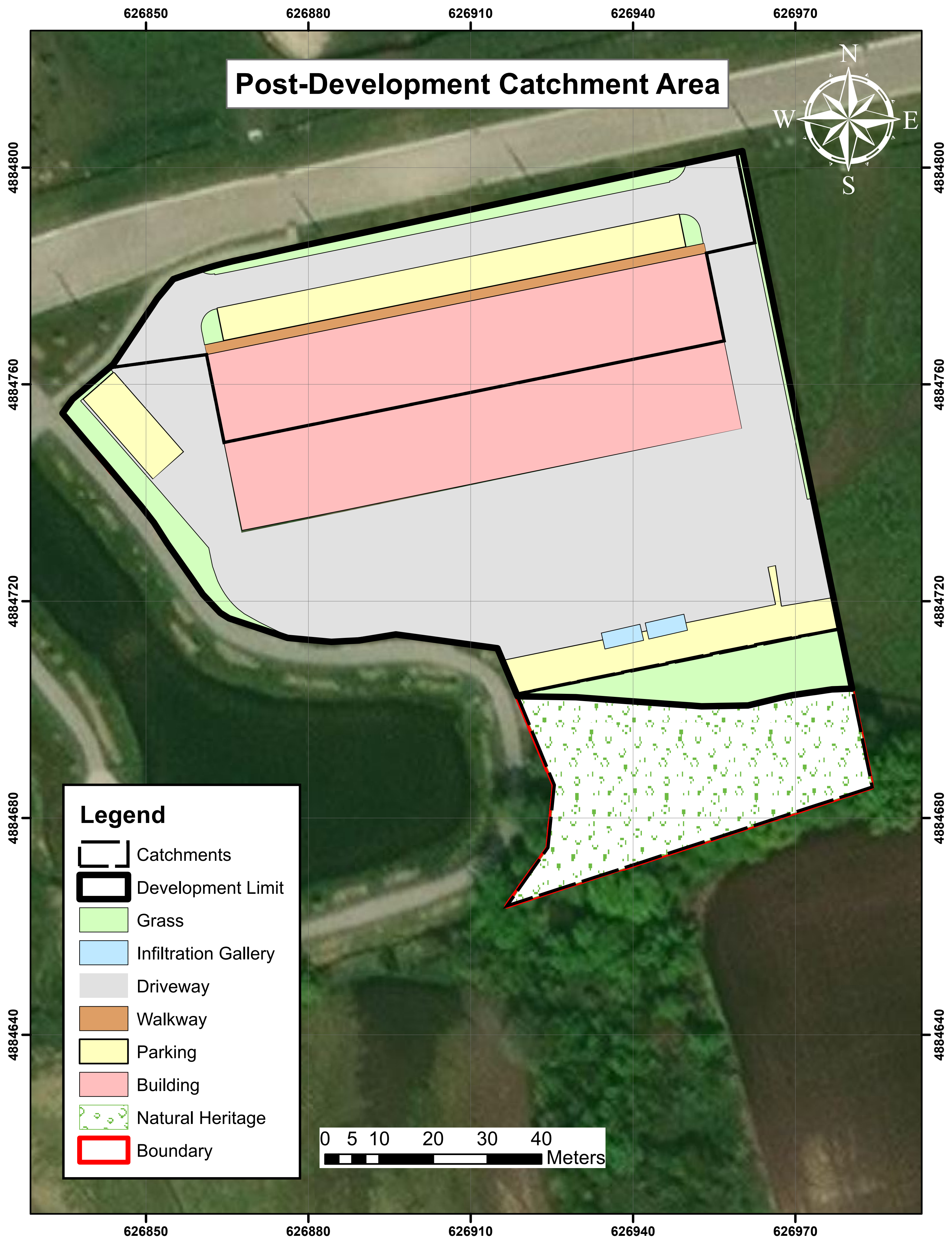


Catchment	Area (m2)	Impervious (m2)
Agriculture	10177.1	0
Natural Heritage	1585.4	0
Total	**11762.5**	**0**


Pre-Development Catchment Area
N
W
E
S
626850
626880
626910
626940
626970
4884800
4884760
4884720
4884680
4884640
Legend
Agriculture/Meadow
Natural Heritage
Boundary
0 5 10 20 30 40
Meters



Catchment	Area (m2)	Impervious (m2)
Building	3118	3118
Parking	1108.5	1108.5
Driveway	4818.8	4818.8
Walkway	136.2	136.2
Lawn	995.6	0
Natural Heritage	1585.4	0
Total	**11762.5**	**9181.5**


Post-Development Catchment Area
N
S
E
W
626850
626880
626910
626940
626970
4884800
4884760
4884720
4884680
4884640
Legend
Catchments
Development Limit
Grass
Infiltration Gallery
Driveway
Walkway
Parking
Building
Natural Heritage
Boundary
0 5 10 20 30 40
Meters

APPENDIX XI –WATER BALANCE CALCULATIONS

WATER BUDGET- PRE-DEVELOPMENT

WATER BALANCE/WATER BUDGET ASSESSMENT

Catchment Designation	Agriculture/Meadow	Total
Area (m^2)	10177	10177
Pervious Area (m^2)	10177	10177
Impervious Area (m^2)	0	0
Infiltration Factors		
Topography Infiltration Factor	0.3	
Soil Infiltratin Factor	0.2	
Land Cover Infiltration Factor	0.1	
MOE Infiltration Factor	0.6	
Actual Infiltratin Factor	0.6	
Run-Off Coefficient	0.4	
Runoff From Impervious Surfaces *	0	
Inputs (per unit area)		
Precipitation (mm/yr)	895	895
Run-On (mm/yr)	0	0
Other Inputs (mm/yr)	0	0
Total Inputs (mm/yr)	**895**	**895**
Outputs (per unit area)		
Precipitation Surplus (mm/yr)	243	243
Net Surplus (mm/yr)	243	243
Evapotranspiratin (mm/yr)	652	652
Infiltration (mm/yr)	145.8	146
Rooftop Infiltration (mm/yr)	0	0
Total Infiltration (mm/yr)	145.8	146
Runoff Pervious Area	97.2	97
Runoff Impervious Area	0.0	0
Total Runoff (mm/yr)	97.2	97
Total Outputs (mm/yr)	895	895
Difference (Inputs-Outputs)	0	0
Inputs (Volumes)		
Precipitaiton (m^3/yr)	9,109	9,109
Run-On (m^3/yr)	0	0
Other Inputs (m^3/yr)	0	0
Total Inputs (m^3/yr)	9,109	9,109
Outputs (Volumes)		
Precipitation Surplus (m^3/yr)	2,473	2,473
Net Surplus (m^3/yr)	2,473	2,473
Evapotranspiratin (m^3/yr)	6,635	6,635
Infiltration (m^3/yr)	1,484	1,484
Rooftop Infiltration (m^3/yr)	0	0
Total Infiltration (m^3/yr)	1,484	1,484
Runoff Pervious Area (m^3/yr)	989	989
Runoff Impervious Area (m^3/yr)	0	0
Total Runoff (m^3/yr)	989	989
Total Outputs (m^3/yr)	9,109	9,109
Difference (Inputs-Outputs)	0	0

WATER BUDGET- POST-DEVELOPMENT

WATER BALANCE/WATER BUDGET ASSESSMENT

Catchment Designation	Site			
	Building	Driveway/Parking/Walkway	Lawn	Total
Area (m^2)	3118	6064	996	10177
Pervious Area (m^2)	0	0	996	996
Impervious Area (m^2)	3115	6064	0	9179
Infiltration Factors				
Topography Infiltration Factor	0	0	0.3	
Soil Infiltratin Factor	0	0	0.2	
Land Cover Infiltration Factor	0	0	0.1	
MOE Infiltration Factor	0	0	0.6	
Actual Infiltratin Factor	0	0	0.6	
Run-Off Coefficient	1	1	0.4	
Runoff From Impervious Surfaces	0.9	0.9	0	
Inputs (per unit area)				
Precipitation (mm/yr)	895	895	895	895
Run-On (mm/yr)	0	0	0	0
Other Inputs (mm/yr)	0	0	0	0
Total Inputs (mm/yr)	**895**	**895**	**895**	**895**
Outputs (per unit area)				
Precipitation Surplus (mm/yr)	805.5	805.5	299	756
Net Surplus (mm/yr)	805.5	805.5	299	756
Evapotranspiratin (mm/yr)	89.5	89.5	596	139
Infiltration (mm/yr)	0	0	179.4	18
Rooftop Infiltration (mm/yr)	0	0	0	0
Total Infiltration (mm/yr)	0	0	179.4	18
Runoff Pervious Area	0	0	119.6	120
Runoff Impervious Area	805.5	805.5	0	806
Total Runoff (mm/yr)	805.5	805.5	119.6	738
Total Outputs (mm/yr)	895	895	895	895
Difference (Inputs-Outputs)	0	0	0	0
Inputs (Volumes)				
Precipitaiton (m^3/yr)	2,791	5,427	891	9,109
Run-On (m^3/yr)	0	0	0	0
Other Inputs (m^3/yr)	0	0	0	0
Total Inputs (m^3/yr)	2,791	5,427	891	9,109
Outputs (Volumes)				
Precipitation Surplus (m^3/yr)	2,512	4,884	298	7,693
Net Surplus (m^3/yr)	2,512	4,884	298	7,693
Evapotranspiratin (m^3/yr)	279	543	593	1,415
Infiltration (m^3/yr)	0	0	179	179
Rooftop Infiltration (m^3/yr)	0	0	0	0
Total Infiltration (m^3/yr)	0	0	179	179
Runoff Pervious Area (m^3/yr)	0	0	119	119
Runoff Impervious Area (m^3/yr)	2,509	4,884	0	7,393
Total Runoff (m^3/yr)	2,509	4,884	119	7,512
Total Outputs (m^3/yr)	2,791	5,427	891	9,109
Difference (Inputs-Outputs)	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 maxmimum ratio of 90% for both Asphalt Pavement and Rooftops in accordance with Township standards for 5 years return period.

WATER BUDGET- POST-DEVELOPMENT WITH MITIGATION

WATER BALANCE/WATER BUDGET ASSESSMENT

Catchment Designation	Site			
	Building	Driveway/Parking/Walkway	Grass	Total
Area (m^2)	3118	6064	996	10177
Pervious Area (m^2)	0	0	996	996
Impervious Area (m^2)	3115	6064	0	9179
Infiltration Factors				
Topography Infiltration Factor	0	0	0.3	
Soil Infiltratin Factor	0	0	0.2	
Land Cover Infiltration Factor	0	0	0.1	
MOE Infiltration Factor	0	0	0.6	
Actual Infiltratin Factor	0	0	0.6	
Run-Off Coefficient	1	1	0.4	
Runoff From Impervious Surfaces *	0.9	0.9	0	
Inputs (per unit area)				
Precipitation (mm/yr)	895	895	895	895
Run-On (mm/yr)	0	0	0	0
Other Inputs (mm/yr)	0	0	0	0
Total Inputs (mm/yr)	**895**	**895**	**895**	**895**
Outputs (per unit area)				
Precipitation Surplus (mm/yr)	805.5	805.5	299	756
Net Surplus (mm/yr)	805.5	805.5	299	756
Evapotranspiratin (mm/yr)	89.5	89.5	596	139
Infiltration (mm/yr)	0	805.50	179.4	497
Rooftop Infiltration (mm/yr)	805.5	0	0	247
Total Infiltration (mm/yr)	805.5	805.50	179.4	744
Runoff Pervious Area	0	0	119.6	120
Runoff Impervious Area	0	0.00	0	0
Total Runoff (mm/yr)	0	0.00	119.6	12
Total Outputs (mm/yr)	895	895	895	807
Difference (Inputs-Outputs)	0	0	0	0
Inputs (Volumes)				
Precipitaiton (m^3/yr)	2,791	5,427	891	9,109
Run-On (m^3/yr)	0	0	0	0
Other Inputs (m^3/yr)	0	0	0	0
Total Inputs (m^3/yr)	2,791	5,427	891	9,109
Outputs (Volumes)				
Precipitation Surplus (m^3/yr)	2,512	4,884	298	7,693
Net Surplus (m^3/yr)	2,512	4,884	298	7,693
Evapotranspiratin (m^3/yr)	279	543	593	1,415
Infiltration (m^3/yr)	0	4,884	179	5,063
Rooftop Infiltration (m^3/yr)	2,512	0	0	2,512
Total Infiltration (m^3/yr)	2,512	4,884	179	7,574
Runoff Pervious Area (m^3/yr)	0	0	119	119
Runoff Impervious Area (m^3/yr)	0	0	0	0
Total Runoff (m^3/yr)	0	0	119	119
Total Outputs (m^3/yr)	2,791	5,427	891	9,109
Difference (Inputs-Outputs)	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 maxmimum ratio of 90% for both Asphalt Pavement and Rooftops in accordance with Township standards for 5 years return period.

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)	9,109	9,109	0.0%	9,109	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)	**9,109**	9,109	0.0%	9,109	0.0%
Outputs (Volumes)					
Precipitation Surplus (m^3/yr)	2,473	7,693	211.1%	7,693	211.1%
Net Surplus (m^3/yr)	2,473	7,693	211.1%	7,693	211.1%
Evapotranspiratin (m^3/yr)	6,635	1,415	-78.7%	1,415	-78.7%
Infiltration (m^3/yr)	1,484	179	-88.0%	5,063	241.2%
Rooftop Infiltration (m^3/yr)	0	0	0.0%	2,512	0.0%
Total Infiltration (m^3/yr)	1,484	179	-88.0%	7,574	410.5%
Runoff Pervious Area (m^3/yr)	989	119	-88.0%	119	-88.0%
Runoff Impervious Area (m^3/yr)	0	7,393	0.0%	0	0.0%
Total Runoff (m^3/yr)	989	7,512	659.4%	119	-88.0%
Total Outputs (m^3/yr)	9,109	9,109	0.0%	9,109	0.0%

APPENDIX XII – PHOSPHRUS BUDGET CALCULATIONS

25 Pagewood Crt., East Gwillimbury

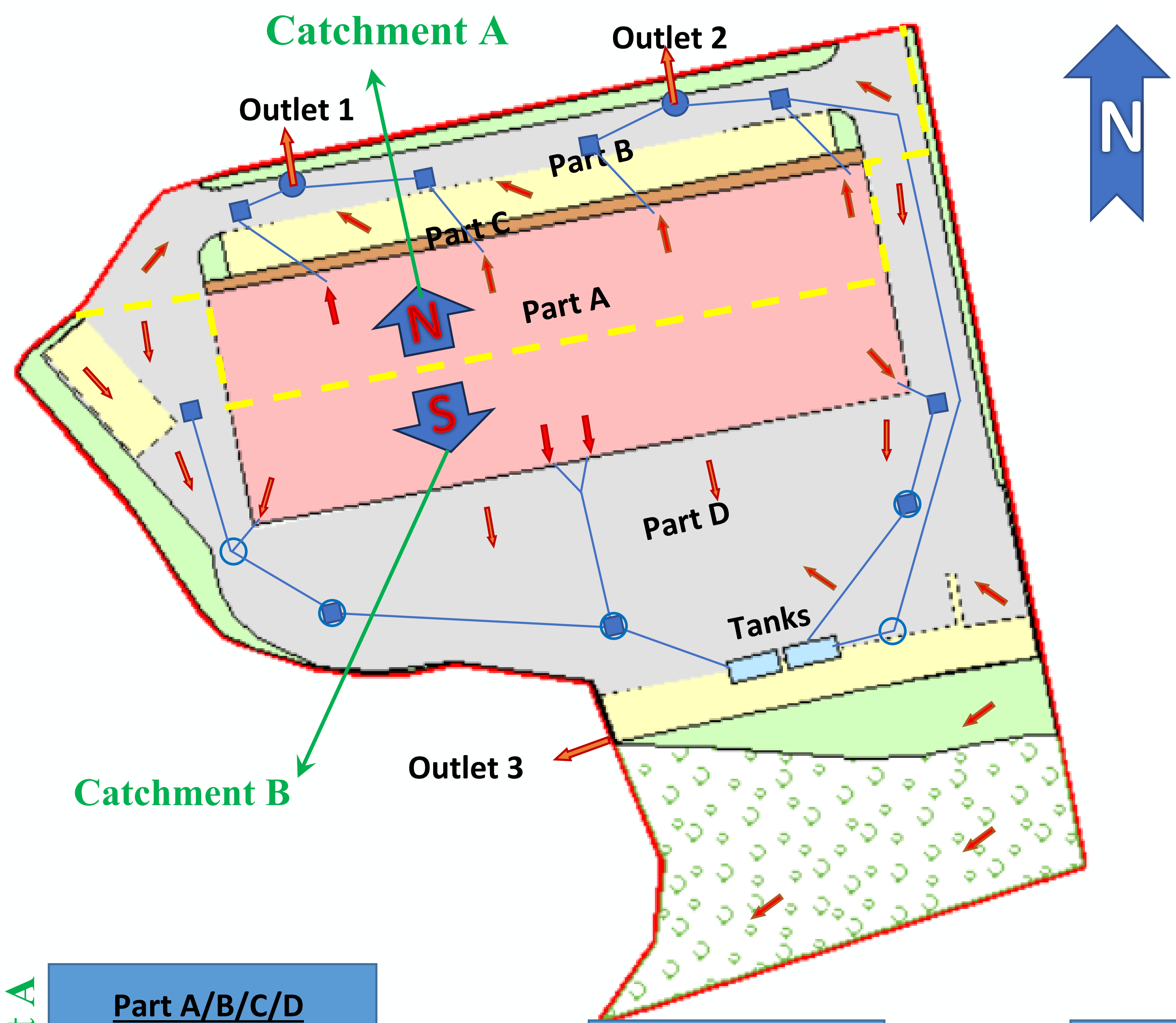
Phosphorus Budget Tool (MECP Hutchinson)

