1766 Wilton Grove, London, Ontario

Functional Servicing and Stormwater Management Report

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Project 23-967

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1766 Wilton Grove Road

Functional Servicing and Stormwater Management Report

1. Introduction

Greck and Associates Limited has been retained by Dancor Construction Limited (The Client) to prepare a Functional Servicing and Stormwater Management Report (FSR) for 1766 Wilton Grove Road in London, Ontario (Subject Property) in support of the proposed industrial development consisting of three buildings along with associated parking.

This report provides an overview of the proposed development plans and examines their functional serviceability, including requirements and proposed design works related to:

- General site grading
- Water distribution
- Sanitary sewer servicing
- Major and minor stormwater drainage systems
- Stormwater management; and
- Construction, erosion and sediment control

This functional servicing report has been prepared in accordance with accepted engineering practices and criteria from the governing approval agencies, including the City of London (City), Upper Thames River Conservation Authority (UTRCA), and Ministry of Environment, Conservation & Parks (MOECP).

In summary, from the completed site servicing, grading, and stormwater management engineering designs, it has determined that the development can be serviced with existing and proposed infrastructure according to policies and guidelines required by the regulating agencies.

1.1 Background

1.1.1 Site Location and Description

The subject property is located in the City of London, approximately 1.65 kilometers east of the Wilton Grove Road and Highbury Avenue South intersection.

Based on a review of current and historical aerial orthophotography, the subject property is partially developed with a commercial development called “Country Paws Boarding”. Within this development there are two existing structures and asphalt driveways to provide access.

The subject property is bound by Wilton Grove Road to the south, Highway 401 East to the north, 1806 Wilton Grove Road to the east, and 1710 Wilton Grove Road to the west.

The subject property is within the Upper Thames Conservation Authority Regulation Limit and is subject to the *Conservation Authorities Act*. Within the subject property, there is Dingman Creek which is a UTRCA regulated watercourse that runs along the north and northeast limit of the subject property.

A topographic survey conducted by Callon Dietz Inc. was completed for the proposed development on July 21, 2022.

The development area slopes north, east and south with an average slope of 1.5%. The area sloping north and east drain directly towards Dingman Creek, and the area sloping south, drains towards Wilton Grove Road which ultimately directs overland flow to the towards Dingman Creek.

The site plan, building plans, and topographic survey are provided in **Appendix A**. Please see **Figure 1** for the site location plan.

1.1.2 Soil Conditions

EXP Service Inc. (EXP) completed a geotechnical investigation report on August 5th, 2022. The work included drilling and sampling eight (8) boreholes spaced throughout the subject property. The soil conditions within the limits of the subject property consist primarily of the following:

- A surficial layer of topsoil with a measured thickness of 250mm to 350mm, was encountered at all boreholes.
- Sandy Silt was found beneath the topsoil at depths ranging from 1.4m to 2.1m below the ground surface at Boreholes BH3, BH5, and BH8. Moist to very moist conditions were observed. Some amounts of clay were present. At BH3, trace amounts of organics were present. At BH1 a lower sandy silt layer was present at a depth ranging from 4.0m to 7.0m.
- Clayey silt was found beneath the topsoil at depths ranging from 1.4m to 4.0m below the ground surface at Boreholes BH1, BH4, and BH7 and underneath the

sandy silt in BH3. Trace to some amounts of sand and gravel were present, along with moist conditions.

- All boreholes terminated in a stratum of till, comprising of clayey silt to silt, ranging from 2.1m to 4.0m below the ground surface. The clayey silt till contained trace to some sand, trace gravel, and occasional cobbles, along with moist conditions. The silt till contained some sand, trace to some clay, and trace gravel, along with moist to very moist conditions.

Monitoring wells were installed in four (4) boreholes (BH1/MW, BH3/MW, BH5/MW, BH6/MW), and groundwater measurements were taken on three (3) dates in 2022 which include May 30, June 8, and June 14. The groundwater levels ranged from 265.42m to 269.67m. The groundwater flow direction within the subject property is northerly towards Dingman Creek. For more details, see the Geotechnical report dated August 5th, 2022, by EXP Services Inc., provided in **Appendix B**.

1.1.3 Flood Hazard Investigations

A flood hazard investigation was completed by Greck to determine the development limits pertaining to flood hazards with respect to the requirements, guidelines, and policies of UTRCA. The assessment included a review of hydraulic modelling of Dingman Creek and the implications that development may have on the overall floodplain.

Dingman Creek flow data and hydraulic modelling geometry files were provided by UTRCA on August 1st 2023. The files were input in GeoHEC-RAS. GeoHEC-RAS utilizes the United States Army Corps of Engineers, HEC-RAS software. This version of GeoHEC-RAS uses Version 6.4.1 of HEC-RAS.

The UTRCA hydraulic model was updated by deleting three (3) cross sections and replacing it with four (4) new cross sections to better characterize the hydraulics within the development area. All new cross sections were cut by extracting topographic data from the topographic survey and Lidar DTM Lake Erie 2016-18 Package I (Lidar DTM) made available from Land Information Ontario (LIO). The Lidar DTM has been raised by 0.45m to convert the vertical datum from CGVD2013 to CGVD28 to be consistent with the topographic survey. The Winston Churchill Boulevard and Highway 401 culvert crossing were also updated based on topographic survey and LIDAR DTM.

UTRCA regulates the Regulatory Floodplain using the 250-year storm event. The Regulatory Floodplain ranges from 268.06m to 267.98m within the proposed development area. The Regulatory Floodplain on the subject property is heavily influenced by upstream backwater effects from the undersized Wilton Grove culvert crossing. See the Floodplain Map provided in **Appendix C**.

A portion of the site plan encroaches onto the proposed development. As such, Greck completed a conceptual grading plan (**Drawing C101** provided in **Appendix J**) with the purpose of raising the lands to ensure the property is sufficiently floodproofed. This resulted in the incorporation of fill along the east and north development limit at a 3:1 stable slope up to the Regulatory Flood elevation + 0.3m of freeboard at a minimum. This slope is to be highly vegetated to ensure a stable slope is maintained.

To compensate for the fill within the floodplain a cut-fill analysis was conducted. The conceptual grading plan resulted in 2216m^3 of fill and 2270m^3 of cut. This resulted in a net cut of 54m^3 within the floodplain. The Regulatory floodplain is subject to backwater effects from the Wilton Grove culvert crossing. Areas within backwater effects do not contribute to channel conveyance. Therefore, any hydraulic impacts due to the proposed development are expected to be negligible.

The existing shear stresses and velocity from the flows from Dingman Creek were analyzed for the subject property to assess the risk for slope erosion .

The development is located on the right bank of Dingman Creek, as such, only the right of bank shear stresses is required to be assessed. The right of bank shear stresses experienced by the subject property is summarized in **Table 1-1** below.

Table 1-1: Dingman Creek Shear Stresses (n/m^2)

HEC-RAS Cross Section	1325	1241	1182	1121	995
2-year	0.13	1.39	1.04	0.28	1.67
5-year	0.10	1.61	1.01	0.77	0.77
10-year	0.08	1.45	1.24	0.85	0.63
50-year	0.09	0.90	1.61	1.14	0.64
100-year	0.12	1.04	2.12	1.50	0.80
250-year	0.15	1.19	2.73	1.92	0.99

The 250-year flow velocities experienced by the subject property is summarized in **Table 1-2** below. The greatest velocities in the watercourse are experienced within the channel, therefore to be conservative, the channel velocities were assessed.

Table 1-2: Dingman Creek Channel Velocities (m/s)

HEC-RAS Cross Section	1325	1241	1182	1121	995
250-year	0.17	0.59	1.13	1.05	0.60

The greatest reported shear stress and velocity is 2.73N/m^2 and 1.13m/s respectively, which occur during the 250-year event at cross section 1182. According to MTO Design Chart 2.16, the permissible shear stress for Bermuda Grass (Class E Vegetation) is 1.7kg/m^2 or 16.7N/m^2, and according to MTO Design Chart 2.17, the permissible flow velocity for Bermuda grass on easily eroded soils for slopes over 10% is 1.2m/s. The reported shear stresses and velocities do not exceed these thresholds, therefore the recommended highly vegetated slope will be sufficient to withstand the shear stresses and velocities from Dingman Creek. The reason for the low shear stress and velocities are primarily due to the backwater effects from the Wilton Grove culvert crossing.

It was determined that the proposed industrial development will be sufficiently floodproofed, and a cut-fill analysis was completed, which will have negligible hydraulic impacts and risk to slope erosion due to backwater effects from the Wilton Grove culvert crossing.

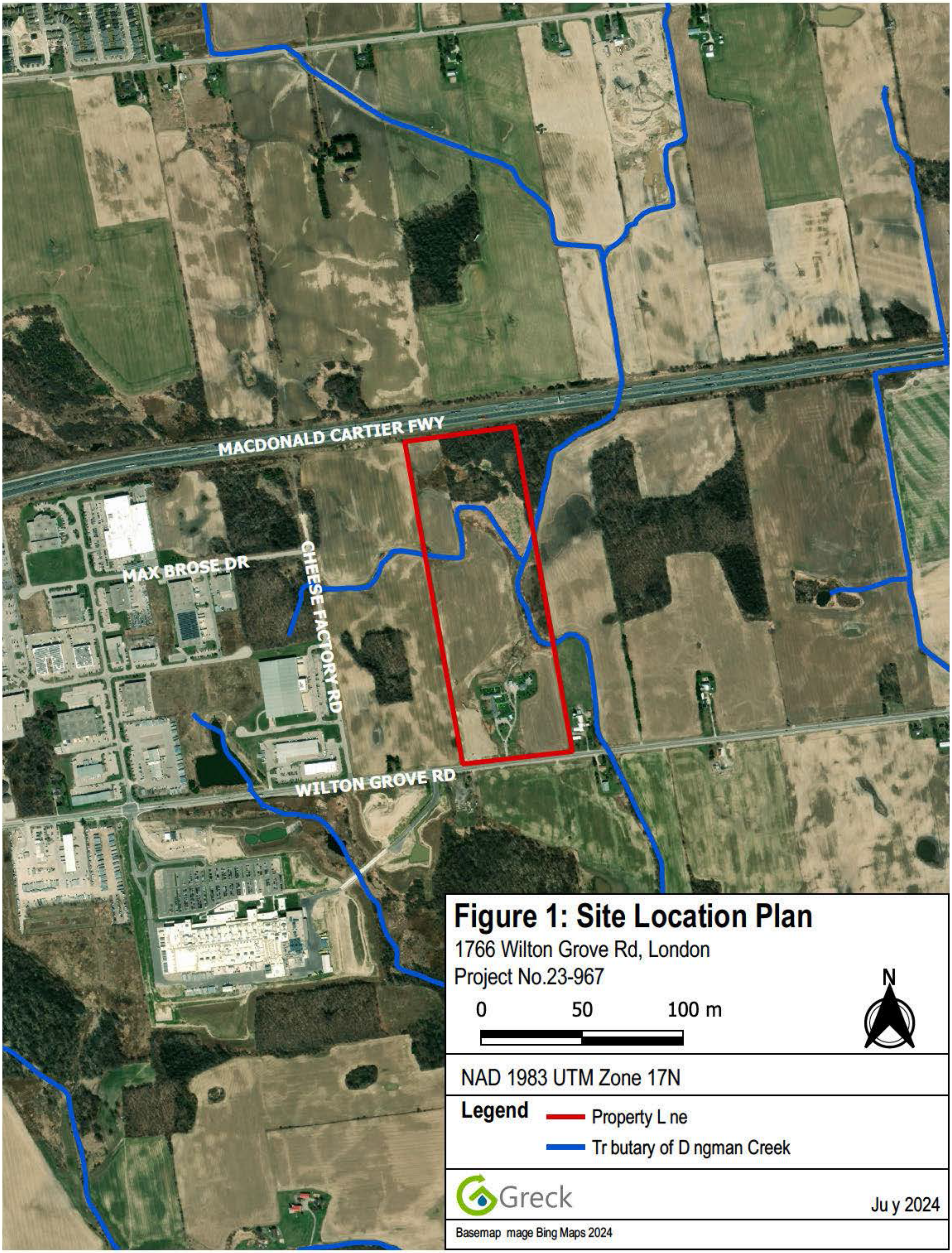


Figure 1: Site Location Plan

2. Proposed Development

The proposed development consists of three (3) industrial warehouse buildings. The development also features car parking with 465 parking spaces at ground level. The total new development area is approximately 147,236m^2 or 14.724 ha. A site plan, prepared by 'Architectural Design Inc.', of the proposed development can be found in **Appendix A.**

As per City of London design guidelines, the proposed industrial development design population is considered based on the site area. The proposed development population statistics can be found in **Table 2-1**.

Table 2-1: Proposed Development Population Breakdown

Type of Development	Flow * Allowance (L/ha/day)	Population Density * (population/ha)	Development Area (ha)	Equivalent Population
Industrial	25,000	100	14.724	1473

* As per City of London Design Specifications & Requirements Manual, January 2024

The proposed development will be serviced by the existing 900mm diameter municipal watermain that is located at the intersection of Wilton Grove Road and Cheese Factory Road. To tie into this location, a 200mm diameter watermain is proposed along Wilton Grove Road, extending from the intersection of Wilton Grove Road and Cheese Factory Road to the west limit of the subject property.The length of the external watermain servicing is approximately 330m.

Sanitary servicing will be provided by connecting a new sanitary sewer to the existing 375mm diameter sanitary sewer at MH114A that is located at the intersection of Max Brose Drive and Cheese Factory Road. To connect at this location, external sanitary servicing of approximately 390m and an extension of Max Brose Drive, from the subject property to Commerce Road, will be required.

For more details on the proposed water and sanitary services, please see Section 5 and 6 respectively, below for more details. Refer to **Drawings C102 and C103** provided in **Appendix J**.

3. Site Grading

As is typical with all development projects, earthmoving is required, to varying degrees, to achieve the municipal design criteria and accommodate the development form. In general, given the existing topography and the proposed development plan, a net fill of

27,668m^3 is anticipated. Surface runoff from the site will drain overland to catch basins and catch basin manholes into the proposed storm sewer system then through the stormwater management dry pond and then ultimately, discharging into the proposed storm outlet located at Dingman Creek along the east limit of the development site. To accommodate the proposed site plan, while following municipal standards, and respecting the development limits, there are two (2) proposed retaining walls. The first is located along the northeast limit of the subject property, across from the principal entrance to building 3, and is approximately 110.5m in length. The second is located along the east limit of the subject property, across from the principal entrance to building 2, and is approximately 33.3m in length. These retaining walls were necessary to accommodate the grade differential resulting from the finished floor elevations (FFE), the elevations required within the loading dock areas (1.2m below the FFE), and then matching into the existing elevations at the subject property's limits.

A grading plan has been provided in **Drawing C101**; see **Appendix J**. The plan will follow municipal design standards, as required, considering the following key design factors:

- Provide positive drainage from above ground structures/buildings,
- Match external grades,
- Meet minimum and maximum grades for landscape, roadways, and swales,
- Achieve municipal lot grading criteria,
- Provide safe overland flow relief,
- Provide sufficient cover for underground infrastructure,
- Minimize grading and earthworks where necessary.

4. Road Access

Currently, an existing asphalt pathway provides access to Wilton Grove Road servicing the existing development on the subject property. This existing asphalt pathway will be removed. For the proposed development, access will be provided with two proposed asphalt roadways adjacent to Wilton Grove Road. One at the southeast limit and the other at the southwest limit of the subject property. The driveway, including the fire route and parking stalls, will consist of asphalt and curbs. In accordance with the geotechnical report prepared by EXP, dated August 5th, 2022, the minimum pavement structure for the proposed road will be as follows in **Table 4-1**.

Table 4-1: Pavement Structure

Material	Thickness (mm)	
	Light Duty Pavement Structure (Cars Only)	**Heavy Duty Pavement Structure (Cars and Trucks)**
Asphalt		
Surface Course (HL3)	40	50
Basecourse (HL8)	50	60
Total Asphalt Depth	90	110
Base		
Granular A Base (OPSS 1010)	150	150
Granular B Type 1 Sub-Base (OPSS 1010)	300	450
Total Thickness	**540**	**710**

The details of the pavement structure are to be confirmed by the geotechnical consultant, and detailed traffic planning will be provided by others as required.

5. Water Servicing

This section serves to provide anticipated water demands and required fire flow calculations in support of functional servicing.

Greck obtained as-built drawings from the City (provided in **Appendix D**) for the areas adjacent to the subject property. The existing municipal watermain infrastructure is as follows:

- No existing watermain infrastructure directly adjacent to the subject property along Wilton Grove Road (Plan File No. 30607).
- 900mm diameter concrete watermain on Wilton Grove Road, within the municipal right-of-way, and is located west of the subject site, at the intersection of Wilton Grove Road and Cheese Factory Road. This watermain continues westerly along Wilton Grove Road and continues northerly along Cheese Factory Road (Plan File No. 18040, 18051, and 18052).
- 300mm diameter PVC watermain on Max Brose Drive, within the municipal right-of-way, along the south curb of the roadway (Plan File No. 18049 and 18050).
- 300mm diameter PVC watermain on Commerce Road, within the municipal right-of-way, along the east curb of the roadway (Plan File No. 18041 and 18042).

Existing fire hydrant locations near the subject property are as follows:

- Municipal fire hydrant located on Commerce Road at the northeast quadrant of the intersection of Commerce Road and Wilton Grove Road. This hydrant was the flow hydrant for the hydrant flow test.
- Municipal fire hydrant located on Wilton Grove Road approximately 100m west of the intersection of Wilton Grove Road and Commerce Road. This hydrant was the residual hydrant for the hydrant flow test.
- Privately owned fire hydrant located on Cheese Factory Road approximately 70m north of the intersection of Wilton Grove Road and Cheese Factory Road.

5.1 Water Supply and Appurtenances

As-built records for Wilton Grove Road and Cheese Factory Road indicate an existing 900mm diameter concrete watermain located on both roads. The extent of the 900mm diameter concrete watermain along Wilton Grove Road ends at the intersection of Wilton Grove Road and Cheese Factory Road. Water servicing for the subject property will be supplied by a new 200mm diameter PVC watermain that will tie into the existing 900mm diameter concrete watermain at the intersection of Wilton Grove Road and Cheese Factory Road to provide the domestic and fire flow requirements for the development. This requires external servicing to be completed westerly along Wilton Grove Road for a length of approximately 330m.

Within the subject property, the 200mm diameter watermain will tee-off the 200mm diameter watermain proposed along Wilton Grove Road and extend through the subject property to adequately service the three (3) proposed industrial buildings and provide services for the proposed private hydrants that are spread throughout the subject property.

The proposed 200mm diameter watermain will tee-off to the 150mm watermains that provide domestic and fire service to the three (3) proposed industrial buildings. The domestic service connection will be built as per City of London Standard Drawing W-CS-31. The proposed 200mm diameter watermain will service the development for fire and domestic water services, and the three (3) new industrial buildings will be fully equipped with a sprinkler system for fire protection. Backflow prevention at the three (3) internal building connections will be provided as per City of London Standard Drawing W-CS-31. Refer to **Drawing C102** in **Appendix J.**

Hydrants are to be located within a 90m radius of any portion of the building perimeter that is required to face a street. The fire department connection for an automatic sprinkler system is to be located not more than 45m to a hydrant and located on the outside of a building adjacent to a street or an access route, not less than 300mm and not more than 900mm above ground level, and provided with two 65mm hose connections with female swivel hose couplings (as per Ontario Building Code Section 3.2.5.16).