	Cropland	Commercial/Industrial	Low Intensity Development
Phosphorus Export (kg/ha/year)	0.23	1.82	0.17
Pre-Development Condition			
Total Area	Cropland	Commercial/Industrial	Low Intensity Development
Area (ha):	1.02	0.00	0.16
Total P (kg):	0.23	0.00	0.03
Total Pre-Development P (kg):		**0.26**	
Post-Development Condition (Uncontrolled)			
Total Area	Cropland	Commercial/Industrial	Low Intensity Development
Area (ha):	0.00	0.92	0.26
Total P (kg):	0.00	1.67	0.04
Total Post-Development P (kg):		**1.71**	
Post Development Condition (Controlled)			
Total Area	Cropland	Commercial/Industrial	Low Intensity Development
Area (ha):	0.00	0.92	0.06
Total P (kg):	0.00	1.67	0.01
Area Draining to Wet Pond			
Area (ha):	0.00	0.92	0.06
Total P (kg):	0.00	1.67	0.01
Wet Pond Treatment (63%)			
Total P to be Treated (kg):		1.67	0.01
Wet Pond Proficiency (%):		63	63
P Removed (kg):		1.05	0.01
P Remaining (kg):		0.62	0.004
Untreated Area			
Area (ha):	0.00	0.00	0.20
Total P (kg):	0.00	0.00	0.03
Total Post-Development P (kg) :		0.62+0.004+0.03	
	=	**0.66**	

Pre-Development:	**0.26**
Post-Development:	**1.71**
Change (Pre-Post):	**-1.45**
	558% Net Increase In Load
Post-Development (with BMPs):	**0.66**
Change (Pre-Post):	**-0.4**
	154% Net Increase in Load

APPENDIX XIII –SWM FLOWCHART

Stormwater Management Flowchart/Diagram



Site Impervious Areas Layout

Part A: Building

Part B: Parking

Part C: Walkway

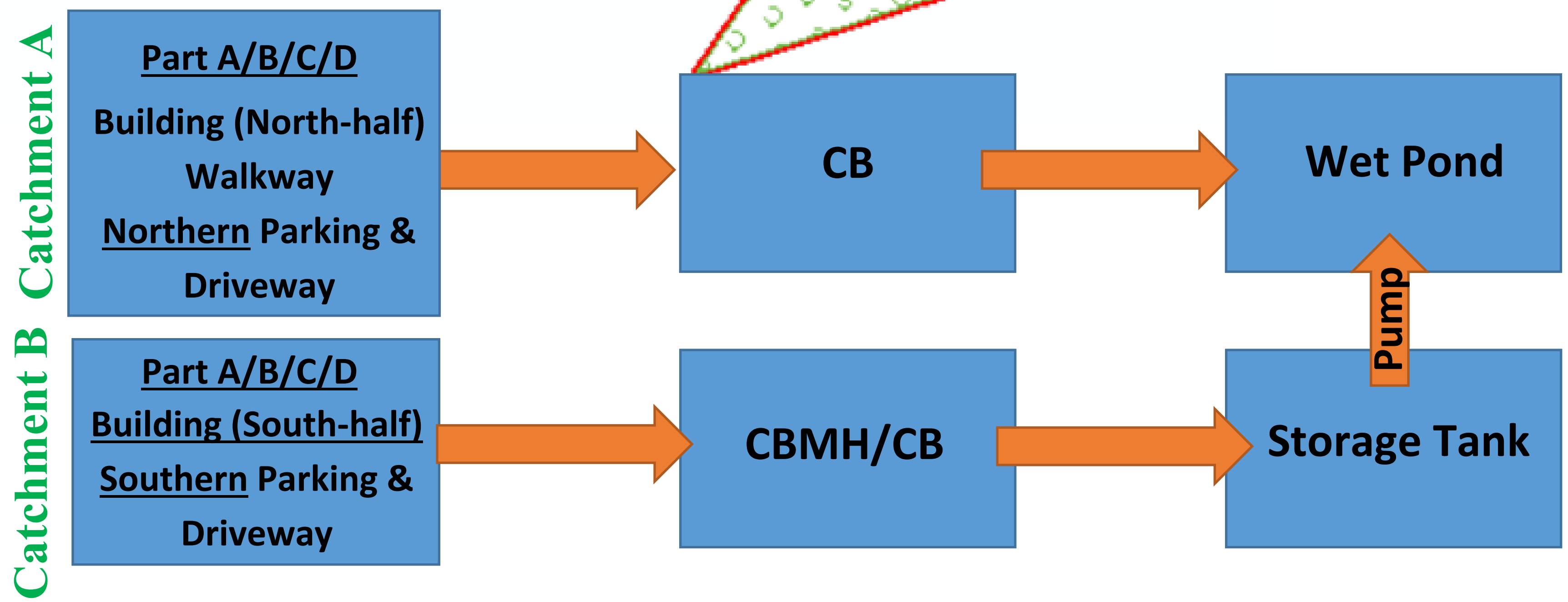
Part D: Driveway

Catchbasin (CB)

Manhole (MH)

Catchbasin Manhole (CBMH)

North/South Flow Direction Boundary



APPENDIX XIV – MECP APPROVED SUBDIVISION STORM SEWER & DRAINAGE PLANS

SITE LOCATION

SCALE: NTS

Town of East Gwillimbury

Our town, Our future

TOWN OF EAST GWILLIMBURY

PART OF LOT 10, CONCESSION 4

PROJECT NUMBER 2020

TOWN OF E.G. PROJECT # 19T-13002

PLANNING PROJECT #

THE REGIONAL MUNICIPALITY OF YORK
COMMUNITY PLANNING & DEVELOPMENT SERVICES

RECOMMENDED FOR MECP APPROVAL

FOR

	YES	NO
UNDERGROUND SERVICES	✓	
STORMWATER MANAGEMENT		✓
PUMPING STATION/FORCEMAIN		✓

SUBJECT TO RECOMMENDATION LETTER, DATED July 31, 2020

FILE NO. W03 EG.02.20 (MOE.20.E.0017)

MUNICIPALITY:

Town of East Gwillimbury
Our town, Our future

Town of East Gwillimbury
19000 Leslie Street
Sharon, ON
L0G 1V0
TEL: (905) 478-4282

DEVELOPER:

1625489 Ontario Inc.
3660 Midland Avenue, Suite 200
Toronto, ON
M1V 0B8

DEVELOPER'S ENGINEER:

SCS consulting group ltd

30 Centurian Drive, Suite 100
Markham, Ontario L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

LICENSED PROFESSIONAL ENGINEER
D. WOO
100073980
JULY 14, 2020
PROVINCE OF ONTARIO

REVIEWED
TOWN OF E.G. ENGINEERING DEPT

SIGNATURE DATE

File: P:\2020 Woodbine and Mt. Albert\Drawings\Phase 1\Base\2020D-BASE(MNT)-C3D.dwg - Revised by <JCASEY> : Tue, Jul 14 2020 - 3:36pm

GENERAL NOTES:

1. GENERAL NOTES

1.1 ALL SERVICES ARE TO BE TO THE TOWN OF EAST GWILLIMBURY ENGINEERING DEPARTMENT STANDARDS AND SPECIFICATIONS AND TO THE SATISFACTION OF THE TOWN.
1.2 LOCATIONS OF EXISTING SERVICES IS NOT GUARANTEED. THE CONTRACTOR IS TO NOTIFY UTILITY COMPANIES FORTY-EIGHT (48) HOURS PRIOR TO COMMENCEMENT OF ANY WORK.
1.3 FOR DIMENSIONS AND DETAILS NOT SHOWN, SEE STANDARD DRAWING REFERRED TO ON THE PROFILE.
1.4 ALL WORKS MUST BE CARRIED OUT ACCORDING TO THE OCCUPATIONAL HEALTH AND SAFETY ACT (UPDATED 2018), REGULATIONS FOR CONSTRUCTION PROJECTS AND ALL RELATED ONTARIO REGULATIONS APPLICABLE TO CONSTRUCTION ACTIVITY.
1.5 SEWER AND WATERMAIN TRENCHES SHALL BE BACKFILLED TO TOWN OF EAST GUILLIMBURY STANDARDS AND COMPACTED TO A MINIMUM OF 95% STANDARDS PROCTOR DENSITY.
1.6 ALL STANDARD DRAWINGS SHALL BE PER O.P.S.D. (MOST RECENT REVISION) UNLESS OTHERWISE SPECIFIED.

2. MEASUREMENTS

2.1 ALL DIMENSIONS SHALL BE IN METERS EXCEPT PIPE DIAMETER, WHICH IS IN MILLIMETRES, UNLESS OTHERWISE SPECIFIED.

3. ROADWORKS

3.1 COMPACTION: ROAD SUBGRADE TO BE COMPACTED TO MINIMUM 95% STANDARD PROCTOR DENSITY. GRANULAR MATERIALS ARE TO BE SPREAD AND COMPACTED IN 200 mm LAYERS TO A MINIMUM OF 100% STANDARD PROCTOR DENSITY. ASPHALT IS TO BE COMPACTED TO MINIMUM 96% STANDARD PROCTOR DENSITY.

ITEM	COMPACTION REQUIRED (% OF STANDARD PROCTOR DENSITY)
Granular "B"	Minimum 95%
Granular "A"	Minimum 100%
HL-6 or HL-8	Minimum 96%
HL-3	Minimum 96%

3.2 ROAD DESIGN - (MINIMUM)

ITEM	COMPACTED THICKNESS (LOCAL - INDUSTRIAL)
HL-3	40 mm
HL-6 or HL-8	80 mm
Granular "A"	150 mm
Granular "B"	450 mm

NOTE: ASPHALT AND GRANULAR THICKNESS IS SUITED FOR INDUSTRIAL/COMMERCIAL ROAD USE, COMPOSITION AND THICKNESS AS PER THE GEOTECHNICAL REPORT SUBJECT TO THE TOWN'S APPROVAL.

3.3 CURBS:
URBAN - TOWN STANDARDS OPSD 600.070 (TWO-STAGE CURB).
ESTATE RESIDENTIAL - TOWN STANDARD OPSD 600.100.
3.5 SIDEWALKS TO BE CONSTRUCTED AS PER OPSD 310.010. INTERSECTIONS OF CURBS AND SIDEWALKS SHALL BE DEPRESSED AS PER STANDARD OPSD 310.030.
3.6 SUB-DRAINS ARE TO BE INSTALLED THROUGHOUT UNLESS OTHERWISE APPROVED.
3.7 NO MANHOLE COVERS WILL BE PERMITTED TO BE CONSTRUCTED IN ANY PART OF THE SIDEWALK.
3.8 ALL NEW SIGNS WILL BE TYPE IV HIGH REFLECTIVITY SIGNS, MADE OF STEEL AND WILL INCLUDE THE TOWN NAME AND YEAR OF MANUFACTURE ON THE BORDER OF THE SIGN. SIGN RETRO-REFLECTIVITY IS DETERMINED USING THE MUTCD TABLE 2A.3 (THE STANDARDS IN THE OTM REFER TO NEW SIGNS RETRO-REFLECTIVITY).

4. STORM SEWERS

4.1 ALL CONCRETE PIPE SHALL HAVE SEALED JOINTS WITH GASKETS AND PIPE CLASS AS SHOWN ON DRAWINGS.
4.2 ALL PVC GRAVITY SEWER PIPE SHALL CONFORM TO ASTM SPEC. D-3034-C SDR-35 WITH "LOCK-IN" RUBBER SEALING RING.
4.3 MANHOLES:
4.3.1 MANHOLES SHALL BE AS PER STANDARD DRAWINGS OPS 701.010 TO 701.015 (INCLUSIVE).
4.3.2 ALL STORM MANHOLES TO BE BENCHED THROUGHOUT TO THE CROWN OF ALL PIPES ON A VERTICAL PROJECTION FROM SPRING LINE, AS PER STANDARD DRAWINGS, EXCEPT AS OTHERWISE NOTED.
4.4 SEWER BEDDING SHALL BE TO STANDARD DRAWING OPSD 802.030 CLASS "B" BEDDING OR AS APPROVED BY THE TOWN.
4.5 CATCHBASINS:
4.5.1 CATCHBASIN FRAMES AND GRATES SHALL BE AS PER STANDARD DRAWINGS (OPSD 400.010 OR OPSD 610.010).
4.5.2 LEADS FOR A SINGLE CATCHBASIN SHALL BE 250 mm AND FOR A DOUBLE CATCHBASIN 300 mm.
4.5.3 ALL CATCHBASINS SHALL BE CONNECTED TO THE STORM SEWER BY TEES WHERE POSSIBLE. STANDARD DRAWINGS OPSD 708.010 AND 708.030.
4.6 ALL STORM OUTFALLS THAT EMPTY INTO A DITCH OR WATERCOURSE MUST BLEND WITH THE FLOW OF THE SAME.
4.7 ALL PVC JOINTS AT MANHOLES SHALL BE CONSTRUCTED BY MEANS OF A PVC MANHOLE ADAPTER.
4.8 STORM SERVICE CONNECTION SIZE AND MATERIAL AS SPECIFIED IN THE ENGINEERING DRAWINGS, C/W 1200mmØ INSPECTION MANHOLE AS PER OPSD 701.010. SERVICE TO EXTEND FROM STORM MAIN TO INSPECTION MANHOLE. INSPECTION MANHOLE TO BE LOCATED ON PRIVATE PROPERTY 1.5m FROM PROPERTY LINE.

5. WATERMAIN - FOR CONCEPTUAL FUTURE SERVICING

5.1 WATERMAIN PIPE SHALL BE PVC C900 (THICK WALL PIPE). PIPES IS TO BE WRAPPED WITH STRAND 14-GAUGE STRAND COPPER WIRE AND WIRE IS TO BE BROUGHT TO GRADE AT ALL MAINLINE VALVES AND HYDRANT SECONDARY VALVES, AND A HOLE DRILLED SIX INCHES (6") DOWN FROM UPPER SECTION AND WIRE INSERTED THROUGH THIS HOLE FOR PROTECTION. TOP OF WATERMAIN SHALL BE MINIMUM 1.7 m BELOW CENTERLINE OF ROAD GRADE. ALL SPLICES ARE TO BE DONE ABOVE GRADE OR USING A MOISTURE-PROOF SEAL.
5.2 HYDRANTS AND VALVES SHALL BE PER TOWN STANDARDS DRAWING NO. OPSD 1105.01. ALL HYDRANTS ARE TO BE SELF-DRAINING (UNLESS IN AREAS WITH HIGH WATER TABLE). ALL HYDRANTS ARE TO BE EQUIPPED WITH ONE (1) FOUR-INCH (4") PUMPER PORT WITH MANUFACTURER'S "STORTZ" FITTING. TOWN-APPROVED HYDRANTS ARE CANADA VALVE (CanVal) - ONLY.
5.3 SERVICES:
5.3.1 ALL COMMERCIAL/INDUSTRIAL SERVICE CONNECTIONS SHALL BRANCH INTO SEPARATE DOMESTIC AND FIRE LINES IN A VALVE CHAMBER LOCATED IN THE BOULEVARD, AS PER TOWN STANDARD "FIRE & COMMERCIAL SERVICE VALVE CHAMBER" DETAIL. EACH LINE SHALL HAVE INDIVIDUAL SHUTOFF VALVES.
5.4 ALL SERVICE CONNECTION STUBS SHALL BE MARKED WITH 50 mm X 100 mm X 2.4 m STAKES, PAINTED BLUE FOR WATER.
5.5 ALL CURB STOPS, MAIN STOPS AND COUPLINGS ARE TO BE COMPRESSION-TYPE FITTINGS, I.E. CAMBRIDGE SUCCESSOR BALL VALVE TYPE, WHICH MUST BE APPROVED BY THE TOWN C/W STAINLESS STEEL RODS AND BRASS PIN.
5.6 ALL BENDS AND TEES SHALL BE OPSD 1103.01 AND 1103.02 AND BLOCKED TO UNDISTURBED GROUND.
5.7 WHERE THE TOWN APPROVES WATERMAIN CONSTRUCTION WITH LESS THAN THE ABOVE NOTED MINIMUM COVER, THE WATERMAIN SHALL BE INSULATED TO THE TOWN'S SATISFACTION.
5.8 ALL MECHANICAL CONNECTIONS SHALL BE PROTECTED AGAINST CORROSION THROUGH THE USE OF CORROSION PROTECTION DURATION NUTS. NUTS SHALL BE USED ON 50% OF ALL T-BOLTS PER CONNECTION AND ARE TO BE USED IN ADDITION TO STANDARD FASTENING NUTS, NOT IN PLACE OF STANDARD NUTS.