The nearest existing hydrants to the subject property exceed the allowable distances to service the development, so ten (10) private hydrants will be proposed throughout the subject property to meet the requirements. These private hydrants will be serviced by the proposed 200mm diameter watermain. Please see **Drawing C102** for the Servicing Plan provided in **Appendix J**, for the proposed watermain and hydrant layout.

A detailed fire protection plan for the building will be undertaken and supplemented by the building's mechanical engineer and/or fire system design consultant to confirm the proposed sizes for the water services.

5.1.1 Domestic Water Demands

The design criteria used to determine water demands were based on the City of London Design Specifications & Requirements Manual, January 2024, the Fire Underwriters Survey, 2020, and the Ministry of Environment (MOE) Design Guidelines for Drinking Water Systems, 2008, as required.

Average Day Demand (ADD), Maximum Day Demand (MDD), and Peak Hour Demand (PHD) factors were calculated using demand peaking factors and population values as per Table provided in section 7.3.2.2 per City of London Design Specifications. The industrial water demand rate used is as per section 3.4.4 of the MOE Design Guidelines for Drinking Water systems, 2008.

The estimated domestic water system demands for the proposed development of the subject property is summarized below in **Table 5-1**.

Table 5-1: Project Domestic Water Demands

Water Demand Rate	35,000 L/hectare/day
Maximum Day Factor	3.5
Peak Hour Factor	7.8
Average Daily Demand (ADD)	357.88 L/min (5.96 L/s)
Maximum Daily Demand (MDD)	1,252.56 L/min (20.88 L/s)
Peak Hour Daily Demand (PHD)	2,791.43 L/min (46.52 L/s)

A detailed breakdown of the calculated demands can be found in **Appendix E**.

5.1.2 Fire Flow Demands

Fire demands have been calculated using the *Water Supply for Public Fire Protection* (2020) prepared by Fire Underwriters survey (FUS). Detailed fire flow calculations are provided in **Appendix E**, and the results are summarized below in **Table 5-2**.

Table 5-2: Recommended Fire Flow

Proposed Development	Recommended Fire Flow (L/s)
Building 1	283.33
Building	250.00
Building 3	216.67

From the fire flow calculations, it was determined that the recommended fire flow of 283.33 L/s is required for the proposed development.

The first hydrant flow test was conducted by Northern Sprinkler Design on August 8th, 2024, at 9:00am. The flow hydrant was the previously mentioned existing fire hydrant located on Commerce Road connected to the 300mm diameter watermain located west of the subject property. The residual hydrant was the previously mentioned existing fire hydrant located on Wilton Grove Road connected to the 900mm diameter watermain located west of the subject property. The results indicate an available maximum flow of 2507.7 GPM (158.2 L/S) at a residual pressure of 20 psi. This is lower than the calculated required fire flow demand of 283.33 l/s. Thus, the existing watermain infrastructure is not sufficient to supply the proposed development with the required fire flow. Greck has been in coordination with the City of London regarding this matter, which is currently under review by the City's Staff. The FSR will be revised and updated in the subsequent submission upon receipt of the City's feedback concerning the inadequate capacity of the existing 900mm diameter watermain.

5.1.3 Total Water Demand

Based on the total residential water demand and the fire flow requirements, the fire flow plus the maximum daily demand (MDD) is 304.21 L/s (283.33 L/s + 20.88 L/s).

5.1.4 Watermain Hydraulic Analysis

A watermain hydraulic analysis will be completed at a future date once the proposed watermain layout has been reviewed and confirmed by the City of London.

6. Sanitary Servicing

This section serves to provide anticipated sanitary demands and an overview of the proposed sanitary servicing in support of functional servicing. As-built records for Wilton Grove Road (Plan File No. 30607) indicate that there is no existing sanitary infrastructure adjacent to the subject property.

As-built records indicate that a 375mm sanitary sewer is located on Max Brose Drive (Plan File No. 18050 and 18049). Additionally, a 450mm sanitary sewer is located on Cheese Factory Road (Plan File No. 18052). Both of these sanitary sewers are located to the west of the subject property. According to the City Plan File No. 18033 and 30559 and in the pre-consultation records with the City of London, the subject property is tributary to the 375mm sanitary sewer on Max Brose Drive. Refer to the as-built records provided in **Appendix D**. Sanitary servicing for the subject property will be provided by a new sanitary sewer network connecting to the existing 375mm diameter municipal sanitary sewer located at the intersection of Max Brose Road and Cheese Factory Road. This will require external servicing and an extension of the existing Max Brose Drive to tie into the existing 375mm diameter sanitary sewer at the existing sanitary manhole MH114A that can be found on Plan File No. 18050. The length of the external sanitary servicing is approximately 390m.

Based on the elevation of the subject property and the elevation of the invert at Ex. MH114A it is not possible to convey the proposed sanitary flows solely by gravity, therefore a pumping station will be required at the intersection of Max Brose Drive and Cheese Factory Road. The proposed sanitary pumping station is proposed within the municipal right-of-way, and it is our understanding that this pumping station will be assumed by the City of London in the future. The proposed sanitary flows will be conveyed by gravity throughout the subject property and along the proposed Max Brose Drive extension until it reaches the end of the length at proposed MH1A where the proposed pumping station will pump the sanitary flow up and into the tie in point at the existing MH114A.

Please see **Drawing C102** for the Servicing Plan provided in **Appendix J**, for the proposed sanitary layout.

6.1 Sanitary Design Parameter

The sanitary design parameters, as outlined in **Table 6-1** for the proposed development are based on the municipal design criteria from the City of London *Design Specifications and Requirements Manual, January 2024:*

Table 6-1: Sanitary Design Parameters

Population Densities	Industrial – 100 persons/ha
Development Area	14.724 hectares
Unit Industrial Sewage Flows	25000 L/ha/day
Infiltration Rate	0.10 L/s/ha
Minimum Flow Velocity	0.6 m/s
Maximum Flow Velocity	4.5 m/s
Minimum Sewer Pipe	200 mm dia.
Minimum Sewer Pipe Grade	0.61%
Minimum Sewer Depth of Cover	2.4 m

6.2 Sanitary Development Demands

The sanitary demand has been calculated during the design parameters as described in **Section 6.1,** and the results are summarized in **Table 6-2**.

Table 6-2: Proposed Development Sanitary Demand Summary

	Area (ha)	Average Daily Flow (L/s)	Harmon Peaking Factor	Peak Daily Flow (L/s)	Infiltration Rate (L/s/ha)	Extraneous Flow (L/s)	Total Design Flow (L/s)
Proposed Development	14.724	4.69	3.52	16.49	0.10	1.47	17.96

The proposed development will produce a total sanitary demand of 17.96L/s. Detailed sanitary development calculations can be found in the sanitary sewer design sheet provided in **Appendix F**.

7. Utilities

The proposed development is located within the serviced area of the City of London. Electrical, natural gas, and telecommunications infrastructure is available from the adjacent public road allowances. Existing overhead electrical lines traversing the property will need to be relocated as part of the development process. London Hydro will be contacted by the developer to initiate the process.

During the detailed engineering design stage, consultation with each of the service providers will be undertaken to provide them with specific load requirements for the development and proposed service entry locations. Detailed electrical, gas and utility design and coordination will be managed by a qualified engineer.

8. Surface Drainage

This section provides an outline of the drainage strategy for the proposed site plan and areas affected by the development. The proposed design will be in accordance with the City, UTRCA, and MOECP standards and guidelines.

8.1 Existing Drainage

Four (4) drainage areas characterize the existing drainage pattern within the development limits: Area A, B, C, and EXT-1. Please see **Figure 2** below for the pre-development drainage area plan. The existing land use of the subject property mostly consists of agricultural fields, forested area, and wetlands. The south portion of the property that fronts Wilton Grove Road consists of buildings, landscaping, and an asphalt driveway.

A description of each existing drainage area is summarized below:

- Area A (5.82ha) consists of buildings, landscaping, asphalt driveway, and agricultural fields. Area A drains south towards the roadside ditch servicing Wilton Grove Road.
- Area B (7.39ha) consists of buildings, landscaping, and agricultural fields. Area B drains east towards the tributary of Dingman Creek.
- Area C (3.43ha) consists of agricultural fields. Area C drains north towards the north tributary of Dingman Creek and the wetland.
- Area EXT-1 (0.55ha) represents external drainage area from the west which consists of agricultural fields. Area EXT-1 drains east towards Area B under existing conditions.

The roofs, asphalt, and landscaped areas were assigned a runoff coefficient of 0.90 as per the City of London Design Specifications & Requirements Manual dated October 2003 and updated August 2019. The pervious areas were assigned a runoff coefficient of 0.35 as per MTO Design Chart 1.07 for cultivated landuse at 0-5% slopes with loam or silt loam soils.

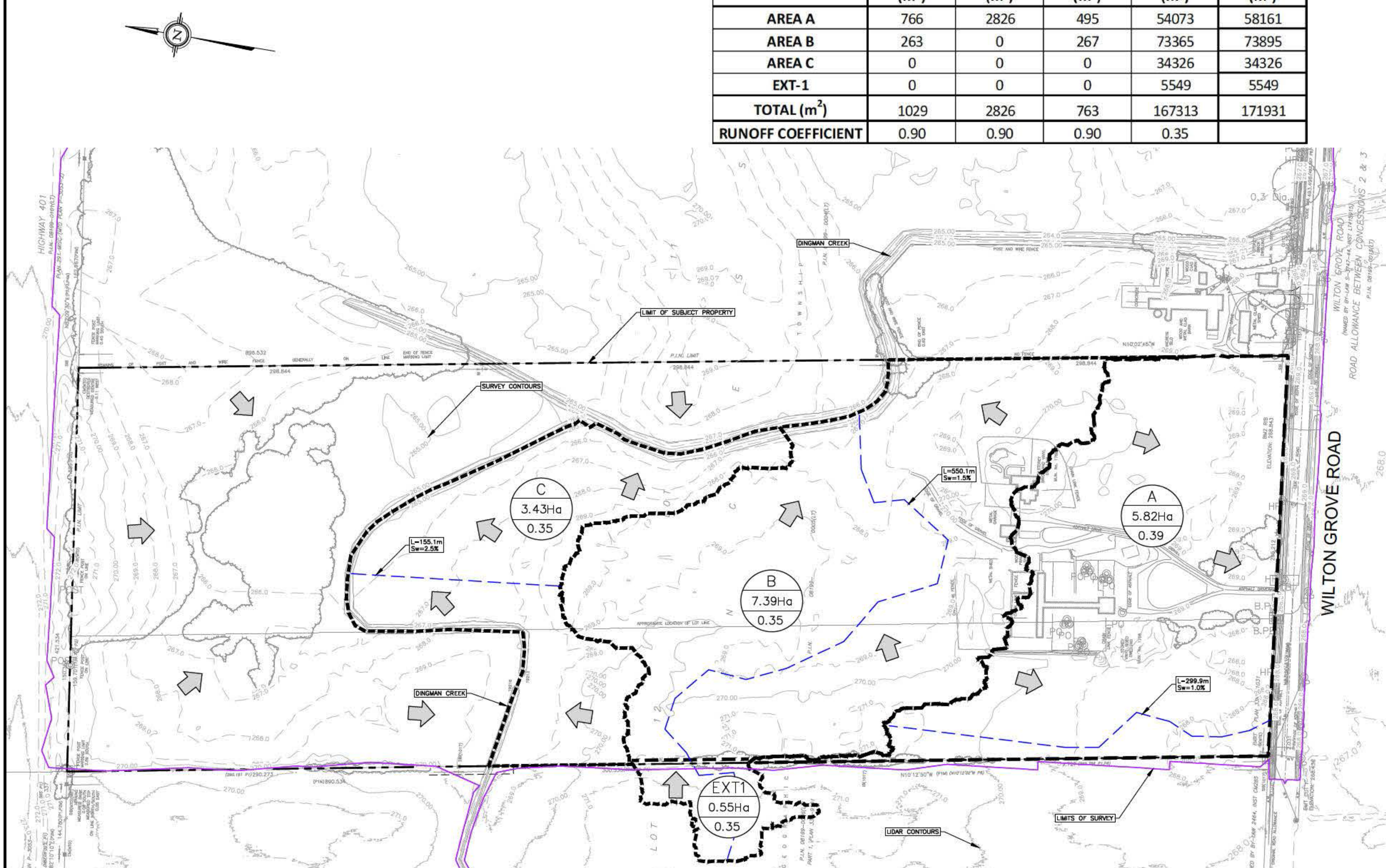
A summary of the pre-development land cover is provided below in **Table 8-1**.

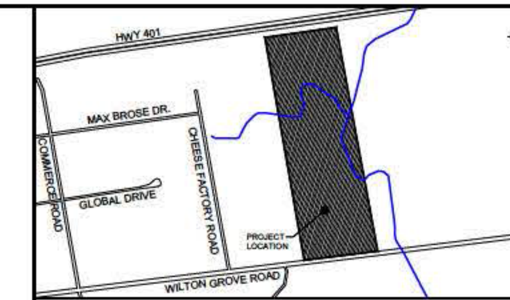
Table 8-1: Predevelopment Land-Use Summary

Surface	Runoff Coeff.	A (m^2)	B (m^2)	C (m^2)	EXT-1 (m^2)	Coverage
Roof	0.90	766	263	0	0	0.6%
Driveway	0.90	2,826	0	0	0	1.6%
Hardscape	0.90	495	267	0	0	0.4%
Grassed	0.35	54,073	73,365	34,326	5,549	97.3%
Total	0.36	58,160	73,895	34,326	5,549	100.0%

The total imperviousness of the existing site was calculated to be 23%, and the corresponding runoff coefficient was calculated to be 0.36 based on the City of London Design Specifications & Requirements Manual dated October 2003 and updated August 2019. For more details, please see **Appendix G**.

EXISTING CONDITIONS					
	ROOF (m^2)	DRIVEWAY (m^2)	HARDSCAPE (m^2)	GRASSED (m^2)	TOTAL (m^2)
AREA A	766	2826	495	54073	58161
AREA B	263	0	267	73365	73895
AREA C	0	0	0	34326	34326
EXT-1	0	0	0	5549	5549
TOTAL (m^2)	1029	2826	763	167313	171931
RUNOFF COEFFICIENT	0.90	0.90	0.90	0.35	





KEY PLAN
N.T S.

LEGEND

EXISTING

- EASEMENT
- RIGHT OF WAY
- LOT LINE
- FENCE LINE
- CURB/SIDEWALK
- BUILDING
- MAJOR CONTOUR LABEL
- MINOR CONTOUR LABEL
- SWALE
- OVERLAND FLOW
- STORM DRAINAGE BOUNDARY
- A — DENOTES AREA NUMBER
- 1.312Ha — DENOTES AREA IN HECTARES
- 0.66 — DENOTES RUNOFF COEFFICIENT
- LONGEST TRAVEL PATH

NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. CONTOUR INTERVAL IS 0.50m.
3. LIDAR DATA SOURCE: ONTARIO LIDAR 2016-2018 LAKE ERIE PACKAGE I RAISED BY 0.45m TO CONVERT FROM CGVD2013 TO CGVD28, IN ACCORDANCE WITH TOPOGRAPHIC SURVEY

BENCHMARK

BENCHMARK NUMBER: CITY OF LONDON MONUMENT BM02-107
TYPE: BOLT
GEODETIC ELEVATION = 268.832m, DATUM: CGVD28:78
LOCATION: CITY OF LONDON
DESCRIPTION:
CONCRETE BOX CULVERT ON OLD VICTORIA ROAD 100m+/- SOUTH OF WILTON GROVE ROAD. BOLT SET IN THE EAST FACE OF CULVERT ON THE EAST SIDE OF OLD VITORICA ROAD, 0.18M SOUTH OF THE NORHTEAST CORNER.
COMPLETED BY:
CALLON DIETZ INC. O.L.S.
41 ADELAIDE STREET UNIT 1, LONDON, ON N6B 3P4
(519) 673-0220
COMPLETED ON: JULY 21, 2022

Greck
5770 Highway 7, Woodbridge, Ontario, L4L 1T8 www.greck.ca

CLIENT NAME:
DANCOR CONSTRUCTION LTD.
16 MELANIE DRIVE, SUITE 101
BRAMPTON, ONTARIO, CANADA
L6T 4K9

PROJECT NAME:
PROPOSED INDUSTRIAL BUILDING
1766 WILTON GROVE ROAD, LONDON, ON

DRAINAGE PLAN EXISTING CONDITIONS

DESIGNED BY:	E.P.	SCALES:		PROJECT No.	23-967
CHECKED BY:	E.P.	HORIZONTAL:	1:3000	DRAWING No.	FIG.2
DRAWN BY:	A.J.	VERTICAL:	N/A	SHEET No.	
DATE: SEPT. 12, 2024		SHEET SIZE:	11"x17"		

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Date Plotted: Sep 12, 2024 - 3:04 PM

8.2 Proposed Drainage

Under proposed conditions, the development limits have been delineated into four (4) drainage areas: Area A, B, C, and EXT-1. Please see **Figure 3** below for the post-development drainage area plan. The proposed landuse of the development area will consist of industrial building roofs, asphalt, and landscaped areas.

A description of each proposed drainage area is summarized below:

- Area A (8.67ha) consists of the asphalt driveway and parking lots. Runoff from Area A will be captured by catchbasins and conveyed by storm sewers which ultimately discharge to Dingman Creek.
- Area B (5.68ha) consists of the industrial building roof areas. Area B has been further delineated into three (3) drainage areas: B1, B2, and B3.
  - Area B1 and B2 represents the industrial building roofs located in the center and the south of the development area, where runoff will be captured by roof drains and collected in the storm sewers which ultimately discharge to Dingman Creek
  - Area B3 represents the industrial building roof where runoff will be captured by roof drains and collected in the storm sewers which ultimately discharges towards the north tributary of Dingman Creek and the wetland.
- Area C (2.29ha) consists of landscaped areas. Area C drains north towards the north tributary of Dingman Creek and the wetland.
- Area EXT-1 (0.55ha) represents external drainage area from the west which consists of agricultural fields. Area EXT-1 drains east via overland flow towards Area A and will be collected by storm sewers which ultimately discharge to Dingman Creek

The roofs, asphalt, and landscaped areas were assigned a runoff coefficient of 0.90 as per the City of London Design Specifications & Requirements Manual dated October 2003 and updated August 2019. The pervious areas were assigned a runoff coefficient of 0.35 as per MTO Design Chart 1.07 for cultivated landuse at 0-5% slopes with loam or silt loam soils.