6. SANITARY SEWERS - FOR CONCEPTUAL FUTURE SERVICING

6.1 PIPE:
6.1.1 ALL PVC GRAVITY SEWER PIPE SHALL CONFORM TO CSA SPECIFICATION B182.1 OR B182.2 (OR MOST RECENT REVISION) DR 35 WITH "LOCK IN" RUBBER SEALING RING.
6.1.2 ALL COMMERCIAL/INDUSTRIAL SERVICES SHALL BE CONNECTED TO SEWER WITH TEES. PIPE: 125 mm PVC, C/W 1200mmØ INSPECTION MANHOLE AS PER OPSD 701.010. SERVICE TO EXTEND FROM SANITARY MAIN INSPECTION MANHOLE. INSPECTION MANHOLE TO BE LOCATED ON PRIVATE PROPERTY 1.5m FROM PROPERTY LINE
6.1.3 ALL SEWER CONNECTIONS TO MANHOLES SHALL BE CONSTRUCTED BY MEANS OF A PVC MANHOLE ADAPTER.
6.1.4 THE BEDDING MATERIAL SHALL EXTEND TO 300 mm ABOVE THE PIPE AND COMPACTION TESTS ARE REQUIRED BEFORE THE TRENCH IS BACKFILLED. BACKFILL SHALL BE COMPACTED TO MINIMUM 95% STANDARD PROCTOR DENSITY.
6.2 MANHOLES:
6.2.1 MANHOLES SHALL BE TO STANDARD DRAWINGS OPSD 701.010 TO 701.015 (INCLUSIVE).
6.2.2 ALL SANITARY MANHOLES SHALL BE BENCHED THROUGHOUT TO THE SPRING LINE, AS PER STANDARD DRAWINGS, EXCEPT AS OTHERWISE NOTED.
6.2.3 ALL SANITARY MANHOLES SHALL HAVE MONOLITHIC PRE-BENCHED BASES WITH PRE-MANUFACTURED CONNECTIONS.
6.2.4 ALL SANITARY MANHOLES CONSTRUCTED IN THE VICINITY OF LOW POINTS OR OUTSIDE OF THE PAVED ROADWAY SHALL HAVE WATERTIGHT COVERS. ALL MANHOLES LOCATED IN CUL-DE-SACS SHALL HAVE WATERTIGHT COVERS.
6.3 SANITARY SEWER BEDDING SHALL BE TO STANDARD DRAWING OPSD 802.03 CLASS "B" (UNLESS OTHERWISE NOTED AND APPROVED).
6.4 LATERALS - ALL LATERALS SHALL BE CONSTRUCTED ACCORDING TO STANDARD DRAWINGS OPSD 1006.01 AND 1006.02.

7. GRADING

7.1. ALL GRADES AND SWALES SHALL BE A MAXIMUM 5%.
7.2. DRIVEWAY GRADES SHALL BE A MINIMUM 2% AND MAXIMUM 8%.
7.3. WHERE SLOPES EXCEED 5%, 3:1 SLOPES SHALL BE USED TO MAKE UP DIFFERENCE WHEN THE HEIGHT DIFFERENCE IS LESS THAN 1.0m. WHERE THE HEIGHT DIFFERENCE IS GREATER THAN 1.0m, 4:1 SLOPES SHALL BE USED.
7.4. THE MINIMUM TOPSOIL DEPTH FOR ALL LOTS AND BOULEVARDS SHALL BE 150mm.
7.5. DRIVEWAYS SHALL BE A MINIMUM 1.2m CLEAR DISTANCE FROM ALL STREET HARDWARE (POLES, HYDRANTS, CATCHBASINS, UTILITY PEDESTALS, ETC.).
7.6. SWALE DEPTH TO BE ACCORDING TO FLOW - MINIMUM 150mm.

8. SURVEY MONUMENT INFORMATION

RECORD OF MONUMENTS TO BE CONSTRUCTED WITH THE PROJECT:

SURVEY MONUMENT NO. 1: 1 (SIB)

LOCATION: NORTHWEST CORNER OF BLOCK 6

ELEVATION: 263.64 (TOP OF BAR)

UTM COORDINATES: N4884853.56, E626764.32

SURVEY MONUMENT NO. 2: 3 (SIB)

LOCATION: SOUTHWEST CORNER OF BLOCK 3

ELEVATION: 266.00 (TOP OF BAR)

UTM COORDINATES: N4884729.70, E627139.84

PROJECT SPECIFIC NOTES:

CONSTRUCTION PHASING

1. TEMPORARY CONSTRUCTION ACCESS AS PER YORK REGION DETAIL (REFER TO DRAWING ESC-2).
2. ESC MEASURES AS PER PLANS HEREIN.
3. ROAD CONSTRUCTION WITH DAYLIGHTING TO EXISTING (NO DETAILED SITE PLAN BLOCK GRADING) AS PER ESC PLANS.
4. INDIVIDUAL SITE PLAN APPLICATIONS FOR EACH BLOCK, WITH DETAILED DESIGN AND CONFORMITY EXERCISE TO OVERALL APPROVED LOT GRADING.
5. IMPLEMENTATION OF SITE PLAN DESIGN, INCLUDING EACH LOT DAYLIGHTING ONTO ADJACENT LOTS ON AN AS-NEEDED BASIS (TO BE ALLOWED FOR WITHIN THE AGREEMENTS OF PURCHASE AND SALE).

DRAWING LIST

GN-1	GENERAL NOTES
ESC-1	EROSION CONTROL PLAN
ESC-2	EROSION CONTROL DETAILS
STM-1	STORM DRAINAGE PLAN
GSP-1	GENERAL SERVICING PLAN
CFSP-1	CONCEPTUAL FUTURE SERVICE PLAN
CUP-1	COMPOSITE UTILITY PLAN
LG-1	LOT GRADING PLAN 1
LG-2	LOT GRADING PLAN 2
PP-1	PLAN AND PROFILE – PAGEWOOD COURT (STA 0+000.00 TO 0+318.70)
PND-1	POND PLAN
PND-2	POND DETAILS
SWM-1	PRE-DEVELOPMENT STORMWATER DRAINAGE PLAN
SWM-2	POST-DEVELOPMENT STORMWATER DRAINAGE PLAN
D-1	DETAILS 1
D-2	DETAILS 2
D-3	DETAILS 3
DS-1	5-YEAR STORM DESIGN SHEET

REGION DRAWINGS FOR WOODBINE AVENUE

REGN-1	EXISTING PAVEMENT MARKING REMOVALS
REGN-2	INTERSECTION IMPROVEMENTS 1 (STA 0+000.00 TO 0+240.00)
REGN-3	INTERSECTION IMPROVEMENTS 2 (STA 0+240.00 TO 0+550.00)
REGN-4	PAVEMENT ELEVATIONS
REGN-5	PAVEMENT MARKINGS
REGN-6	ROAD CROSS SECTIONS 1
REGN-7	ROAD CROSS SECTIONS 2
REGN-8	CONSTRUCTION/TRAFFIC MANAGEMENT PLAN 1
REGN-9	REGION OF YORK GENERAL NOTES

CONSTRUCTION TRAFFIC MANAGEMENT PLANS FOR WOODBINE AVENUE IMPROVEMENTS (PREPARED BY JD NORTHCOTE ENGINEERING)

CTMP1	CONSTRUCTION/TRAFFIC MANAGEMENT PLAN – SOUTHBOUND LANE RECONSTRUCTION
CTMP2	CONSTRUCTION/TRAFFIC MANAGEMENT PLAN – NORTHBOUND LANE RECONSTRUCTION

STREETSCAPE, POND AND CHANNEL, LANDSCAPE DRAWINGS (PREPARED BY TERRAPLAN)

L-1	POND & CHANNEL LANDSCAPE PLAN-PLANT KEYS FOR TREES & AQUATICS
L-2	POND & CHANNEL LANDSCAPE PLAN-PLANT KEYS FOR SHRUBS & LIVE STAKES
L-3	POND & CHANNEL SEED MIX PLAN
L-4	STREET TREE & BUFFER AREA LANDSCAPE PLAN
L-5	LANDSCAPE DETAILS

STREET LIGHT DRAWINGS (PREPARED BY MQ ENERGY)

SL-01	STREET LIGHT PLAN
SL-02	STREET LIGHT PLAN DETAILS

CHANNEL DRAWINGS (PREPARED BY GEOMORPHIX)

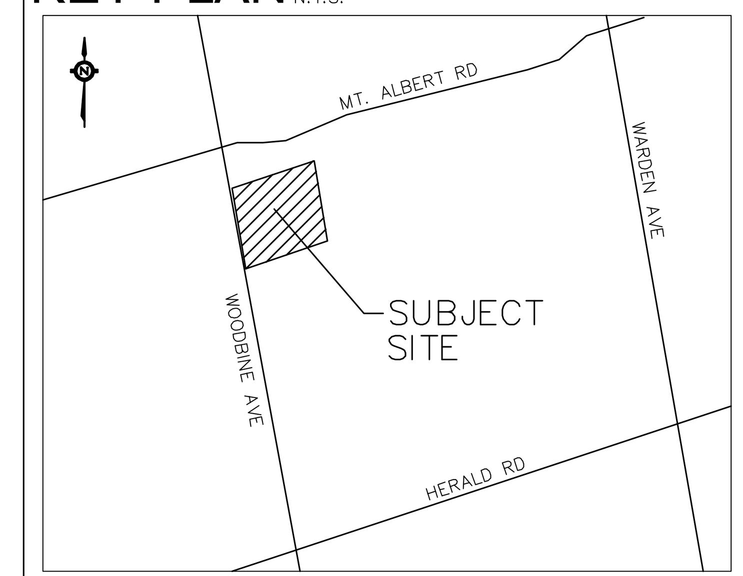
GEO-1	CONCEPTUAL CHANNEL DESIGN PLANFORM AND PROFILE PLAN
GEO-2	CONCEPTUAL CHANNEL DESIGN CROSS SECTIONS
DET-1	CONCEPTUAL CHANNEL DESIGN CROSS SECTIONS
DESC-1	CHANNEL RESTORATION DESIGN PHASING AND EROSION AND SEDIMENT CONTROL

THE REGIONAL MUNICIPALITY OF YORK
COMMUNITY PLANNING & DEVELOPMENT SERVICES
RECOMMENDED FOR MECP APPROVAL
FOR

	YES	NO
UNDERGROUND SERVICES	✓	
STORMWATER MANAGEMENT		✓
PUMPING STATION/FORCEMAIN		✓

SUBJECT TO RECOMMENDATION LETTER, DATED July 31, 2020
FILE NO. W03 EG.02.20 (MOE.20.E.0017)

KEY PLAN N.T.S.



SURVEY NOTES:

1. TOPOGRAPHICAL SURVEY INFORMATION PROVIDED KRCMAR SURVEYORS LTD. ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48-100 HAVING AN ELEVATION OF 271.230 METRES.
2. LEGAL INFORMATION PROVIDED BY WESTON CONSULTING.

GENERAL NOTES:

1. DO NOT SCALE DRAWINGS.
2. ALL DIMENSIONS ARE TO BE CHECKED AND VERIFIED ON SITE PRIOR TO CONSTRUCTION.
3. ANY DISCREPANCIES, OMISSIONS, OR ERRORS ARE TO BE REPORTED TO THE CONSULTANT. NO WORK IS TO PROCEED BEFORE CLARIFICATION OF THE DISCREPANCIES, ERRORS, OR OMISSIONS IS RECEIVED FROM THE CONSULTANT.
4. ALL EXISTING CONDITIONS TO BE CHECKED AND VERIFIED ON SITE PRIOR TO CONSTRUCTION.
5. ONLY LATEST APPROVED DRAWINGS TO BE USED FOR CONSTRUCTION.
6. ALL DRAWINGS AND SPECIFICATIONS REMAIN THE PROPERTY OF THE CONSULTANT, AND SHALL BE RETURNED TO THE CONSULTANT UPON REQUEST. IN NO WAY SHALL THE DRAWINGS AND/OR SPECIFICATIONS IN WHOLE OR IN PART BE REPRODUCED OR DISTRIBUTED WITHOUT THE PERMISSION OF THE CONSULTANT.
7. PRIOR TO ANY WORKS ON MUNICIPAL PROPERTY A ROAD OCCUPANCY PERMIT SHALL BE OBTAINED FROM THE TOWN OF EAST GWILLIMBURY.

LEGEND:

ITEM	DESCRIPTION	DATE	BY	APPRV'D
4	3rd REGIONAL SUBMISSION	APR. 20/20		
3	3rd SUBMISSION	MAR. 08/19		
2	2nd SUBMISSION	AUG. 22/16		
1	1st SUBMISSION	NOV. 13/15		

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER
D. WOO
100073980
JULY 16, 2020
PROVINCE OF ONTARIO

ENGINEER'S SEAL

PROJECT: PART OF LOT 10
CONCESSION 4
TOWN OF EAST GWILLIMBURY
REGIONAL MUNICIPALITY OF YORK

TITLE: GENERAL NOTES

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

Town of East Gwillimbury
Our town, Our future

DRAWN:	SCALE: N.T.S.	DWG. No. GN-1	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: JUL 2020	SHEET No. 2 OF 19	

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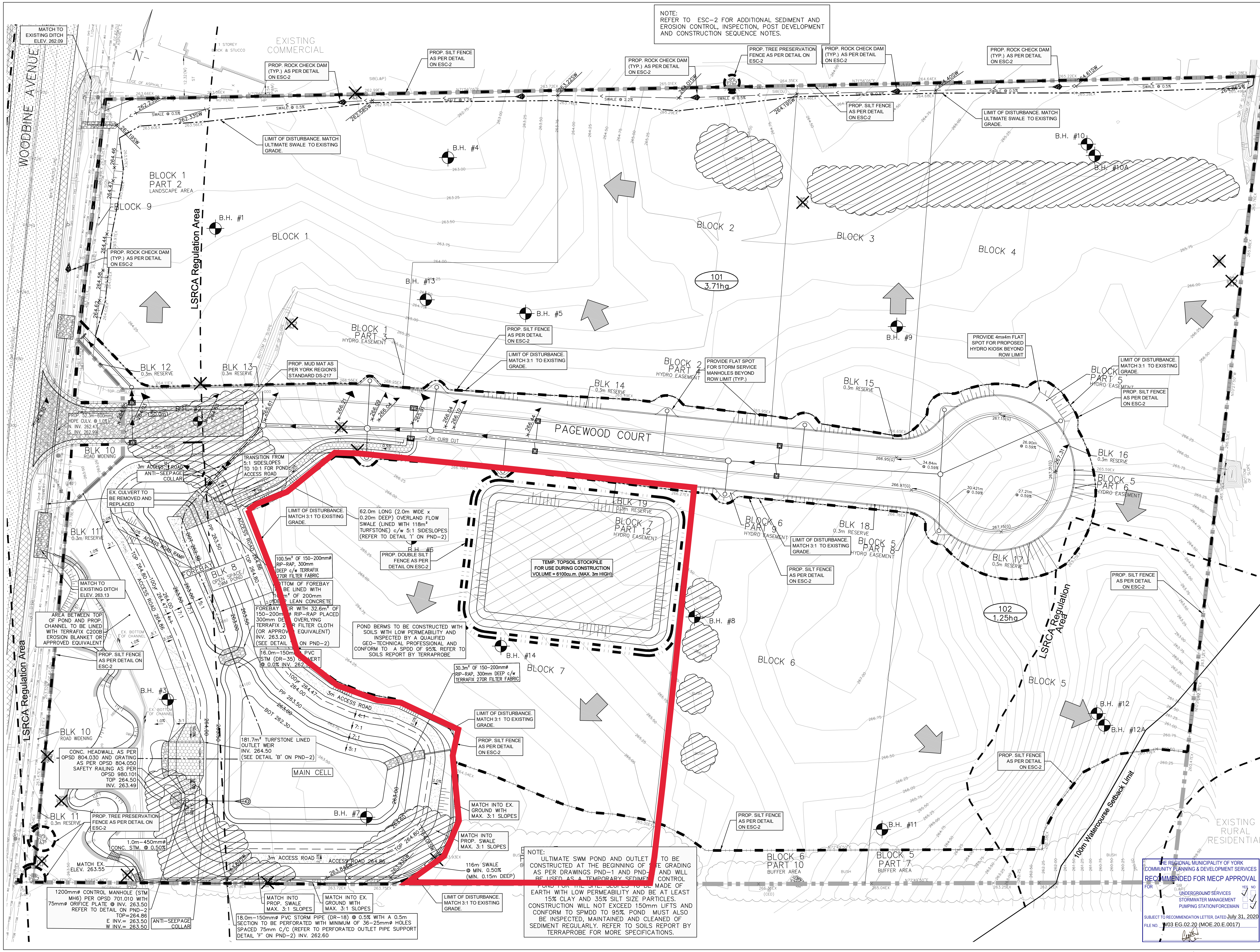
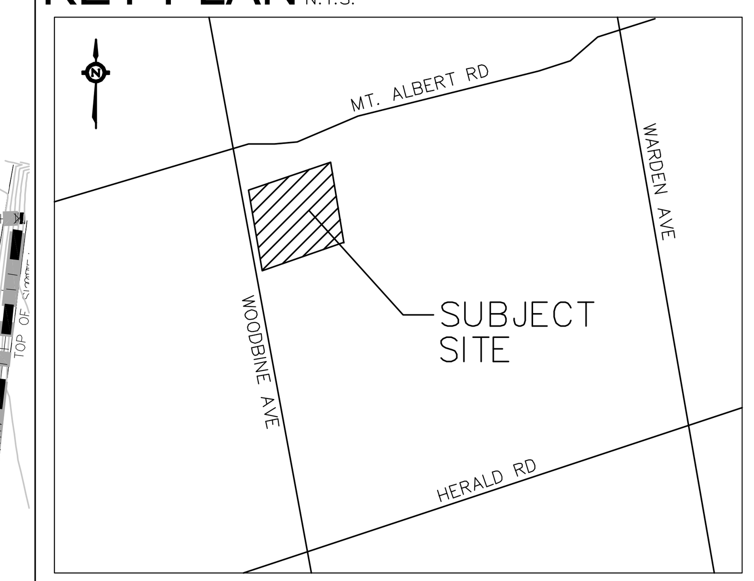


KEY PLAN N.T.S.