A summary of the post-development land cover is provided below in **Table 8-2.**

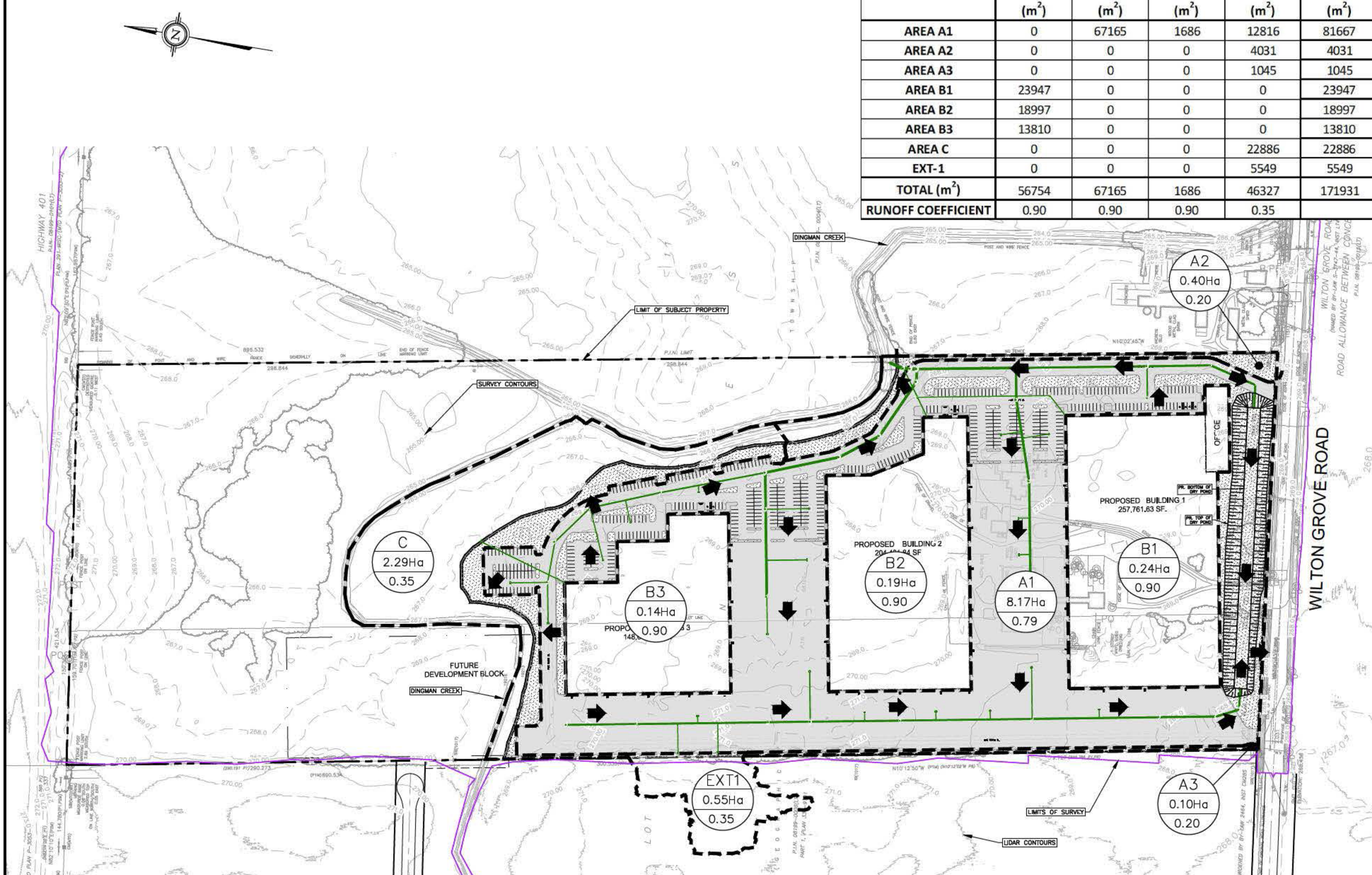
Table 8-2: Post-Development Land-Use Summary

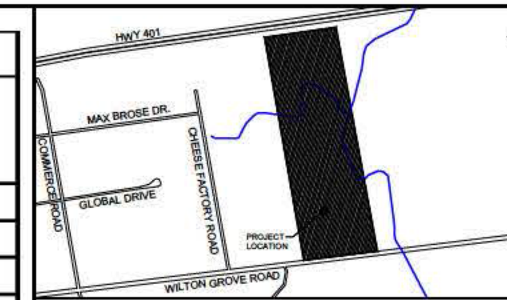
Surface	Runoff Coeff.	A (m^2)	B (m^2)	C (m^2)	EXT-1 (m^2)	Coverage
Asphalt	0.90	0	56,754	0	0	33.0%
Hardscape	0.90	67,165	0	0	0	39.1%
Roof	0.90	1,686	0	0	0	1.0%
Grassed Area	0.35	17,892	0	22,886	5,549	26.9%
Total	0.50	86,743	56,754	22,886	5,549	100.0%

The total imperviousness of the proposed site was calculated to be 42%, and the overall runoff coefficient of the proposed site was calculated to be 0.50 based on the City of London Design Specifications & Requirements Manual dated October 2003 and updated August 2019. For more details, please see **Appendix G**.

The 2-year and 100-year storm events are proposed to be serviced by the storm sewer system. The 250-year storm event will be conveyed overland via overland flow routes. Overland flow route capacity calculations will be completed in detailed design.

PROPOSED CONDITIONS					
	ROOF (m^2)	DRIVEWAY (m^2)	HARDSCAPE (m^2)	GRASSED (m^2)	TOTAL (m^2)
AREA A1	0	67165	1686	12816	81667
AREA A2	0	0	0	4031	4031
AREA A3	0	0	0	1045	1045
AREA B1	23947	0	0	0	23947
AREA B2	18997	0	0	0	18997
AREA B3	13810	0	0	0	13810
AREA C	0	0	0	22886	22886
EXT-1	0	0	0	5549	5549
TOTAL (m^2)	56754	67165	1686	46327	171931
RUNOFF COEFFICIENT	0.90	0.90	0.90	0.35	

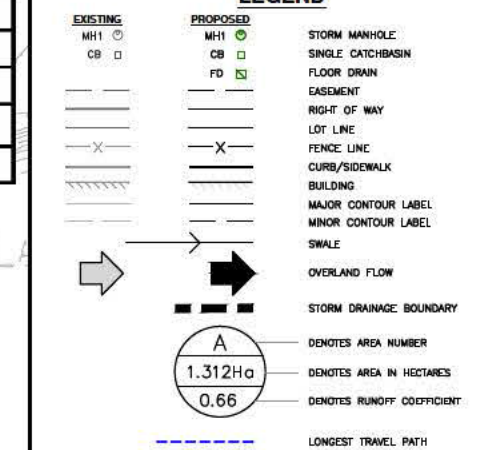




KEY PLAN

N.T.S.

LEGEND



NOTES

1. ALL DIMENSIONS ARE IN METRES UNLESS OTHERWISE NOTED.
2. CONTOUR INTERVAL IS 0.50m.
3. LIDAR DATA SOURCE: ONTARIO LIDAR 2016–2018 LAKE ERIE PACKAGE I RAISED BY 0.45m TO CONVERT FROM CGVD2013 TO CGVD28, IN ACCORDANCE WITH TOPOGRAPHIC SURVEY

BENCHMARK

BENCHMARK NUMBER: CITY OF LONDON MONUMENT BM02–107
TYPE: BOLT
GEODETIC ELEVATION = 268.832m, DATUM: CGVD28:78
LOCATION: CITY OF LONDON
DESCRIPTION:
CONCRETE BOX CULVERT ON OLD VICTORIA ROAD 100m+/– SOUTH OF WILTON GROVE ROAD. BOLT SET IN THE EAST FACE OF CULVERT ON THE EAST SIDE OF OLD VITORICA ROAD, 0.18M SOUTH OF THE NORHTEAST CORNER.
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DESIGNED BY:	E.P.	SCALES:		PROJECT No.	23-967
CHECKED BY:	E.P.	HORIZONTAL:	1:3000	DRAWING No.	FIG.3
DRAWN BY:	A.J.	VERTICAL:	N/A	SHEET No.	
DATE: SEPT. 12, 2024		SHEET SIZE:	11"x17"		

8.3 PCSWMM Assessment

An unsteady state PCSWMM model was developed to characterize peak flows for the 2-year through 250-year design storms for the development. PCSWMM was used as it will more accurately characterize runoff characteristics for the site including details pertaining to infiltration and storage parameters compared to modified rational method. PCSWMM outputs are provided in **Appendix H**.

8.3.1 Subcatchment Delineation

The existing drainage catchments were delineated based on the topographic survey and LIDAR DTM. The proposed drainage catchments were delineated based on the proposed grading design as per the Site Grading Plan (**Drawing C101** provided in **Appendix J**).

8.3.2 Storm Distribution

To select the storm distribution for the PCSWMM analysis, the 100-year peak flow generated under various storm distributions were compared using the 100-year IDF parameters reported in the City of London Design Specifications & Requirements Manual dated October 2003 and updated August 2019. This included the AES 1 hour, Chicago 3 hour and 6 hour, SCS Type II 6 hour, 12 hour, and 24 hour storm distribution. The Chicago 6 hour produced the greatest 100-year peak flow under proposed conditions and will be the storm distribution to model the 2-year to 250-year storm events in PCSWMM.

8.3.3 Flow Length and Slope

The flow length and the slope of each subcatchment is modelled in the PCSWMM model to account for the timing of peak runoff for each subcatchment. The maximum flow length and slope was measured for each subcatchment under existing and proposed conditions. The existing flow lengths were measured using the topographic survey, and the proposed flow lengths were measured based on the proposed grading design as per the Site Grading Plan (**Drawing C101** provided in **Appendix J**).

8.3.4 Manning Roughness Coefficient

The Manning's roughness coefficient is applied to subcatchments for overland flow and storm sewers to account for friction losses. The Drainage Management Manual by MTO, industry standard values, and engineering judgement was used to determine roughness values for the impervious and pervious areas within a subcatchment, and the

City of London Design Specifications & Requirements Manual was used to determine roughness values for the storm sewer pipe.