SURVEY NOTES:

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2. LEGAL INFORMATION PROVIDED BY WESTON CONSULTING.

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LEGEND:

- SUBJECT PROPERTY LINE
- 14 / BH — LOT NUMBER / BOREHOLE
- LIMIT OF DISTURBANCE
- SILT FENCE AS PER LSRCA DETAIL ON DWG ESC-2
- DOUBLE SILT FENCE
- TREE PRESERVATION FENCE (SEE DETAIL ON DRAWING ESC-2
- LSRCA REGULATION AREA
- CONCEPTUAL LOT LINE
- 240.00 — EXISTING CONTOUR
- MUD MAT
- SWALE @ 0.5% — TEMPORARY SWALE
- 291.14SW — PROPOSED SWALE ELEVATION
- ×291.14 — PROPOSED ELEVATION
- ×291.19EX — EXISTING ELEVATION
- EXISTING TREES TO BE REMOVED
- TEMPORARY STREET CATCHBASIN SEDIMENT CONTROL DEVICE (SEE DETAIL ON DRAWING ESC-2)
- ROCK CHECK DAM (SEE DETAIL ON DRAWING ESC-2)
- EXISTING OVERLAND FLOW DIRECTION

ITEM	DESCRIPTION	DATE	BY	APPR'VD
4	3rd REGIONAL SUBMISSION	APR. 20/20		
3	3rd SUBMISSION	MAR. 08/19		
2	2nd SUBMISSION	AUG. 22/16		
1	1st SUBMISSION	NOV. 13/15		

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER D. WOO 100073980 JULY 16, 2020 PROVINCE OF ONTARIO

ENGINEER'S SEAL

PROJECT: PART OF LOT 10 CONCESSION 4 TOWN OF EAST GWILLIMBURY REGIONAL MUNICIPALITY OF YORK

TITLE: EROSION AND SEDIMENT CONTROL

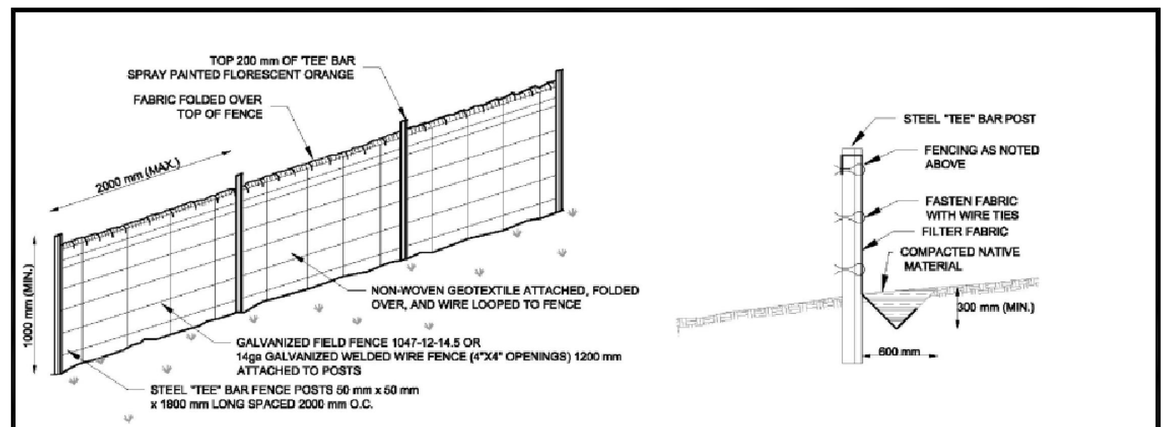
SCS consulting group ltd

30 CENTURIAN DRIVE, SUITE 100 MARKHAM, ONTARIO L3R 8B8 TEL: (905) 475-1900 FAX: (905) 475-8335

Town of East Gwillimbury

DRAWN:	SCALE: 1:500	DWG. No. ESC-1	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: JUL 2020	SHEET No. 3 OF 19	

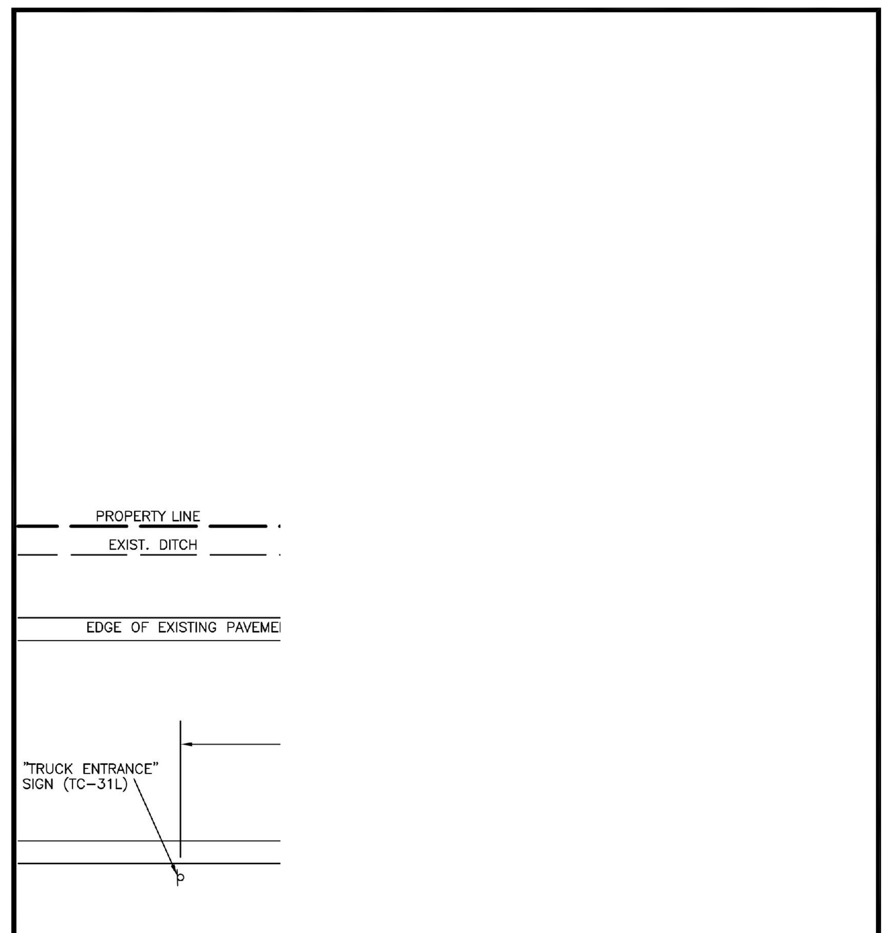
File: P:\2020 Woodbine and Mt. Albert\Drawings\Phase 1\Base\2020D-BASE(MNT)-C3D.dwg - Revised by <JCASEY> : Tue, Jul 14 2020 - 3:36pm



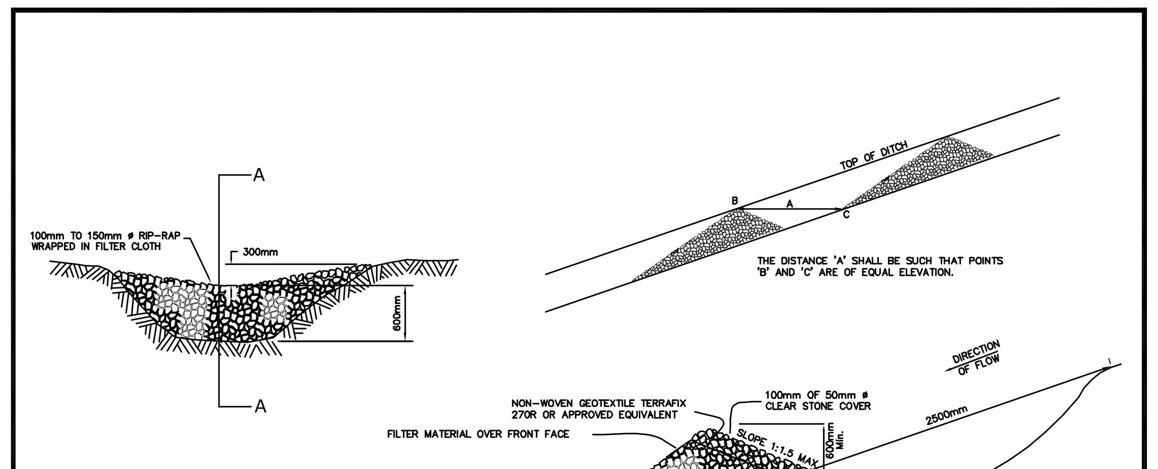
NOTES:

1. SEDIMENT CONTROL FENCE SHOULD BE ALIGNED WITH CONTOURS FOR SHEET OVERLAND FLOW.
2. SEDIMENT CONTROL FENCE IS TO BE LOCATED IN AREAS OF LOW SEDIMENT YIELD ON SLOPES THAT CONFORM TO MTO DRAINAGE MANUAL VOLUME 2 CHART F4-3C TOPOGRAPHIC FACTOR LS BASED ON SLOPE LENGTH AND GRADIENT.
3. SEDIMENT CONTROL FENCE SHALL BE INSTALLED WITH FILTER MEDIA FABRIC TIED INTO THE SOIL A MINIMUM OF 300 mm BY EITHER STATIC SLICING OR TRENCH METHODS WITH COMPACTION OF TRENCH MATERIAL MEETING 95% STANDARD PROCTOR DENSITY.
4. STEEL 'T' BAR POSTS ARE TO BE SPACED A MAXIMUM DISTANCE OF 2000 mm ON CENTER.
5. FROZEN GROUND CONDITIONS REQUIRE FILTER FABRIC TO BE BACKFILLED IN TRENCH WITH CLEAR STONE.
6. GEOTEXTILE FABRIC TO BE COMPRISED OF NON-WOVEN U.V. STABILIZED MATERIAL FABRIC TO BE FOLDED OVER TOP OF FENCE A MINIMUM OF 300 mm AND WIRE FASTENED.
7. CONSTRUCTION OPERATIONS SHALL BE CARRIED OUT IN SUCH A MANNER THAT EROSION AND WATER POLLUTION IS MINIMIZED.
8. ALL DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE SHOWN.

NO.	REVISION	DATE		
1	SWM GUIDELINES UPDATE	06.2016	SEDIMENT CONTROL FENCE	DATE: 06.2016 SCALE: NTS LSRCA ESC-4



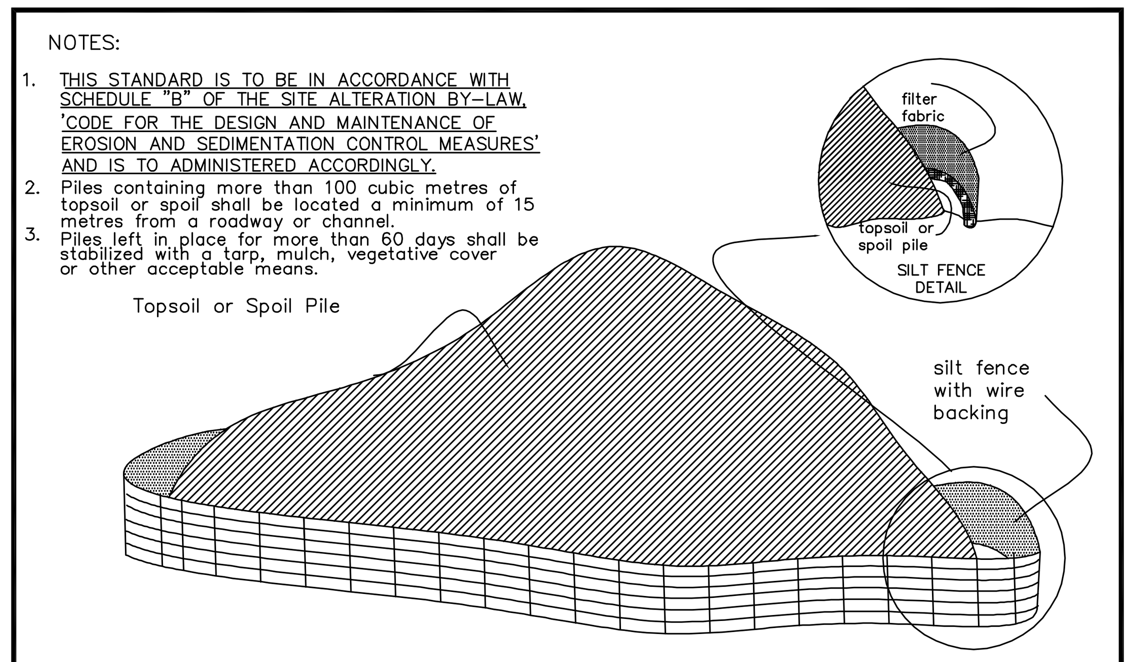
NOTES
1. ALL DIMENSIONS ARE IN mm U NOTED.
2. TEMPORARY "TRUCK ENTRANCE" BE INSTALLED ON THE SHOULDE OF THE ACCESS (NOTE: SEE DE THESE SIGNS IN THE ONTARIO BOOK 7.) THE APPLICANT WILL FOR THE COST OF OBTAINING, I MAINTAINING THESE SIGNS.
3. TEMPORARY CONSTRUCTION ACC REMOVED FROM THE REGIONAL AND ALL DISTURBED AREAS SH TO ORIGINAL OR BETTER CONDI



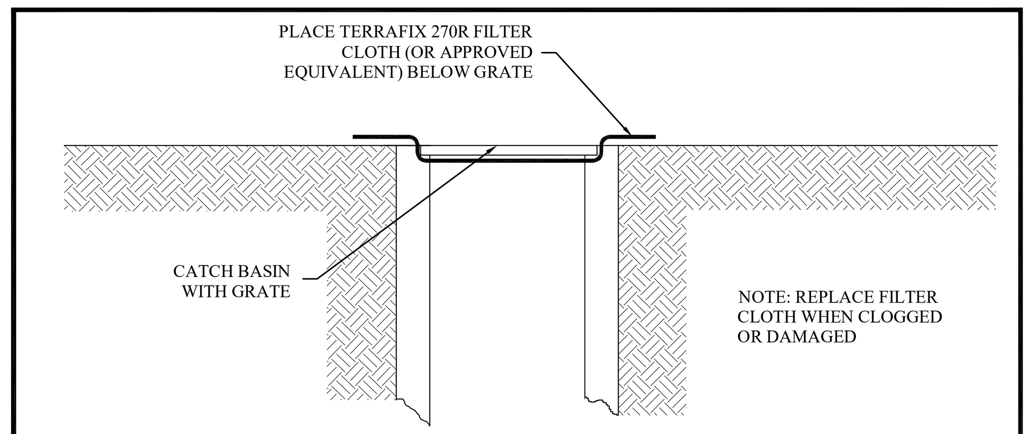
NOTES:

TYPICAL GRANULAR EROSION CONTROL DEVICE	NO.	REVISION	APR'D	DATE	DRAWN: A.S.C	SCALE: N.T.S

NOTES:
1. THIS STANDARD IS TO BE IN ACCORDANCE WITH SCHEDULE "B" OF THE SITE ALTERATION BY-LAW, 'CODE FOR THE DESIGN AND MAINTENANCE OF EROSION AND SEDIMENTATION CONTROL MEASURES' AND IS TO ADMINISTERED ACCORDINGLY.
2. Piles containing more than 100 cubic metres of topsoil or spoil shall be located a minimum of 15 metres from a roadway or channel.
3. Piles left in place for more than 60 days shall be stabilized with a tarp, mulch, vegetative cover or other acceptable means.