A summary of the Manning's roughness coefficient is provided **Table 8-3** below.

Table 8-3: Manning's Roughness Coefficient Summary

Surface	Manning's Roughness
Impervious	0.013
Pervious	0.250
Storm Sewer Pipe	0.013

8.3.5 Depression Storage

Impervious and pervious surfaces provide depression storage that stores precipitation that would otherwise become runoff. The depression storage assigned to the subcatchments for the impervious and pervious areas was determined based on industry standard values and engineering judgement and are summarized in **Table 8-4**.

Table 8-4: Depression Storage Summary

Surface	Depression Storage (mm)
Impervious: Roofs, Driveways, Hardscape	2
Pervious: Landscaped Areas, Trees	5

8.3.6 Infiltration Parameters

The Green-Ampt Method was used to simulate infiltration on PCSWMM. According to the Preliminary Geotechnical Investigation by EXP dated August 5th 2022, the surficial soils in the area are predominantly sandy silt and clayey silt.

Parameters including hydraulic conductivity, initial deficit ratio and suction head were based on a silt loam soil texture class as reported in the "Green-Ampt Infiltration Parameters from Soils Data" technical paper by Rawls et al. The assigned infiltration parameters are provided in **Table 8-5** below.

Table 8-5: Infiltration Parameter Summary

Suction Head	169.93 mm
Conductivity	6.6 mm/hr
Initial Deficit	0.366

8.3.7 PCSWMM Flow Outputs

The PCSWMM flow outputs for the 2-year to 250-year storm events under existing and proposed conditions are summarized in **Table 8-6**.

Table 8-6: PCSWMM Flow Outputs

Storm Event	Existing (m^3/s)	Proposed (m^3/s)
2-year	1.393	5.059
5-year	1.437	5.218
10-year	1.734	6.189
25-year	2.144	7.461
50-year	2.461	8.406
100-year	2.783	9.316
250-year	3.452	11.19

The post-development flows greatly exceed pre-development flows, as such, water quantity controls are required to ensure no net increase in peak flows occurs.

9. Stormwater Management

The site is located within the Dingman Creek Screening Area, and therefore is subject to the SWM targets outlined in the Dingman Creek Subwatershed: Stormwater Servicing Study prepared in September 2020.

The following SWM criteria is to be addressed in accordance with regulatory policy:

- Water quality
- Water quantity
- Extended Detention/Erosion control
- Water balance

SWM is only proposed for Area A and B, which consists of the proposed development limits. No SWM is proposed for Area C, where only minor grading is proposed as no impervious surfaces are proposed within Area C.

The proposed SWM strategy only includes considerations for water quantity control, water quality control, extended detention/erosion control, and water balance for the site. The proposed SWM strategy includes a treatment train approach featuring the following SWM controls:

- Catchbasin manhole and storm manhole debris/sediment traps,
- Oil and Grit Separator (OGS) unit,
- Oversized pipe storage, ponding storage, dry pond, roof storage, flow control orifice and weir

Runoff generated from the road, hardscape, and landscape surfaces will be collected by catchbasins and catchbasin manholes. The runoff captured by the catchbasin and catchbasin manholes will be controlled by an orifice and weir to attenuate flow, and storage is provided via oversized pipe storage, dry pond, and ponding storage. Further downstream, the runoff is treated by an OGS unit. The stormwater ultimately discharges to Dingman Creek via the proposed headwater outlet structure.

Roof runoff from the proposed building is collected by roof flow control drains which attenuate flow, and storage is provided via roof storage. Roof runoff then discharges into stormsewers where the stormwater ultimately discharges to Dingman Creek via the proposed headwater outlet structure.

The stormwater management criteria and how the proposed development will achieve them is discussed in greater detail in the following sections below.

9.1 Water Quality

Stormwater from the development will be characterized by runoff from roofs, hardscape, landscape, and roadway surfaces. The main contaminants of concern being:

- Suspended sediments
- Other (oil, grease, gas, temperature)

Area B only consists of roof areas which are clean surfaces and do not contribute to 'dirty' runoff, as such, water quality controls are only required for Area A.

The feasibility of infiltration was assessed:

- As per the geotechnical report by EXP, the groundwater table ranges from 0.86m to 1.50m below the ground surface (readings taken from May 30th 2022). Therefore, infiltration is not proposed for the at-grade areas, as it is not possible to provide sufficient cover for pipes and the infiltration facility, and a 1m separation from the bottom of the infiltration facility to the groundwater table, as recommended for infiltration facilities as per the MOE Stormwater Management Planning and Design Manual

Due to the reasons above, no infiltration is proposed. The runoff generated by the asphalt and hardscaped areas are to be treated in the form of suspended solids removal. No water quality controls are required for roof runoff since this is considered to be clean.

Water quality controls for runoff generated on asphalt and hardscaped surfaces, the proposed level of suspended solids removal treatment is MOE Enhanced Protection Level (Level 1). This corresponds to a long-term average removal of 80% of total suspended solids (TSS).

A treatment train approach is proposed for capturing and treating contaminated runoff:

- Firstly, a debris/sediment trap is proposed in all catchbasins and catchbasin manholes. This trap will prevent suspended solids from entering the outlet pipe, and allow it to settle within the sump of the manhole structures. A 600mm deep sump is proposed at each manhole structure to allow for the settling of debris and sediment.
- Secondly, an OGS will provide stormwater treatment by trapping free oils and floatable sediments, and settling any captured sediment. The OGS has been sized to provide 60% TSS removal using the CA ETV particle size distribution. To be conservative, it is assumed that the OGS will only provide 50% TSS removal.

The OGS unit will provide a total of 50% TSS removal. It is expected that the proposed debris/sediment trap at each catchbasin and catchbasin manhole would provide the additional 30% TSS removal required to meet the 80% TSS removal target. The OGS will be sized in detail design.

9.2 Water Quantity

The increase in impervious surfaces due to development causes a rise in post-development flows. To mitigate this, the water quantity criteria is to control post-development flows to pre-development levels for the 100-year to 2-year storm events.

The storage requirement to reduce the 100-year post-development flow to pre-development levels was calculated on PCSWMM. PCSWMM estimates the storage requirement by measuring the area of the hydrograph between the post-development and target flow rate. See **Figure 4** for the PCSWMM storage requirement assessment.

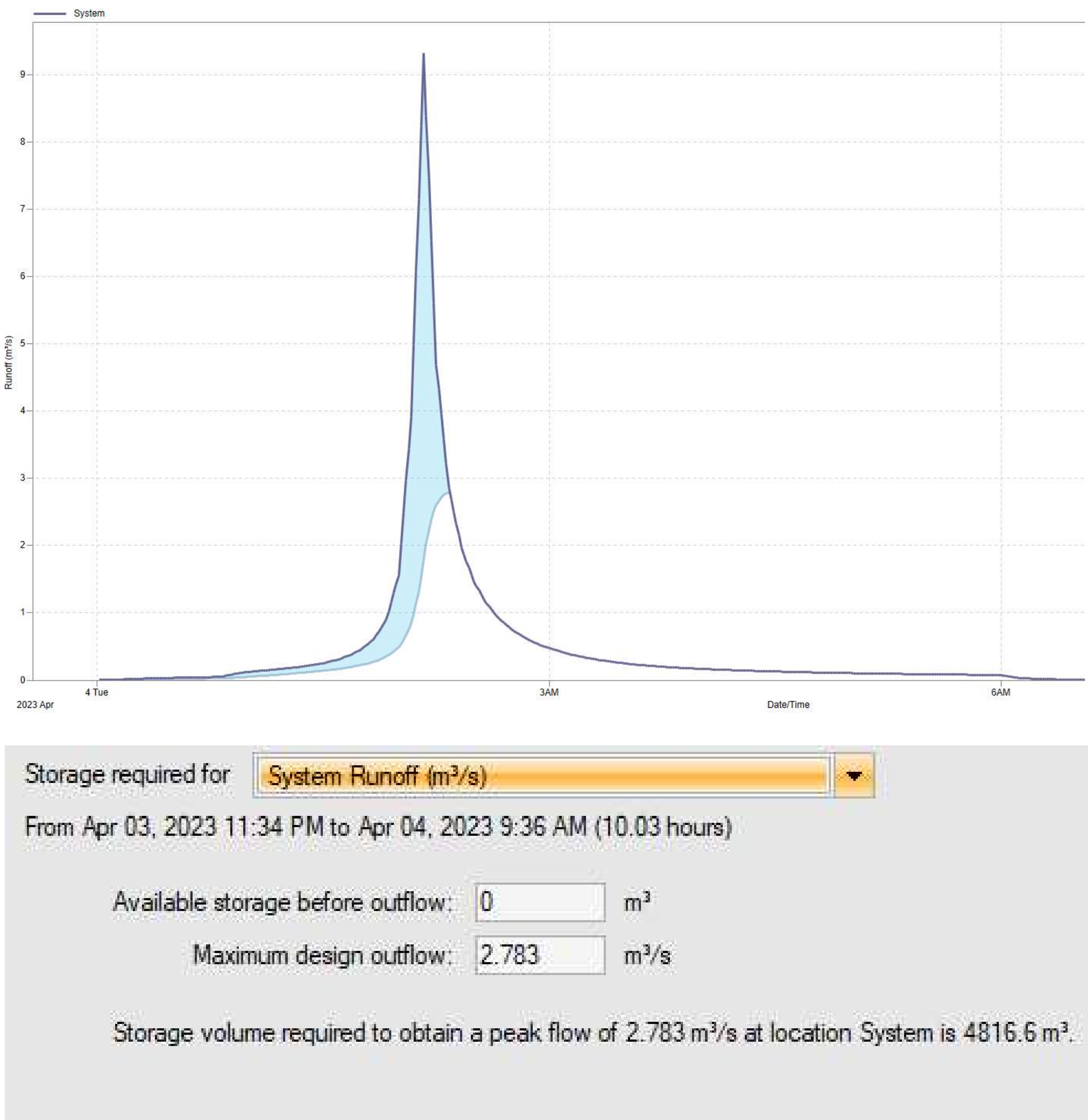

Figure 4: PCSWMM 100-year Storage Requirement Calculation

Stormwater quantity control for Area A is to be provided via underground pipe storage ponding storage, and dry pond storage. Stormwater management control for Area B is to be provided via roof storage. These controls will retain and attenuate stormwater runoff to meet the target flow rate. To achieve the water quantity criteria for Area A, a control manhole with a 175mm diameter orifice and 1m wide weir is proposed in MH42.