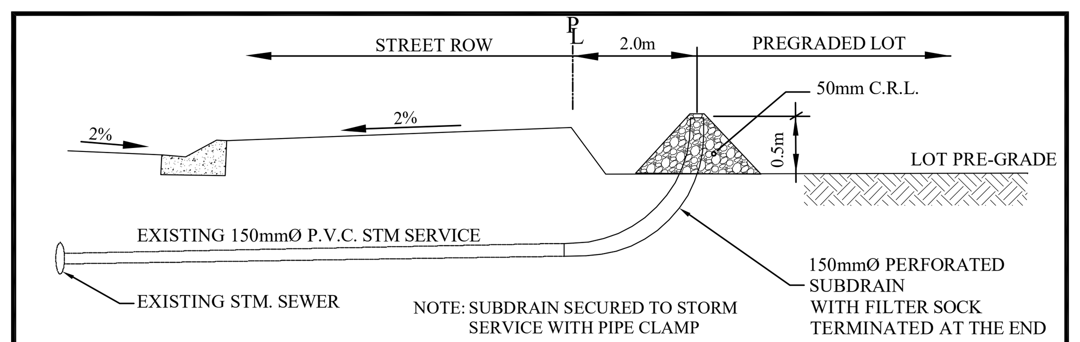


TOPSOIL OR SPOIL PILE SILTATION CONTROL	NO.	REVISION	APR'D	DATE	DRAWN: A.S.C	SCALE: N.T.S



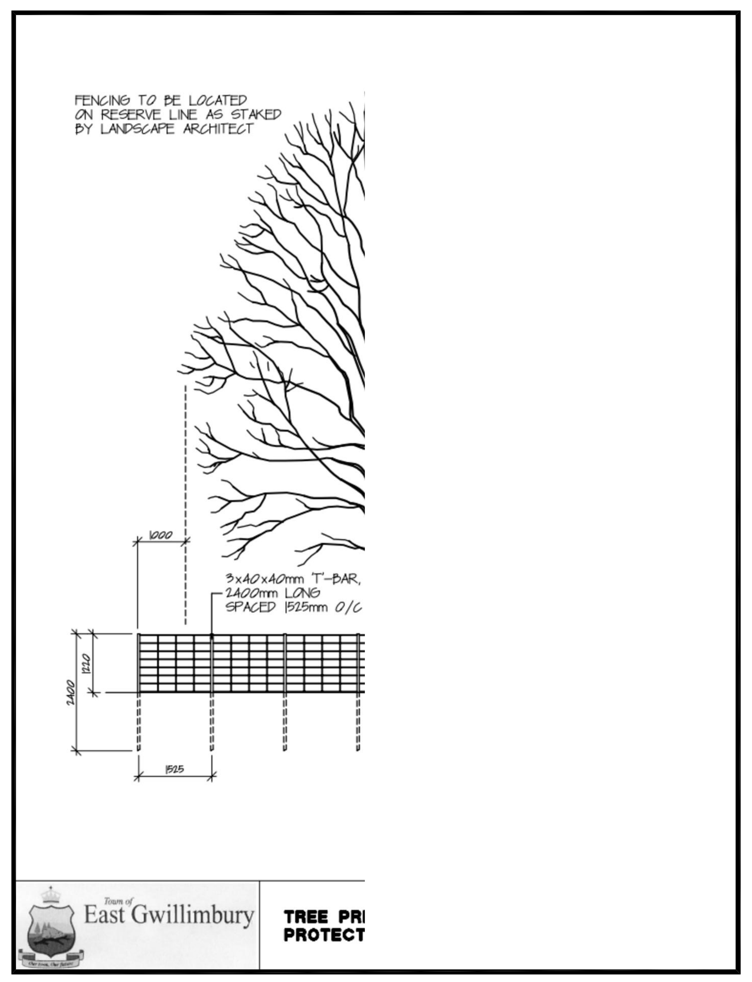
TEMPORARY STREET CATCHBASIN SEDIMENT CONTROL DEVICE

SCALE: N.T.S.



TEMPORARY LOT RISER DRAIN WITH FILTER SOCK

SCALE: N.T.S.



GENERAL NOTES:

1. THE OWNER IS RESPONSIBLE FOR OBTAINING ALL NECESSARY APPROVALS FROM THE MUNICIPALITY AND ANY REQUIRED EXTERNAL AGENCIES PRIOR TO ANY SITE ALTERATION ACTIVITY.
2. PRIOR TO COMMENCEMENT OF ANY SITE ALTERATION ACTIVITY, EROSION & SEDIMENT CONTROL (ESC) MEASURES, AS PER APPROVED EROSION SEDIMENT CONTROL PLAN, MUST BE INSTALLED. ADDITIONAL ESC MEASURES, IF REQUIRED, SHALL BE INSTALLED AS DIRECTED BY THE MUNICIPALITY. THE ESC MEASURES SHALL REMAIN IN PLACE UNTIL DIRECTED BY THE MUNICIPALITY/CONSERVATION AUTHORITY/CIVIL CONSULTANT FOR THEIR REMOVAL.
3. NO CONSTRUCTION ACTIVITY OR MACHINERY SHALL BE ALLOWED BEYOND THE SEDIMENT CONTROL OR LIMITS OF THE SUBDIVISION.
4. THE CONTRACTOR IS RESPONSIBLE TO IMPLEMENT DUST CONTROL MEASURES AND CONSTRUCTION PRACTICE GUIDELINES AS APPROVED BY THE MUNICIPALITY/CONSERVATION AUTHORITY.
5. THE CONTRACTOR IS RESPONSIBLE FOR MAINTAINING ALL ESC MEASURES IN WORKING CONDITIONS AT ALL TIMES TO THE SATISFACTION OF THE MUNICIPALITY/REGION/CONSERVATION AUTHORITY. THE CONTRACTOR SHALL ROUTINELY INSPECT ALL ESC DEVICES AT A MINIMUM OF ONCE A WEEK AND AFTER EACH SIGNIFICANT SNOW MELT AND RAINFALL GREATER THAN 10MM TO ENSURE THAT ESC MEASURES ARE IN PROPER WORKING CONDITION. DURING INACTIVE PERIODS, WHERE THE SITE HAS LITTLE OR NO CONSTRUCTION ACTIVITY FOR 30 DAYS OR LONGER, A MONTHLY INSPECTION WILL BE CONDUCTED. ANY DAMAGES MUST BE REPAIRED WITHIN 48 HOURS.
6. ALL CONSTRUCTION VEHICLES MUST ENTER AND EXIT THE SITE ONLY FROM THE APPROVED ACCESS ROUTE AS SHOWN. TEMPORARY CONSTRUCTION ACCESS WILL BE MAINTAINED TO THE SATISFACTION OF THE MUNICIPALITY/REGION/CONSERVATION AUTHORITY. STREET SWEEPING MAY BE REQUIRED AS NEEDED.
7. ALL DISTURBED GROUND LEFT INACTIVE, OR TO BE LEFT INACTIVE, FOR OVER 30 DAYS SHALL BE STABILIZED AS SOON AS POSSIBLE, SUBJECT TO WEATHER CONDITIONS, BY SEEDING (SUCH AS ANNUAL RYE) OR APPROVED EQUIVALENT TO THE SATISFACTION OF THE MUNICIPALITY/REGION/CONSERVATION AUTHORITY.
8. ALL TOPSOIL STOCKPILES SHALL BE SURROUNDED WITH TEMPORARY SEDIMENT CONTROL. THE MAXIMUM SIDE SLOPES FOR STOCKPILES SHALL BE 1.5 (H) TO 1.0 (V). THE MAXIMUM HEIGHT OF STOCKPILE SHOULD NOT EXCEED 3 m.
9. THE EROSION & SEDIMENT CONTROL STRATEGIES OUTLINED ON THIS PLAN ARE NOT STATIC AND MAY NEED TO BE UPGRADED/AMENDED AS SITE CONDITIONS CHANGE TO PREVENT SEDIMENT RELEASES TO THE NATURAL ENVIRONMENT.
10. CONTRACTOR TO CONFIRM THE LOCATION OF THE EXISTING UTILITIES PRIOR TO INITIATING ANY ON-SITE WORKS.
11. TREES ARE TO BE PRESERVED OR REMOVED AS PER THE APPROVED TREE PRESERVATION PLAN PREPARED BY KUNTZ FORESTRY CONSULTING INC.. TREE RELOCATIONS WILL TAKE PLACE BEFORE STRIPPING COMMENCES.
12. EROSION AND SEDIMENT CONTROL INSPECTION IS TO BE CONDUCTED BY A QUALIFIED AND/OR CERTIFIED PROFESSIONAL (E.G. CAN-CISEC) AND REPORTS ARE TO BE SUBMITTED TO THE MUNICIPALITY'S DEVELOPMENT INSPECTOR AND ANY OTHER RELEVANT STAFF AS REQUIRED BY THE MUNICIPALITY/CONSERVATION AUTHORITY.
13. CONTRACTOR TO ENSURE THAT SEDIMENT LADEN RUNOFF IS DIRECTED TO APPROPRIATE ESC CONTROLS THROUGHOUT THE STAGED CONSTRUCTION PROCESS, INCLUDING TRANSITIONAL WORKS BETWEEN STAGES.
14. ALL ACTIVITIES, INCLUDING MAINTENANCE PROCEDURES, WILL BE CONTROLLED TO PREVENT THE ENTRY OF PETROLEUM PRODUCTS, DEBRIS, RUBBLE, CONCRETE OR OTHER DELETERIOUS SUBSTANCES INTO THE DOWNSTREAM WATERCOURSE. REFUELING AND MAINTENANCE WILL BE CONDUCTED 30 M FROM ANY WATERCOURSE.

ESC STAGE 1 - SITE PREPERATION & TOPSOIL STRIPPING NOTES

1. ALL TEMPORARY SEDIMENT CONTROLS (E.G. FENCING, CHECK DAMS AND CONSTRUCTION ACCESS) TO BE INSTALLED AND INSPECTED BY THE MUNICIPALITY/CONSERVATION AUTHORITY PRIOR TO ANY TOPSOIL STRIPPING WORKS. ULTIMATE SWM POND CONSTRUCTION TO BE STARTED COINCIDENT WITH INITIAL TOPSOIL STRIPPING NECESSARY TO CONSTRUCT THE POND.
2. INSTALL ULTIMATE SWM FACILITY OUTLET AS PER DETAIL ON DRAWING PND-1 AND PND-2.
3. SEDIMENTS COLLECTED IN THE SWM POND SHALL BE REMOVED WHEN 50% OF THE STORAGE CAPACITY IS FILLED.
4. PROPOSED SWM POND TO BE STABILIZED WITH 50MM OF FILTREXX GROWTH MEDIA EROSION CONTROL BLANKET (OR APPROVED EQUIVALENT), ABOVE THE NORMAL WATER LEVEL WITHIN 48 HOURS FOLLOWING ESC POND CONSTRUCTION.
5. CONTRACTOR TO COORDINATE WITH CIVIL CONSULTANT TO EXPLORE THE POSSIBILITY OF A STAGED STRIPPING AND EARTHWORKS APPROACH TO MINIMIZE THE AREA DISTURBED AT ONE TIME.
6. TEMPORARY INTERCEPTOR SWALES ALONG THE SITE PERIMETER TO BE SEEDED (SUCH AS ANNUAL RYE) AND STABILIZED WITH EROSION CONTROL BLANKETS (LAYFIELD LPS-1, 1" OF FILTREXX GROWTH MEDIA OR APPROVED EQUIVALENT) WITHIN 48 HOURS FOLLOWING SWALE GRADING.
7. SURFACE ROUGHENING TO TAKE PLACE WHEREVER POSSIBLE AS A MEANS OF TEMPORARY EROSION AND SEDIMENT CONTROL. SURFACE ROUGHENING TO BE ACHIEVED BY TRACKWALKING PERPENDICULAR TO THE DIRECTION OF FLOW OR WITH A SHEEP'S FOOT PACKER.
8. CONTRACTOR TO ENSURE THAT SEDIMENT LADEN RUNOFF IS DIRECTED TO APPROPRIATE ESC CONTROLS THROUGHOUT THE STAGED CONSTRUCTION PROCESS, INCLUDING TRANSITIONAL WORKS BETWEEN STAGES.

ESC STAGE 2 - CUT AND FILL OPERATION NOTES

1. SEDIMENT COLLECTED IN THE SWM POND SHALL BE REMOVED WHEN 50% OF THE STORAGE CAPACITY IS FILLED.
2. PROPOSED SWM POND TO BE STABILIZED WITH 50MM OF FILTREXX GROWTH MEDIA EROSION CONTROL BLANKET (OR APPROVED EQUIVALENT) ABOVE THE NORMAL WATER LEVEL WITHIN 48 HOURS FOLLOWING POND CONSTRUCTION.
3. SEDIMENT COLLECTED IN THE TEMPORARY SEDIMENT TRAPS SHALL BE REMOVED WHEN 50% OF THE STORAGE CAPACITY IS FILLED. THE TRAPS SHALL BE KEPT IN OPERATION UNTIL THE DRAINAGE AREA IS REDUCED TO 0.5 HA OR LESS.
4. CONTRACTOR TO COORDINATE WITH CONSULTANT TO EXPLORE THE POSSIBILITY OF A STAGED STRIPPING AND EARTHWORKS APPROACH TO MINIMIZE THE AREA DISTURBED AT ONE TIME.
5. TEMPORARY INTERCEPTOR SWALES ALONG THE SITE PERIMETER TO BE SEEDED (SUCH AS ANNUAL RYE) AND STABILIZED WITH EROSION CONTROL BLANKETS (LAYFIELD LPS-1 OR APPROVED EQUIVALENT) WITHIN 48 HOURS FOLLOWING SWALE GRADING.
6. SURFACE ROUGHENING TO TAKE PLACE WHEREVER POSSIBLE AS A MEANS OF TEMPORARY EROSION AND SEDIMENT CONTROL. SURFACE ROUGHENING TO BE ACHIEVED BY TRACKWALKING PERPENDICULAR TO THE DIRECTION OF FLOW OR WITH A SHEEP'S FOOT PACKER.

ESC STAGE 3 - UNDERGROUND SERVICES & ROAD CONSTRUCTION NOTES

1. CATCHBASIN SEDIMENT CONTROL DEVICES FOR STREET CATCHBASINS TO BE INSTALLED IMMEDIATELY AFTER BASE ASPHALT (DETAIL ON THIS DRAWING).
2. TEMPORARY RISER DRAINS WITH FILTER SOCKS TO SERVICE CONNECTIONS ARE TO BE PROVIDED AS REQUIRED, TO DRAIN PREGRADED LOTS WHICH WILL REMAIN UNTIL THE BUILDING PROGRAM FOR THE ASSOCIATED LOTS IS INITIATED (SEE DETAIL ON THIS DRAWING).
3. STREET SWEEPING/CATCHBASIN CLEANING PROGRAM SHALL BE IMPLEMENTED TO THE SATISFACTION OF THE MUNICIPALITY. THIS SHALL BE THE RESPONSIBILITY OF THE DEVELOPER UNTIL ASSUMPTION.

SITE ALTERATION SEQUENCE

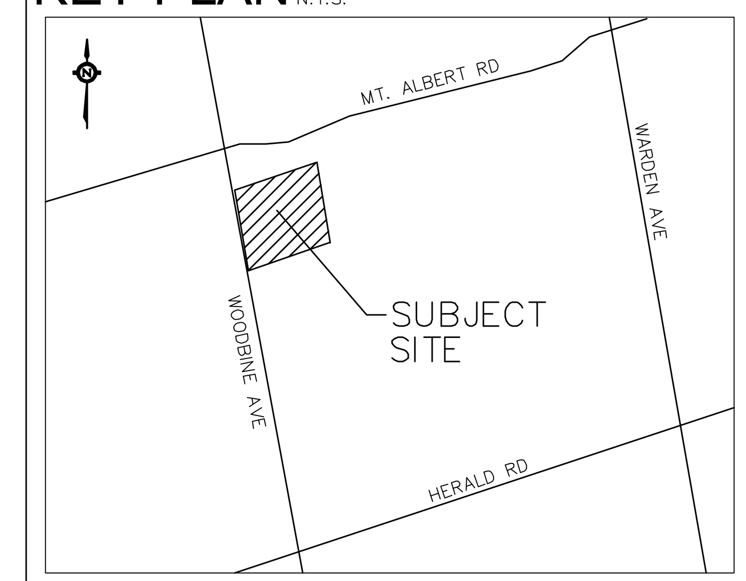
1. TREE PRESERVATION FENCE INSTALLATION.
2. SILT FENCE INSTALLATION AS PER BSD 21-A, 23-A AND 23-E.
3. MUD MAT INSTALLATION AS PER BSD 23-D.
4. TOPOIL REMOVAL AND CUT/FILL OPERATIONS.
5. SWM POND CONSTRUCTION.
6. UNDERGROUND SERVICING INCLUDING STORM SEWER.
7. ROAD CONSTRUCTION.

THE REGIONAL MUNICIPALITY OF YORK
COMMUNITY PLANNING & DEVELOPMENT SERVICES
RECOMMENDED FOR MECP APPROVAL
FOR

	YES	NO
UNDERGROUND SERVICES	✓	
STORMWATER MANAGEMENT		✓
PUMPING STATION/FORCEMAIN		✓

SUBJECT TO RECOMMENDATION LETTER, DATED July 31, 2020
FILE NO. W03 EG.02.20 (MOE.20.E.0017)

KEY PLAN N.T.S.



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LEGEND:

ITEM	DESCRIPTION	DATE	BY	APPRV'D
4	3rd REGIONAL SUBMISSION	APR. 20/20		
3	3rd SUBMISSION	MAR. 08/19		
2	2nd SUBMISSION	AUG. 22/16		
1	1st SUBMISSION	NOV. 13/15		

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER
D. WOO
100073980
JULY 16, 2020
PROVINCE OF ONTARIO

ENGINEER'S SEAL

PROJECT: PART OF LOT 10
CONCESSION 4
TOWN OF EAST GWILLIMBURY
REGIONAL MUNICIPALITY OF YORK

TITLE: EROSION AND SEDIMENT CONTROL

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475–1900
FAX: (905) 475–8335

Town of East Gwillimbury

DRAWN:	SCALE: N.T.S.	DWG. No. ESC-2	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: JUL 2020	SHEET No. 4 OF 19	

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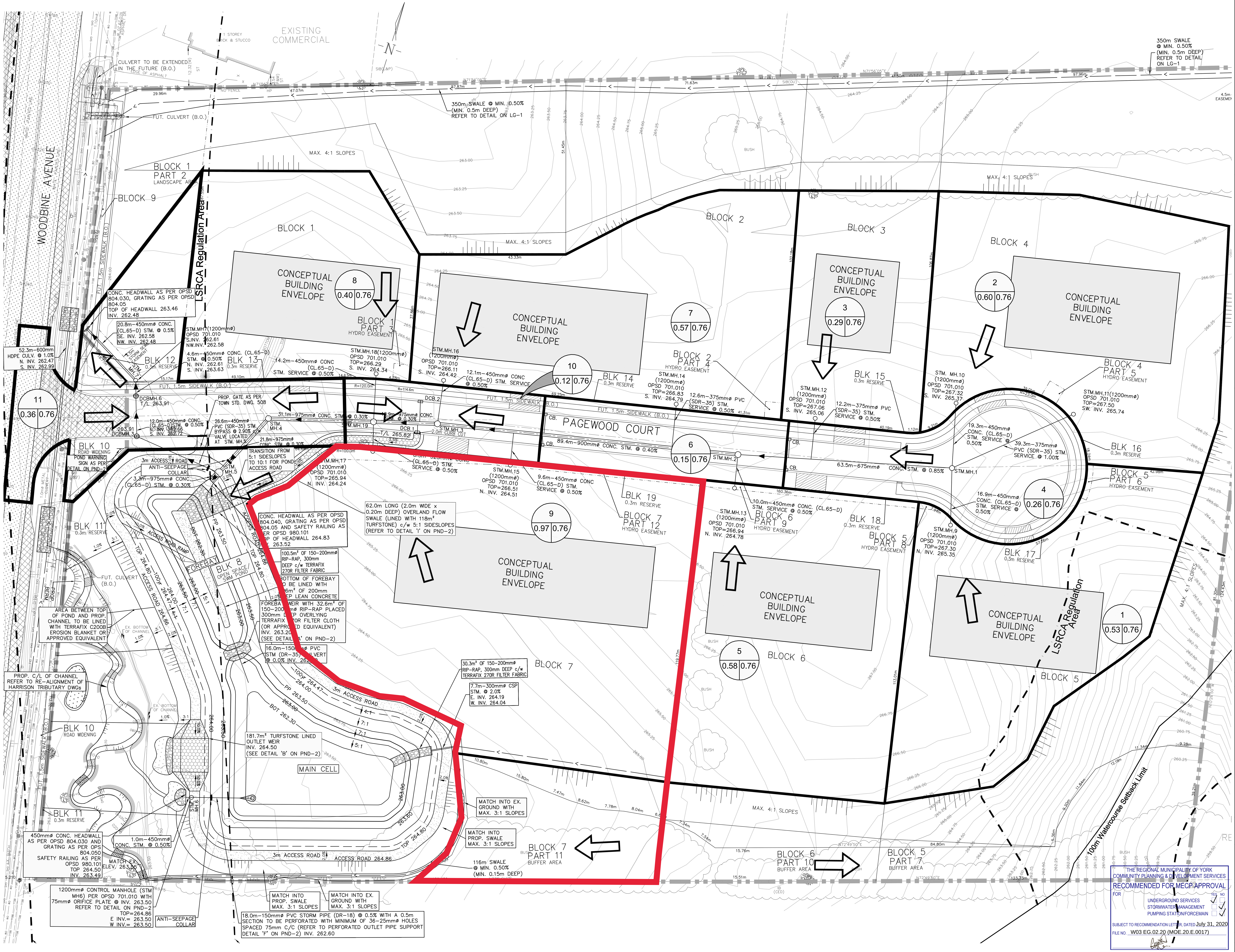
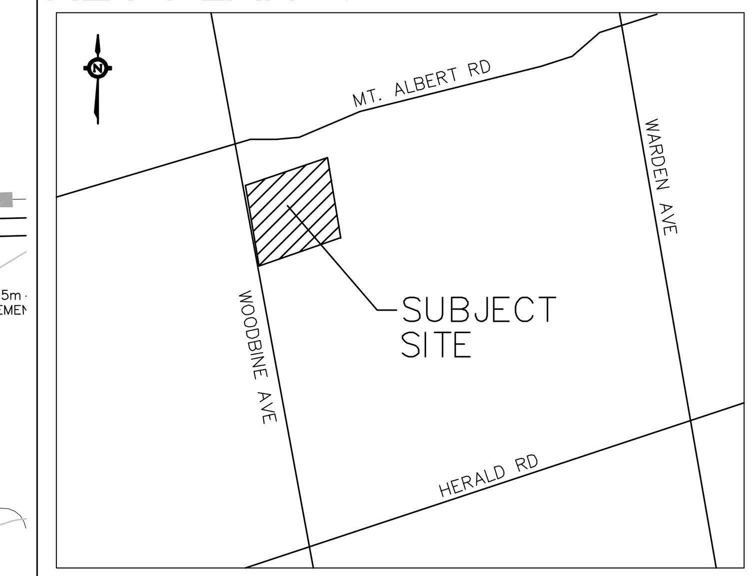


KEY PLAN N.T.S.



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LEGEND:

- SUBJECT PROPERTY LINE
- 291.19 EXISTING ELEVATION
- 291.14 PROPOSED ELEVATION
- STM.MH.1 PROPOSED STORM MANHOLE
- PROPOSED STORM SEWER
- PROPOSED STORM SERVICE
- PROPOSED SWALE
- CONCEPTUAL BUILDING ENVELOPE
- CATCHMENT AREA BOUNDARY
- PROPOSED OVERLAND FLOW DIRECTION
- 34 CATCHMENT AREA; 0.11 RUN OFF COEFFICIENT; 0.75 AREA (ha)
- CONCEPTUAL LOT LINE
- LSRCA REGULATION AREA
- 263.50 EXISTING CONTOUR
- RIGHT OF WAY

ITEM	DESCRIPTION	DATE	BY	APPRV'D
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2	2nd SUBMISSION	AUG. 22/16		
1	1st SUBMISSION	NOV. 13/15		

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER
D. WOO
100073980
JULY 30, 2020
PROVINCE OF ONTARIO

ENGINEER'S SEAL

PROJECT:
PART OF LOT 10 CONCESSION 4 TOWN OF EAST GWILLIMBURY REGIONAL MUNICIPALITY OF YORK

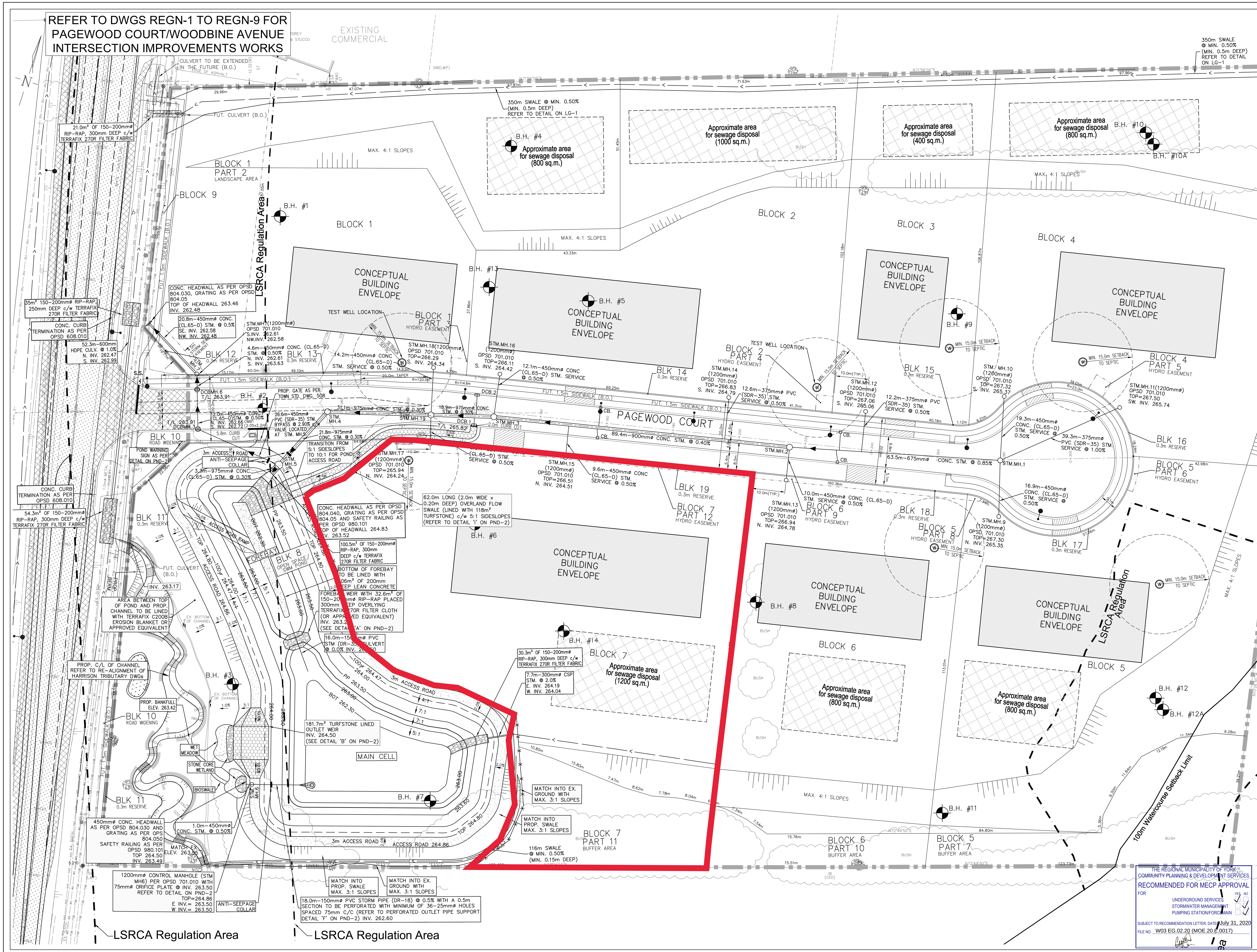
TITLE:
STORM DRAINAGE PLAN

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475–1900
FAX: (905) 475–8335

Town of East Gwillimbury
Our town, Our future

DRAWN:	SCALE: 1:500	DWG. No. STM-1	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: APR 2020	SHEET No. 5 OF 19	

File: P:\2020 Woodbine and Mt. Albert\Drawings\Phase 1\Base\2020D–BASE(MNT)–C3D.dwg – Revised by <JCASEY> : Thu, Jul 30 2020 – 12:28pm



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LEGEND:

- SUBJECT PROPERTY LINE
- PROPOSED STORM MANHOLE
- PROPOSED STORM SEWER
- PROPOSED STORM SERVICE
- PROPOSED STORM DOUBLE CATCHBASIN
- PROPOSED STORM CATCHBASIN
- CONCEPTUAL WELL
- PROPOSED LIGHT STANDARD
- CONCEPTUAL BUILDING ENVELOPE
- CONCEPTUAL SEWAGE DISPOSAL LOCATION
- CONCEPTUAL DRIVEWAY LOCATION
- PROPOSED PAVEMENT WIDENING/ADJUSTMENT (REFER TO REGION DWGS. FOR PAVEMENT STRUCTURE SPECIFICATIONS)
- LSRCA REGULATION AREA
- CONCEPTUAL LOT LINE
- RIGHT OF WAY
- COMMUNITY MAILBOX PAD (SEE DETAILS ON 900 SERIES DRAWINGS) NUMBER OF MODULES
- EXISTING GAS
- EXISTING BELL
- EXISTING YORKNET FIBRE OPTIC CABLE
- EXISTING BURIED HYDRO

ITEM	DESCRIPTION	DATE	BY	APPR'D
4	3rd REGIONAL SUBMISSION	APR. 20/20		
3	3rd SUBMISSION	MAR. 08/19		
2	2nd SUBMISSION	AUG. 22/16		
1	1st SUBMISSION	NOV. 13/15		

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER D. WOO 100073980 JULY 16, 2020 PROVINCE OF ONTARIO

ENGINEER'S SEAL

PROJECT: PART OF LOT 10 CONCESSION 4 TOWN OF EAST GWILLIMBURY REGIONAL MUNICIPALITY OF YORK

TITLE: GENERAL SERVICE PLAN

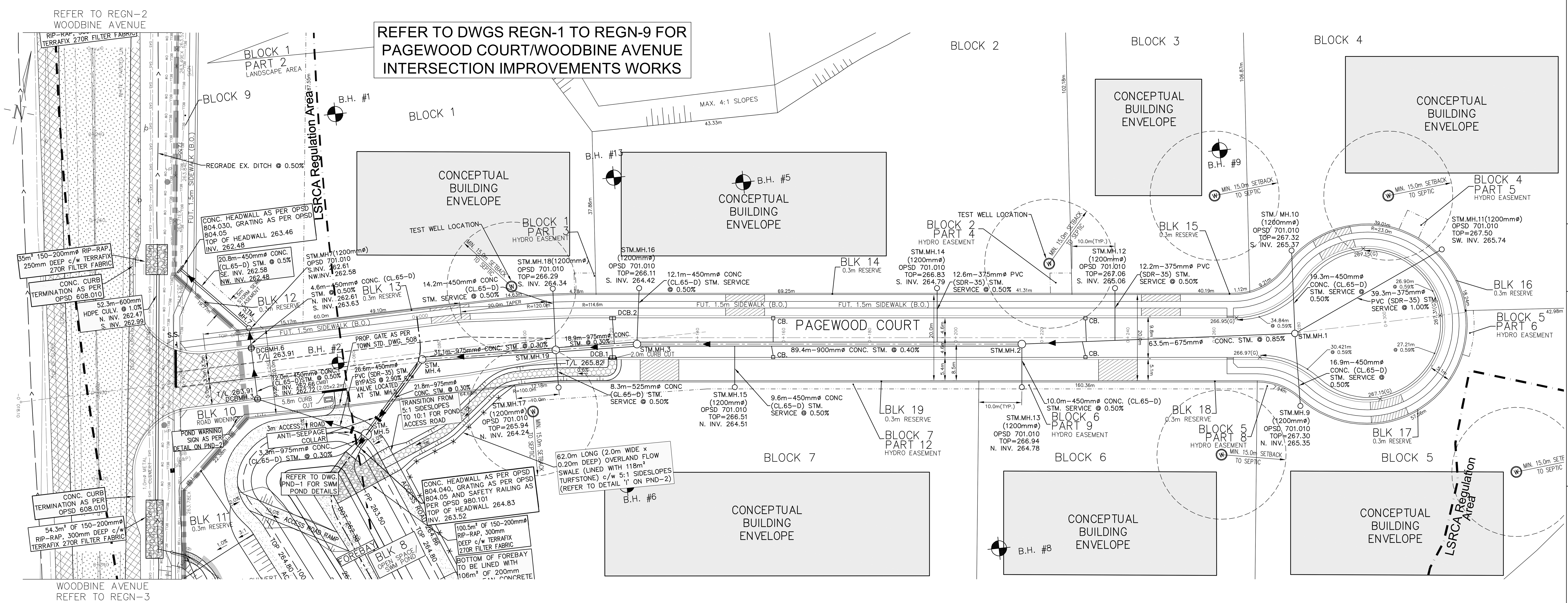
SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