To achieve the water quantity criteria for Area B, Zurn Z105 roof drains are proposed to attenuate roof runoff. The number of Zurn Z105 roof drains for each building is summarized below:

- 27 Zurn Z105 roof drains for Area B1
- 22 Zurn Z105 roof drains for Area B2
- 16 Zurn Z105 roof drains for Area B3

A summary of the available storage provided by the underground pipes, ponding, dry pond and the roofs is summarized below in **Table 9-1**.

Table 9-1: Storage Availability

Storage	Storage Available (m^3)
Underground Pipe Storage	975
Ponding Storage	1,355
Dry Pond	12,228
Roof Storage	8,513
Total Storage Available	23,071
100-year Volume Requirement	4,817

A summary of peak flows and the target release rate is provided in **Table 9-2**.

Table 9-2: PCSWMM Flow Outputs – Water Quantity Control

Storm Event	Existing (m^3/s)	Proposed with SWM (m^3/s)
2-year	1.393	0.500
5-year	1.437	0.519
10-year	1.734	0.618
25-year	2.144	0.757
50-year	2.461	1.064
100-year	2.783	1.305

The proposed water quantity controls will reduce post-development flows below pre-development levels for the 2-year to 100-year storm events.

The sizing and configuration of the roof storage, roof drains, pipe storage, surface ponding, dry pond, orifice and weir will be confirmed during detailed design. The roof plan and number of roof drains is to be confirmed and provided by the building's mechanical engineer during detailed design.

PCSWMM outputs are provided in **Appendix H**.

9.3 Extended Detention

Extended detention will be provided on site to ensure that the greater of 40m^3/ha (as per MECP SWMP Manual) or the runoff volume from the 25mm 4-hour Chicago storm is detained for at least 24 hours. A 175mmmm diameter orifice is proposed at MH42 to provide extended detention for Area A, and Zurn Z105 flow control roof drains will provide extended detention for Area B.

A summary of the extended detention volumes for Area A and Area B are provided below in **Table 9.3** respectively.

Table 9.3: Extended Detention Volume Requirements

Area	A1	B1	B2	B3
Drainage Area (ha)	8.17	2.40	1.90	1.38
Unitary Extended Detention Volume (m^3/ha)	40	40	40	40
Extended Detention Volume (m^3)	326.7	95.8	76.0	55.2
25mm 4-hour Chicago Volume (m^3)	1597.7	605.2	480.1	348.9

The 25mm 4-hour Chicago runoff volume is greater than the MOE extended detention volume requirement. Therefore, the extended detention volume requirement is governed by the 25mm 4-hour Chicago storm event. The outflow hydrograph from the development area during the 25mm 4-hour Chicago storm under proposed conditions are presented below in **Figure 5**.

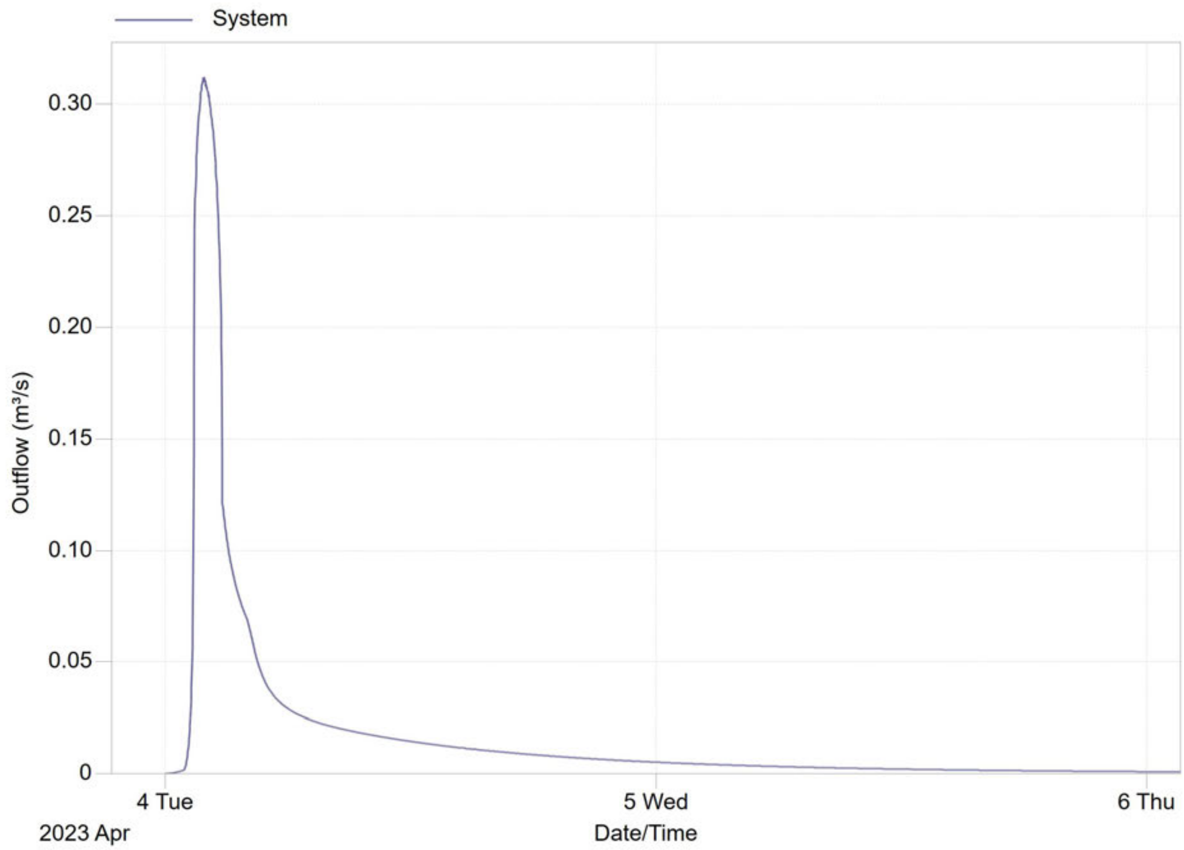

Figure 5: 25mm 4-hr Chicago Storm Outflow Hydrograph

From **Figure 5**, runoff from the 25mm 4-hour Chicago storm event will be retained and released in 48 hours, which meets the drawdown time requirement of 24 to 48 hours recommended as per the MOE SWM Planning & Design Manual.

Extended detention control measures will be confirmed in detailed design. Extended detention calculations are provided in **Appendix G** and PCSWMM outputs are provided in **Appendix H**.

9.4 Water Balance

Urbanization increases impervious cover, which, if left unmitigated, results in a decrease in infiltration. This infiltration reduces groundwater recharge and soil moisture replenishment. It also reduces stream baseflow needed for sustaining aquatic life. Therefore, it is important to maintain the natural hydrologic cycle. Groundwater recharge helps maintain aquifer water levels and supports significant watershed features that are necessary components to the maintenance of a healthy watershed. As a result, a water balance analysis is required to estimate the pre-development and post-development infiltration and runoff.

A site-specific water balance was completed for the development area using MOE's "Stormwater Management Planning and Design Manual", March 2003. This approach uses the method developed by Thornthwaite and Mather. A summary of the previous and impervious areas is provided below in **Table 9-4**.

Table 9-4: Existing and Proposed Land Cover

Area 201, 202, 203	Existing (m^2)	Proposed (m^2)
Pervious	117,113	46,348
Impervious	96	70,863
Total	117,209	117,211

The parameters used for the water balance analysis are provided in **Table 9-5**.

Table 9-5: MOE Water Balance Infiltration Parameters

	Comment	Factor
Topography	Hilly Land	0.1
Soils	Tight Impervious Clay	0.1
Cover	Cultivated Land	0.1

An annual precipitation of 961.6mm was determined. Assuming that 10% of the rainfall is evaporated, an impervious annual surplus of 865.4mm was determined.

A summary of the infiltration volumes is provided in **Figure 6**.