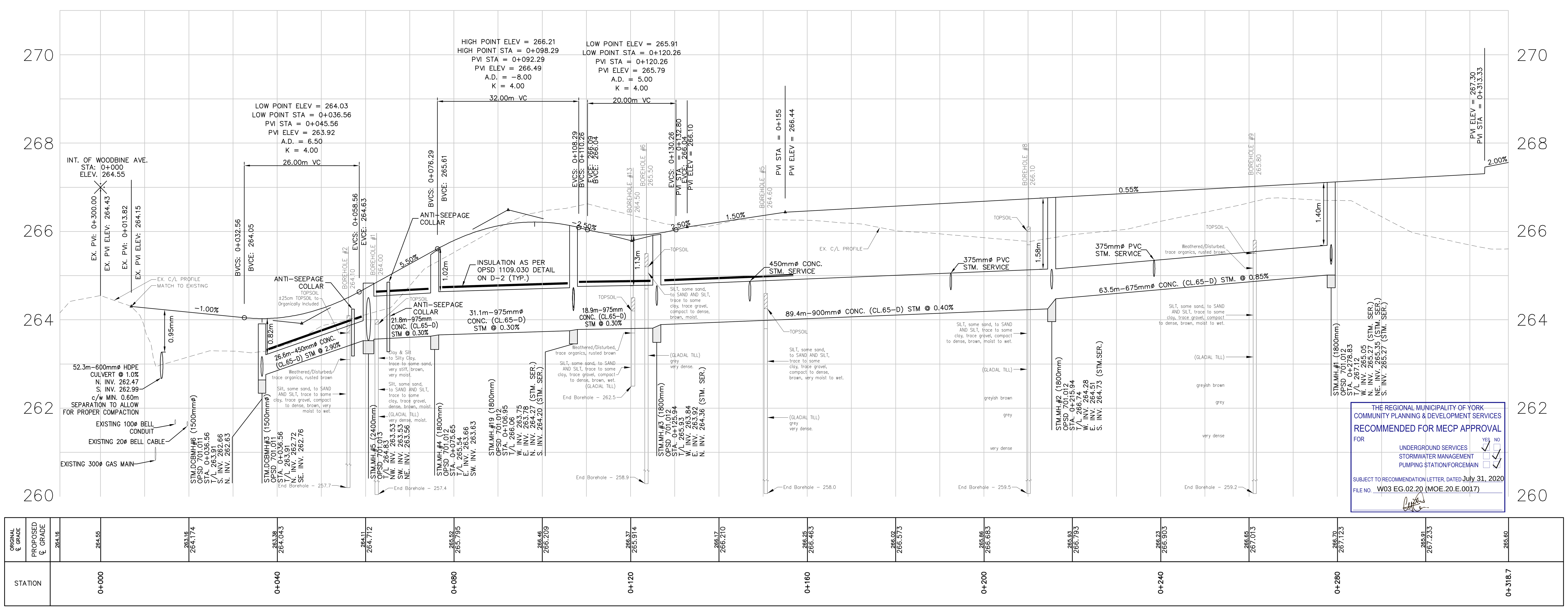
Town of East Gwillimbury

THE REGIONAL MUNICIPALITY OF YORK COMMUNITY PLANNING & DEVELOPMENT SERVICES
RECOMMENDED FOR MECP APPROVAL
FOR: UNDERGROUND SERVICES / STORMWATER MANAGEMENT / PUMPING STATION/FORCEMAIN
SUBJECT TO RECOMMENDATION LETTER, DATED July 31, 2020
FILE NO. W03 EG.02.20 (MOE.20.E.0017)

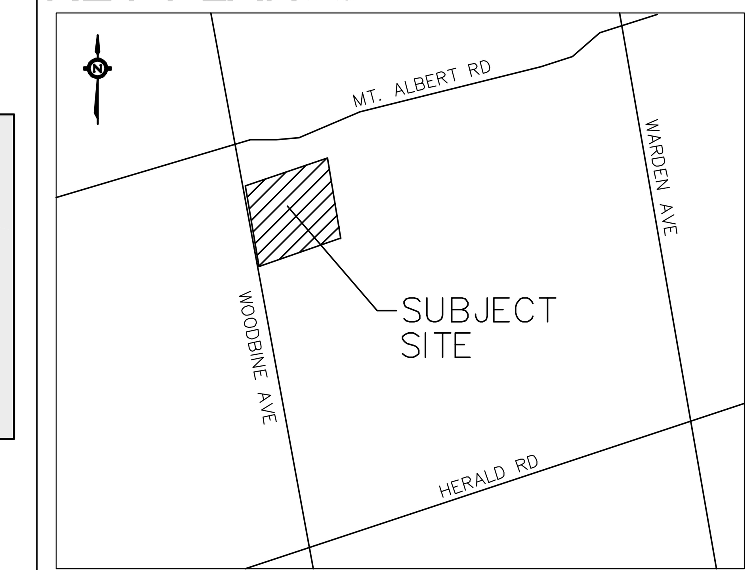
DRAWN:	SCALE: 1:500	DWG. No. GSP-1	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: JUL 2020	SHEET No. 6 OF 19	

File: P:\2020 Woodbine and Mt. Albert\Drawings\Phase 1\Base\2020D-BASE(MNT)-C30.dwg - Revised by <JCASEY> : Tue, Jul 14 2020 - 3:36pm





KEY PLAN N.T.S.



SURVEY NOTES:

1. TOPOGRAPHICAL SURVEY INFORMATION PROVIDED KRCMAR SURVEYORS LTD. ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48-100 HAVING AN ELEVATION OF 271.230 METRES.
2. LEGAL INFORMATION PROVIDED BY WESTON CONSULTING.

GENERAL NOTES:

1. DO NOT SCALE DRAWINGS.
2. ALL DIMENSIONS ARE TO BE CHECKED AND VERIFIED ON SITE PRIOR TO CONSTRUCTION.
3. ANY DISCREPANCIES, OMISSIONS, OR ERRORS ARE TO BE REPORTED TO THE CONSULTANT. NO WORK IS TO PROCEED BEFORE CLARIFICATION OF THE DISCREPANCIES, ERRORS, OR OMISSIONS IS RECEIVED FROM THE CONSULTANT.
4. ALL EXISTING CONDITIONS TO BE CHECKED AND VERIFIED ON SITE PRIOR TO CONSTRUCTION.
5. ONLY LATEST APPROVED DRAWINGS TO BE USED FOR CONSTRUCTION.
6. ALL DRAWINGS AND SPECIFICATIONS REMAIN THE PROPERTY OF THE CONSULTANT, AND SHALL BE RETURNED TO THE CONSULTANT UPON REQUEST. IN NO WAY SHALL THE DRAWINGS AND/OR SPECIFICATIONS IN WHOLE OR IN PART BE REPRODUCED OR DISTRIBUTED WITHOUT THE PERMISSION OF THE CONSULTANT.
7. PRIOR TO ANY WORKS ON MUNICIPAL PROPERTY A ROAD OCCUPANCY PERMIT SHALL BE OBTAINED FROM THE TOWN OF EAST GWILLIMBURY.

LEGEND:

PAVEMENT MARKING
S.S. STOP SIGN
SUBJECT PROPERTY LINE
291.19 EXISTING ELEVATION
291.14 PROPOSED ELEVATION
2.0% FLOW DIRECTION AND SLOPE GRADE
CONCEPTUAL WELL
STM.MH.10 PROPOSED STORM MANHOLE
PROPOSED STORM SEWER
PROPOSED STORM SERVICE
PROPOSED RIGHT-OF-WAY
CONCEPTUAL BUILDING ENVELOPE
CONCEPTUAL SEWAGE DISPOSAL LOCATION
CONCEPTUAL DRIVEWAY LOCATION
PROPOSED PAVEMENT WIDENING/ADJUSTMENT (REFER TO REGION DWGS. FOR PAVEMENT STRUCTURE SPECIFICATIONS)
CONCEPTUAL LOT LINE
3:1 SLOPE (UNLESS OTHERWISE NOTED)
EXISTING TREE LINE
RIGHT OF WAY
EXISTING GAS
EXISTING BELL
EXISTING YORKNET FIBRE OPTIC CABLE
EXISTING BURIED HYDRO

ITEM	DESCRIPTION	DATE	BY	APPR'VD
4	3rd REGIONAL SUBMISSION	APR. 20/20		
3	3rd SUBMISSION	MAR. 08/19		
2	2nd SUBMISSION	AUG. 22/16		
1	1st SUBMISSION	NOV. 13/15		

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER
D. WOO
100073980
JULY 14, 2020
PROVINCE OF ONTARIO

ENGINEER'S SEAL

THE REGIONAL MUNICIPALITY OF YORK
COMMUNITY PLANNING & DEVELOPMENT SERVICES
RECOMMENDED FOR MECP APPROVAL
FOR: UNDERGROUND SERVICES YES ✓; STORMWATER MANAGEMENT ✓; PUMPING STATION/FORCEMAIN
SUBJECT TO RECOMMENDATION LETTER, DATED July 31, 2020
FILE NO. W03 EG.02.20 (MOE.20.E.0017)

PROJECT:
PART OF LOT 10
CONCESSION 4
TOWN OF EAST GWILLIMBURY
REGIONAL MUNICIPALITY OF YORK

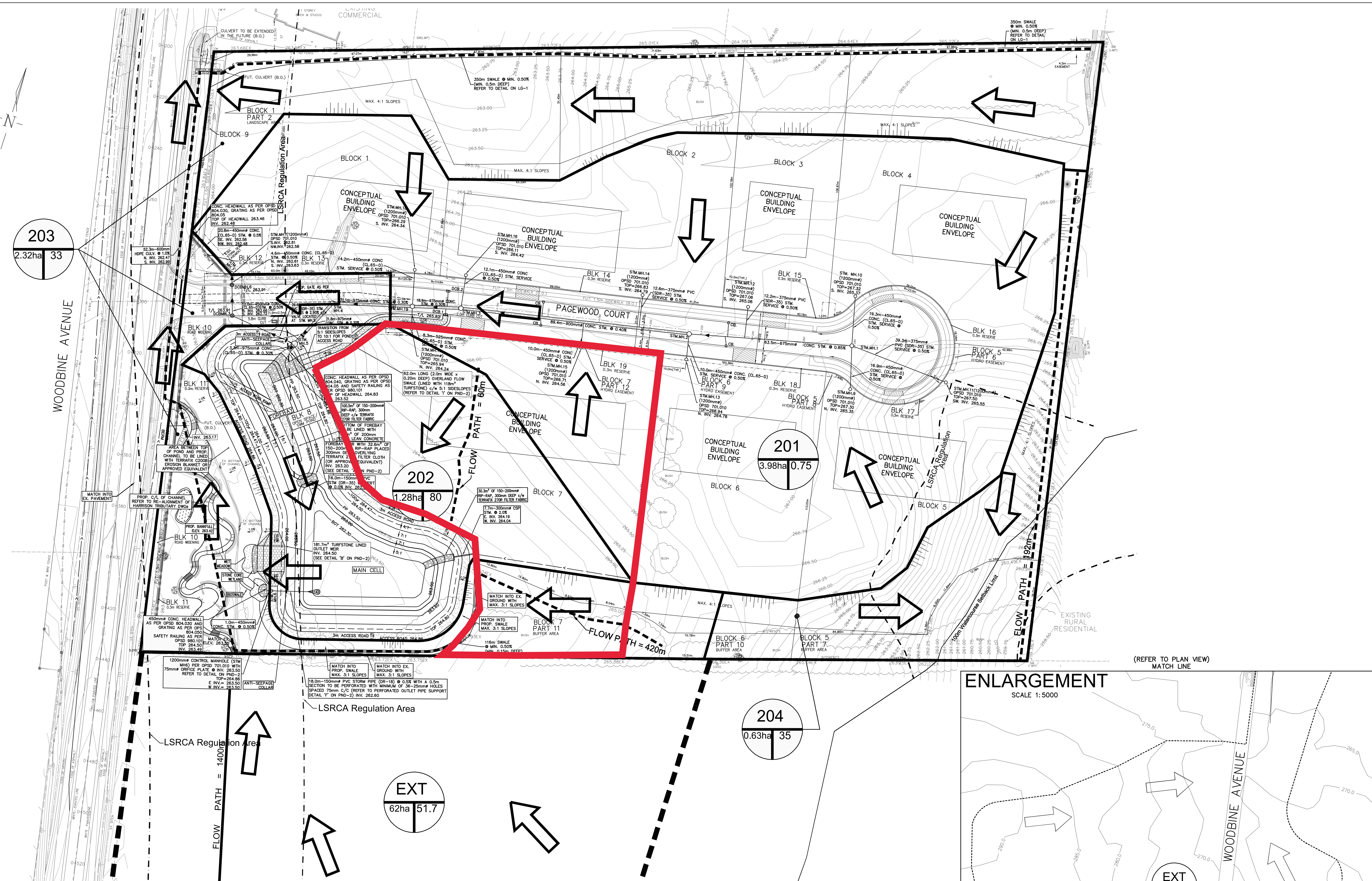
TITLE:
PLAN AND PROFILE PAGEWOOD CT.
STATION 0+000 TO 0+318.7

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475-1900
FAX: (905) 475-8335

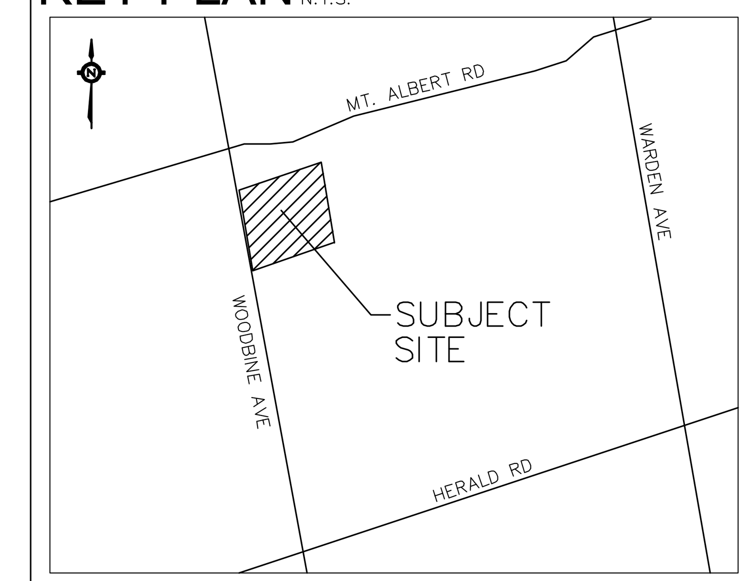
Town of East Gwillimbury

DRAWN:	SCALE: HOR. 1:500 VERT. 1:50	DWG. No. PP-1	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: JUL 2020	SHEET No. 11 OF 19	

File: P:\2020 Woodbine and Mt. Albert\Drawings\Phase 1\Base\20200-BASE(MNT)-C3D.dwg – Revised by <JCASEY> : Tue, Jul 14 2020 – 3:36pm



KEY PLAN N.T.S.



SURVEY NOTES:

1. TOPOGRAPHICAL SURVEY INFORMATION PROVIDED KRCMAR SURVEYORS LTD. ELEVATIONS SHOWN HEREON ARE GEODETIC AND ARE RELATED TO TOWN OF EAST GWILLIMBURY BENCH MARK No. 48–100 HAVING AN ELEVATION OF 271.230 METRES.
2. LEGAL INFORMATION PROVIDED BY WESTON CONSULTING.

GENERAL NOTES:

1. DO NOT SCALE DRAWINGS.
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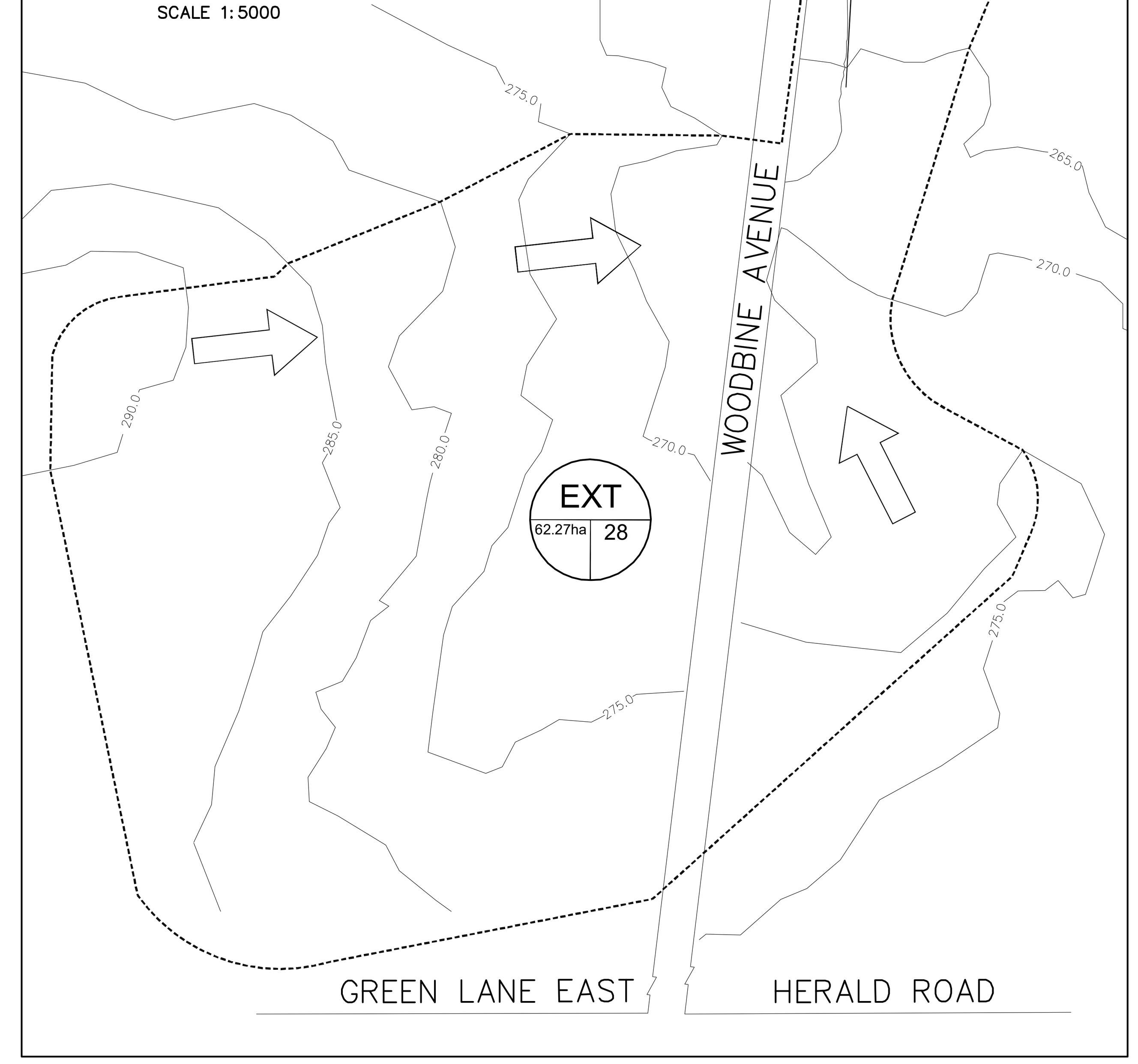
7. PRIOR TO ANY WORKS ON MUNICIPAL PROPERTY A ROAD OCCUPANCY PERMIT SHALL BE OBTAINED FROM THE TOWN OF EAST GWILLIMBURY.

LEGEND:

- SUBJECT PROPERTY LINE
- STM.MH.1 — PROPOSED STORM MANHOLE
- PROPOSED STORM SEWER
- PROPOSED STORM SERVICE
- PROPOSED RIGHT–OF–WAY
- CONCEPTUAL LOT LINE
- 3:1 SLOPE (UNLESS OTHERWISE NOTED)
- EXISTING TREE LINE
- 240 — EXISTING MAJOR CONTOUR
- 242 — EXISTING MINOR CONTOUR
- CATCHMENT BOUNDARY
- 201 — CATCHMENT AREA
- 0.75 — RUN OFF COEFFICIENT/CN
- 2.10ha — AREA (ha)
- OVERLAND FLOW DIRECTION
- RIGHT OF WAY

ENLARGEMENT

SCALE 1:5000



ITEM	DESCRIPTION	DATE	BY

SCHEDULE OF REVISIONS

LICENSED PROFESSIONAL ENGINEER S. E. KURTZ FEB 10, 2021 PROVINCE OF ONTARIO

ENGINEER'S SEAL

PROJECT: PART OF LOT 10 CONCESSION 4 TOWN OF EAST GWILLIMBURY REGIONAL MUNICIPALITY OF YORK

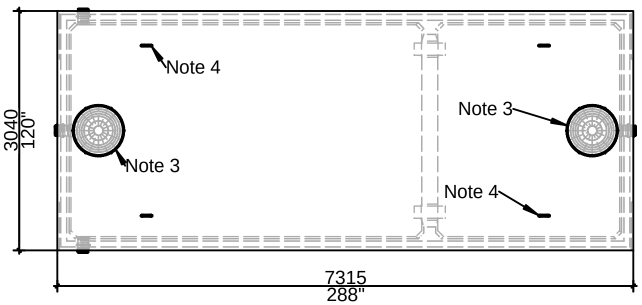
TITLE: POST-DEVELOPMENT STORM DRAINAGE PLAN

SCS consulting group ltd
30 CENTURIAN DRIVE, SUITE 100
MARKHAM, ONTARIO L3R 8B8
TEL: (905) 475–1900
FAX: (905) 475–8335

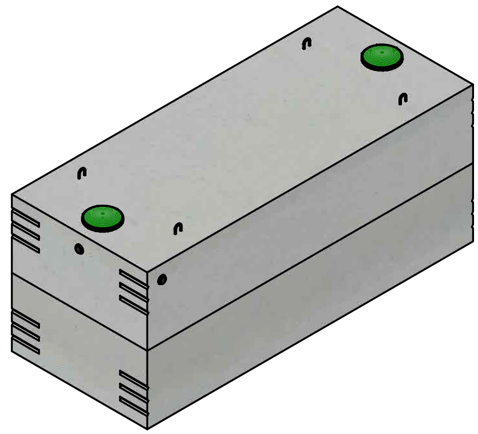
Town of East Gwillimbury

DRAWN:	SCALE: 1:750	DWG. No. SWM-2	PROJECT No. 2020
DESIGNED:			
CHECKED:	DATE: FEB 2021	SHEET No. 15 OF 19	

APPENDIX XV – STORAGE TANK



Top View



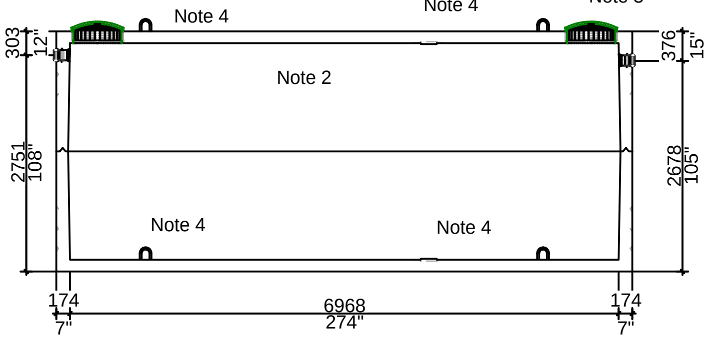
Isometric View

WORKING CAPACITY: 46,851L (10,306 IG)

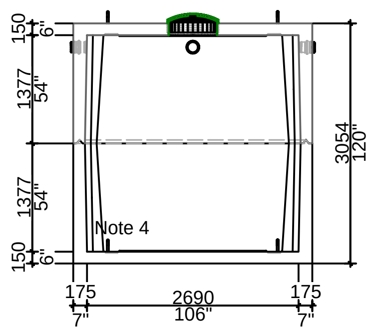
Bare Base Weight.....................19,137 Kg (42,190 Lbs)
Lid Weight................................18,735 Kg (41,304 Lbs)
Total Tank Weight.....................37,872 Kg (83,494 Lbs)

Notes

1. 75mm dia. PVC pipe cast-in to facilitate pump installation or venting
2. Water Storage Capacity to underside of top 52,064 Litres (11,453 Gal)
3. Cast in place 610mm ID Tuf-tite riser with dome lid secured with stainless steel fasteners extending 28mm above top
4. Lifting Hooks - 4 Points Each Sections



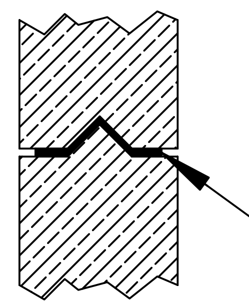
Side View



End View

Tank Markings Observed:
Inlet & Outlet are Marked
Tank is Marked on Tank Lid - Inlet End

NPCP
HOLDING 50000 L
B.D. 2 METRE
WC 46851
NON SULPHATE
AGINP



Mastic Sealant

Tongue and Groove Joint

NEWMARKET PRE-CAST
Solid For 60 Years

20 Victoria St
Uxbridge, ON L9P 1N4
Tel: 905-852-6111
Toll Free: 1-800-263-1297
Fax: 905-852-4340
Info@Newmarketprecast.com
Newmarketprecast.com

Model 50000 Holding

10000 Imperial Gallon

Designed for up to **Two Metre** burial over top of tank - Deep burial and vehicle traffic tanks are available upon request. Specialty conformations may be possible

- This tank design is installed in multiple units.
- Each unit is tongue and groove design and is sealed with a fibrous mastic sealant
- Tanks are cast with 45 MPA strength concrete at 7 days with 5-7% air
- Steel reinforced with 20m rebar, spaced at 250mm in each direction at 25mm cover.
- Rebar Cover 25mm, CAN/CSA-G30.18.
- Tanks conform to National Standard of Canada CAN/CSA B66-16
- Non-sulphate resistant concrete
- Standard three 100mm Inlets and one 100mm Outlet

H/W = 8 1/2" / 11"

Allplan 2021

APPENDIX XVI – CofA FOR EXISTING SWM FACILITY

East Gwillimbury

Suggest an address correction

Owner Name

THE CORPORATION OF THE TOWN OF EAST GWILLIMBURY

Last Sale

$2

Feb 18, 2022

Lot Size

122,278 ft² — Area

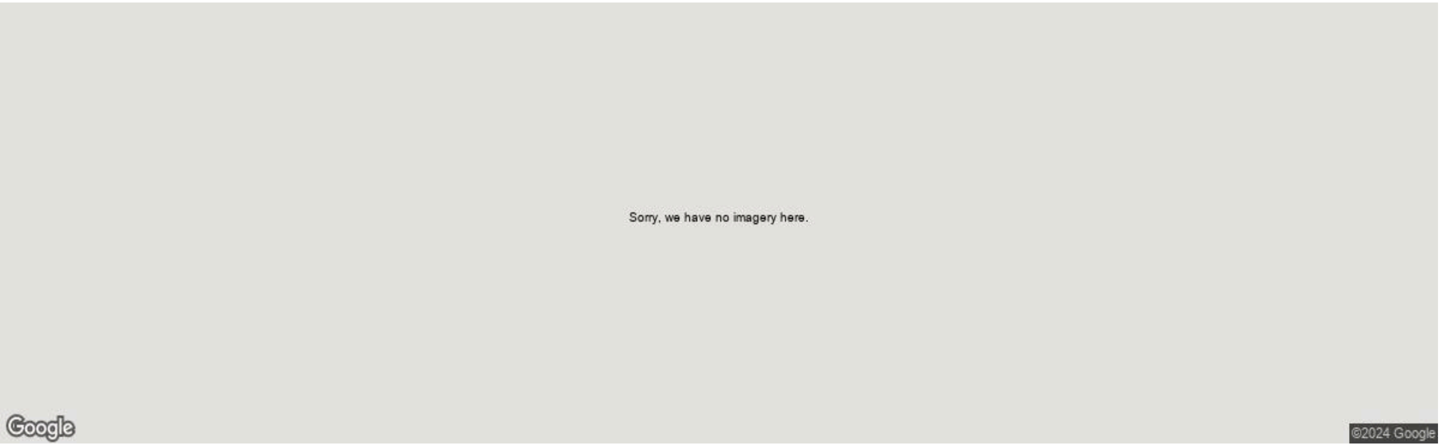
1,617 ft — Perimeter

Measurements Available
(See Site & Structure)

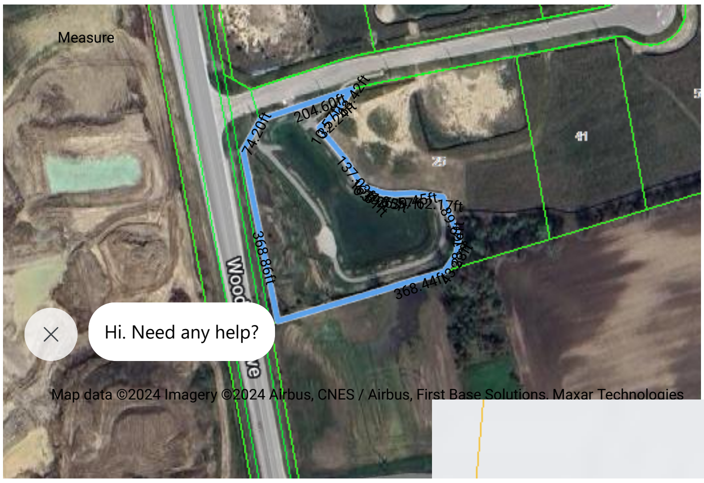
Legal Description

BLOCK 8, PLAN 65M4720 TOWN OF EAST GWILLIMBURY

Property Details



GeoWarehouse Address EAST GWILLIMBURY	**Ownership Type** Freehold	**Registration Type** Certified (Land Titles)
Land Registry Office York Region (65)	**Land Registry Status** Active	**PIN** 034400348
Owner Names THE CORPORATION OF THE TOWN OF EAST GWILLIMBURY	**Property Type** OTHER NON RES	



Lot Size

Area: 122,277.90 ft² (2.807 ac) **Perimeter:** 1,617.45 ft

Measurements: 63.42 ft x 204.60 ft x 74.20 ft x 368.86 ft x 368.44 ft x 43.23 ft x 38.14 ft x 89.69 ft x 62.17 ft x 39.45 ft x 26.57 ft x 37.85 ft x 6.79 ft x 16.61 ft x 137.03 ft x 10.57 ft x 32.24 ft

Lot Measurement Accuracy: LOW

Site & Structure

Please note, all information contained in the Site & Structure section of the Property Report is owned and maintained by MPAC. If any data in this section is missing or incorrect, please contact MPAC for assistance at propertyline@mpac.ca.

Assessment 1 | ARN : 195400002348413

mpac propertyline

Site — **Frontage:** N/A — **Depth:** N/A

Structure — **Property Description:** N/A — **Property Code:** N/A

Assessment Details — **Current Assessed Value :** N/A

Valuation Date: N/A

Assessment Property Information

Property Address:
Unit Number:
N/A
Municipality:
N/A

Valuation & Sales

Sales History

Sale Date	Sale Amount	Type	Party To	Notes
Feb 18, 2022	$2	Transfer	THE CORPORATION OF THE TOWN OF EAST GWILLIMBURY;	

HoodQ™

SCHOOLS
3 public & 4 Catholic schools serve this home. Of these, 7 have catchments. There are 2 private schools nearby.

PARKS & REC
3 playgrounds, 3 ball diamonds and 8 other facilities are within a 20 min walk of this home.

TRANSIT
Street transit stop less than a 2 min walk away. Rail transit stop less than 3 km away.

powered by HoodQ

Demographics

Neighbourhood (NBH) | Community (COM) | City (CITY)

Neighbourhood: refers to the property's Dissemination Area as defined by Statistics Canada.

Community: refers to the property's Forward Sortation Area (FSA), the first 3 digits of the property's postal code.

City: refers to the property's Census Subdivision as defined by Statistics Canada.

For questions, please contact GeoWarehouse.support@teranet.ca

Population

Total Population NBH 460 COM 45,443 CITY 27,824

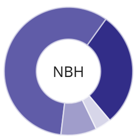
Gender

	Female	Male
NBH	51%	49%

Family Status

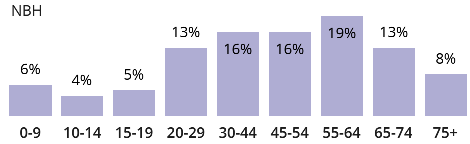
	>15 Married with Children	>15 Single with Children
NBH	15%	4%

Marital Status



Single	29%
Married	58%
Separated / Divorced	9%
Other	4%

Age Distribution



Households

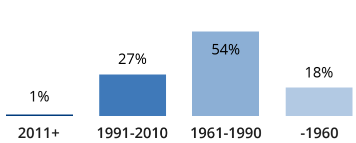
Structure Details

	Total # of Occupied Private Dwellings	Dominant Year Built
NBH	146	1961-1980

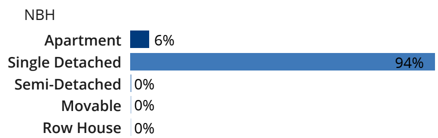
Ownership

	Owned	Rented
NBH	86%	14%

Age of Home (Years)



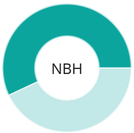
Structural Type



Socio-Economic

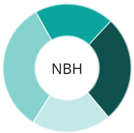
Avg. Household Income	NBH $146,090	COM $148,371	CITY $148,806

Employment



43%
Unemployed

Highest Level of Education



University	27%
College	20%
High School	33%
Other	20%

57%
Employed

Dominant Profession #1
Management

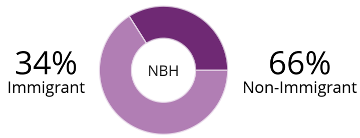
Dominant Profession #2

Occupations In Trades, Transport, Operators

Cultural

Dominant Language

Immigration Status



Dominant Market Group

[Prizm Group #14]

SOURCE
Prizm is the latest release of the pioneering segmentation system from Environics Analytics that classifies Canada's neighbourhoods into 68 unique lifestyle types.