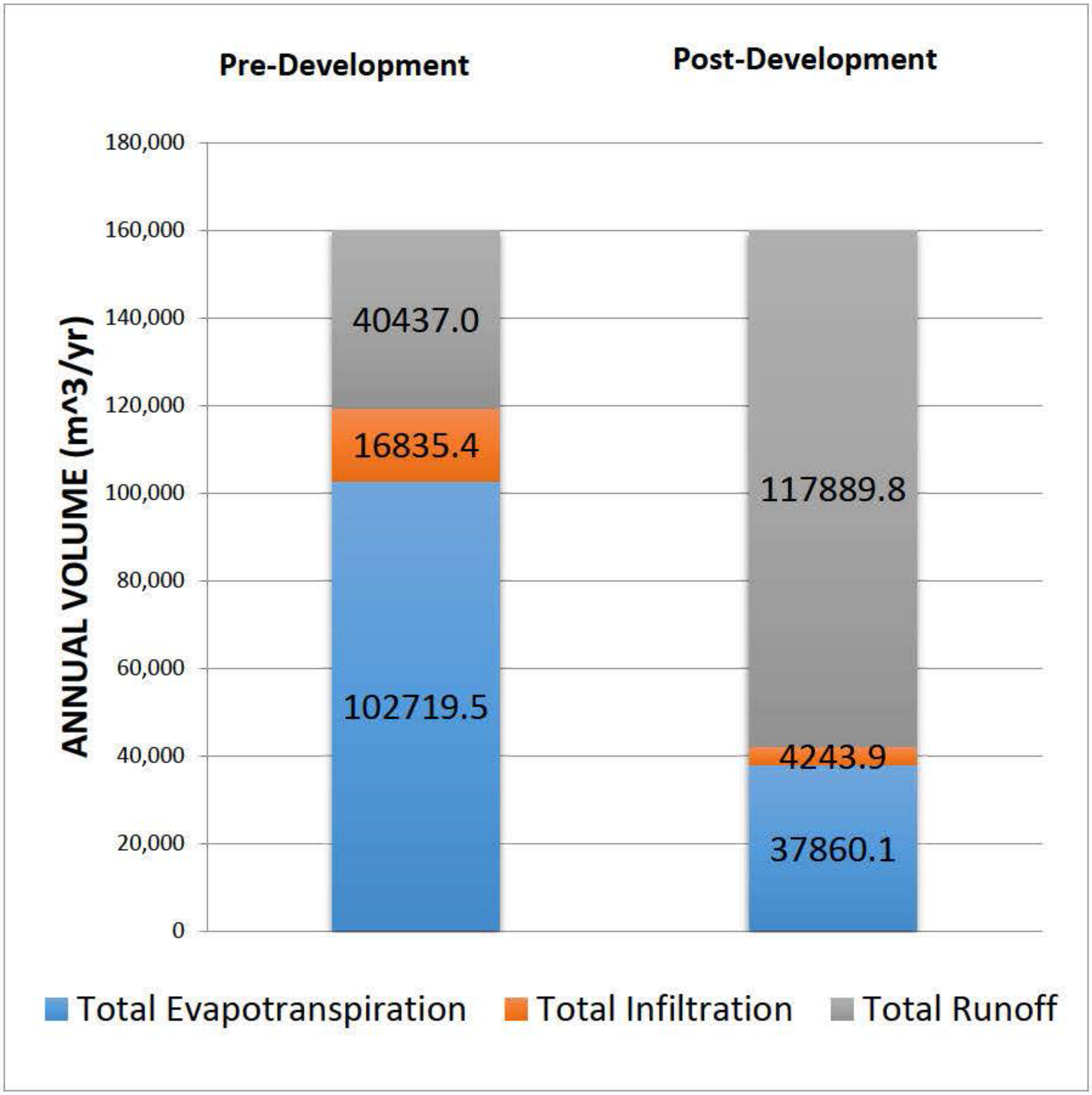

Figure 6: Water Balance Summary

A total deficit volume of 12,591.5m^3/year will not be infiltrated into the ground given the proposed development plan and resulting change in pervious cover.

As mentioned in Section 8.1, the feasibility of infiltration under proposed conditions was assessed. As per the geotechnical report by EXP, the groundwater table ranges from 0.86m to 1.50m below the ground surface (readings taken from May 30th 2022). Therefore, infiltration is not proposed, as it is not possible to provide sufficient cover for pipes and the infiltration facility, and maintain a 1m separation from the bottom of the infiltration facility to the groundwater table, as recommended for infiltration facilities as per the MOE Stormwater Management Planning and Design Manual.

Under existing conditions, a portion of the development area drains towards the adjacent wetland. To ensure that the proposed works will not impact the quantity of runoff the adjacent wetland receives, it is proposed to direct clean roof runoff from Area B3 towards the wetland. A comparison of the runoff contribution to the wetland under existing and proposed conditions was completed. See **Table 9-6** for a summary of the results.

Table 9-6: Wetland Runoff Contribution Assessment

	Existing Runoff (m^3/year)	Proposed Runoff (m^3/year)
Runoff Contribution to Wetland	7,733	17,108

Under proposed conditions, the wetland will receive an increase in 9,375m^3/year runoff volume, which compensates for the reduction in infiltration volumes. As such, the proposed works will not negatively impact the runoff contribution to the wetland when compared to existing conditions.

For water balance calculations, please see **Appendix G**.

9.5 Operations and Maintenance

The rate at which sediments enter the system will determine the longevity and performance of the proposed treatment train. Overtime, sediment will fill the sumps of the catchbasins, catchbasin manholes, storm manholes, and with improper maintenance and care, can overflow and clog the storm sewer system. The OGS unit will also have to be maintained in order to prevent a reduction in performance. Therefore, regularly scheduled inspection and maintenance is recommended in order to prolong the longevity and performance of the treatment train. The following items are recommended:

- The driveways and parking lot should be regularly inspected for debris and trash accumulation on the surface and disposed of as necessary.
- The sumps of the catchbasins, catchbasin manholes, storm manhole, and the OGS unit should be inspected every six months for the first year to determine the pollutant accumulation rate. In subsequent years, inspections can be based on observations. The sumps are recommended to be cleaned using a vacuum truck annually.
- Maintenance for the dry pond should not be required if the sumps of the catchbasins, catchbasin manholes, storm manholes are inspected and cleaned regularly. However, scheduled visual inspections are still recommended to

assess the dry pond for any sediment build up. If the depth of sediment within the dry pond is in excess of 3 inches, the dry pond must be cleaned out. The dry pond is recommended to be cleaned using a vacuum truck annually.

If maintenance is required, the maintenance of the catchbasin, catchbasin manhole, storm manhole, and OGS unit should be coordinated and scheduled at the same time. After the first year, it is recommended to inspect all the above items on an annual basis.

See **Appendix I** for operation and maintenance guides, checklists and recommendations provided by the suppliers for additional information.

10. Erosion and Sediment Control

Erosion and sediment controls (ESC) will be implemented for all construction activities, including topsoil stripping, material stockpiling, pavement construction, and grading operations. Please see **Drawing C104** for the ESC plan which has been provided in **Appendix J**.

The following highlights key elements of the ESC plan:

- Sediment control fence – Fencing will be constructed downslope of the proposed development area prior to all construction activities. Geotextile material should have a non-woven density of 270R or equivalent;
- A mud mat, consisting of a geotextile overlain by clear stone will be placed at the access to the site during construction to prevent equipment and vehicles from tracking sediments off-site.
- Straw bale flow check dams are to be placed within drainage swales and low points to hold back water and reduce velocities to prevent erosion and promote sedimentation.
- Siltsacks are to be installed in active catch basins and floor drains to filter any stormwater leaving the construction area to prevent sediment from entering the drainage system.
- Native seeding – all exposed soil after grading is to be immediately seeded to promote vegetation growth and protection for erosion and sediment control.
- ESCs will be erected prior to the start of construction works and maintained through all phases of development. ESC strategies are not static and may need to be upgraded/amended as site conditions change to minimize sediment laden runoff from leaving the work areas;
- Sediment controls must be inspected on a regular basis and after every rainfall event. Repairs must be done in a timely manner to prevent movement of sediment.

11. Conclusions

As presented in this report, the proposed development will meet the following municipal and provincial standards and regulations specified for:

- General site grading;
  - Two retaining walls are proposed with lengths of 110.5m and 33.3m.
  - A net fill of 27,668m^3 is anticipated.
- Water distribution;
  - A new 200mm diameter PVC watermain will provide servicing for the subject property.
  - External watermain servicing is required to connect the subject property to the existing 900mm diameter concrete watermain located at the intersection of Wilton Grove Road and Cheese Factory Road.
  - Ten (10) private hydrants are proposed throughout the site to adequately service the 3 proposed industrial buildings.
  - The recommended fire flow for the proposed development is 283.33L/s.
  - Hydrant flow testing indicates that a maximum of 158.2L/s is available at a residual pressure of 20psi.
  - Therefore, the existing watermain infrastructure is unable to adequately service the subject property in the case of fire flows.
  - The total water demand for the site including both domestic and fire flows is 304.21L/s.
- Sanitary sewer servicing;
  - A new sanitary sewer will provide servicing for the subject property.
  - Approximately 390m of external sanitary sewer servicing is required to connect the proposed sanitary sewer to the existing 375mm diameter sanitary sewer located on Max Brose Drive.
  - Sanitary sewer flows from the subject property are not able to be conveyed by gravity to the connection to the 375mm diameter sanitary sewer on Max Brose Drive, so a pumping station is proposed.
  - The total sanitary design flow for the subject property is 17.96L/s
- Stormwater management;
  - Water quantity control is provided via an orifice and a weir, underground pipe storage and ponding storage for the at-grade areas, and roof storage and flow control roof drains for the roof areas.
  - Water quality control is provided via a treatment train approach featuring debris/sediment traps proposed at every catchbasin and catchbasin manhole and OGS unit.

- Extended detention control is provided via an orifice and underground pipe storage for the at-grade areas, and roof storage and flow control roof drains for the roof areas.
- A water balance assessment has been completed and the proposed works will not negatively impact the runoff contribution to the wetland when compared to existing conditions.

- Construction erosion and sediment controls.

In summary, it has been determined that the development can be serviced with existing and proposed infrastructure that is in accordance with policies and guidelines required by the City of London, UTRCA, and other regulating agencies.

12. References

Aquafor Beech Limited – Dingman Creek Subwatershed: Stormwater Servicing Study, September 2020

City of London – Design Specifications & Requirements Manual, October 2003 (Updated August 2019)

Upper Thames River Conservation Authority – Environmental Planning Policy Manual, June 2006 (Updated October 2017)

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