

FUNCTIONAL SERVICING AND STORMWATER MANAGEMENT REPORT IN SUPPORT OF SITE PLAN APPROVAL APPLICATION

320 McCowan Road
City of Toronto
Scarborough District



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File Number: 19066

Prepared For:
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4	Issued for Re-Zoning Approval	May 19, 2023
3	Issued for Site Plan Approval	July 28, 2021
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No.	Revision	Date



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Appendix B	Water Demand Calculations Hydrant Pressure Flow Test prepared by Lozzi Aqua Check
Appendix C	Sanitary Design Flow Calculations Downstream Sanitary Capacity Design Sheets HGL Analysis Hydrogeological Investigation, Prepared by EXP Services Inc. Groundwater Pump Letter, Prepared by United Engineering Inc.
Appendix D	Stormwater Management Calculations Irrigation Demand Calculations On-Site Dye Testing Investigation, Prepared by I ² S



1.0 INTRODUCTION

1.1 BACKGROUND

Counterpoint Engineering Inc. has been retained by Blauson Assets Management Ltd. c/o Ling Kee Group to prepare this Functional Servicing and Stormwater Management Report (“FSR”) in support of a Re-Zoning Approval application. The application is in support of a proposed high-rise residential building at 320 McCowan Road in the City of Toronto.

The subject site is located on the west side of McCowan Road south of Eglinton Avenue East and immediately north of the Metrolinx rail corridor. The site is bound by mixed use; Employment lands to the west, high-rise residential to the north, a Metrolinx rail corridor to the south, and Landmark Boulevard to the east (with big-box commercial retail lands on the east side of McCowan Road). **Figure 1 – Site Location** illustrates the subject site within the context of its surroundings. Existing underground servicing infrastructure within an easement immediately north of the site. The **1.689 ha** site currently consists of an above-grade parking structure and associated driveway areas and landscaped areas in addition to an existing high-rise residential building and amenity building. The latter two buildings will remain untouched. As such, the developable area of the site considered in this Report is **0.81 ha**. For the existing conditions, refer to the topographical survey of the site included in **Appendix A**.

The proposed development will entail removing the existing parking structure. Upon redevelopment, the proposed building will consist of 29 & 25-storey residential point towers with a 6-storey podium. There will be a total of 483 residential units with a single level of underground parking. The proposed residential building will consist of parking on the ground floor through floor 5 along the south portion of the building immediately adjacent to the rail corridor. Vehicular access will be via Landmark Boulevard which is an access road off McCowan Road. Refer to **Appendix A** for the proposed Site Plan prepared by BDP. Quadrangle Architects Limited.

This FSR has been prepared to address the site servicing strategy (stormwater, sanitary, and water) in support of a Re-Zoning Approval application. The proposed servicing works (including stormwater conveyance) will be designed to meet City of Toronto Design Guidelines.



SITE LOCATION

MCCOWAN DISTRICT PARK

counterpoint
ENGINEERING



COUNTERPOINT ENGINEERING INC.
8395 Jane St., Suite 100, Vaughan, ON L4K 5Y2 Phone 905.326.1404 Fax 905.326.1405

SITE LOCATION PLAN

PROPOSED DEVELOPMENT
RIOCAN HALL
TORONTO, ON
SCALE: NTS

DATE: FEBRUARY 2020

PROJECT No. 19066

FIGURE No. 1



2.0 STUDY PARAMETERS

This functional servicing and stormwater management report is based on the review of the following documents and drawings:

- **Architectural Plans** prepared by BDP. Quadrangle Architects Limited
- **Hydrogeological Investigation** prepared by EXP Services Inc.
- **Plan and Profile Drawings, Eglinton Avenue East (1388, B-71315), West Driveway, Easement 557 (5673-105), Landmark Boulevard (5673-101, 5673-102)** received from the City of Toronto
- **City of Toronto Wet Weather Flow Management Guidelines**, prepared by City of Toronto, Revised November 2009
- **City of Toronto Sewer Atlas Maps**, prepared by City of Toronto, Third Edition January 2010
- **City of Toronto Sewer and Watermains Design Criteria**, prepared by City of Toronto, Revised January 2021
- **Fire Underwriters Survey**, 1999



3.0 WATER SUPPLY

3.1 EXISTING WATER SUPPLY

The existing parking garage appears to be serviced by a connection to an existing 300mm \varnothing watermain north of the site that is an extension of the existing 300mm \varnothing watermain on Landmark Boulevard. The existing connection to the 300mm \varnothing watermain extension will be abandoned upon redevelopment. There is two existing hydrants near the development area, one just north of the site and another on the west side of Landmark Boulevard west of the existing high-rise residential building.

3.2 PROPOSED WATER SUPPLY

Based on review of the municipal easement agreement for the easement immediately north of the site, there is currently no condition that prohibits connection of the proposed service connection to the 300mm \varnothing watermain extension. As such, the proposed development will be serviced by new connections to the existing 300mm \varnothing watermain extension off of Landmark Boulevard. As per Building Code 3.2.9.7 (4), since the proposed building is higher than 84 m high, measured between grade and the ceiling level of the top storey, the towers shall be served by no fewer than two sources of water supply from a public water system. Therefore, an additional fire connection will be provided. Refer to the **Drawing S1, Servicing Plan** for the proposed watermain connection locations. There are two existing municipal hydrants in close proximity to the subject site, one north of the site and another along the west portion of Landmark Blvd just northeast of the site. However, since the City of Toronto Sewer and Watermain Guidelines states that a hydrant will not be located more than 45 m unobstructed from the siamese connection, the hydrant north of the site is proposed to be relocated such that the proposed siamese connections are within 45m unobstructed from the proposed hydrant as shown on the Servicing Plan.

The City of Toronto's design criteria states that the water demand used for watermain size selection should be sufficient to satisfy maximum day demand plus fire flow or the peak hour demand, whichever is greater. Fire flow for residential areas will not be less than 4,800 L/min for a 2-hour duration in addition to the maximum daily domestic demand, delivered with a residual pressure of not less than 140kPa. For commercial, institutional and industrial areas, the minimum fire flow available will not be less than 5,000 L/min for 4 hours, delivered with a residual pressure of not less



than 140kPa. Fire demand was calculated as per the Fire Underwriter's Survey (FUS) guidelines (1999).

Refer to **Appendix B** for the supporting calculations of the following proposed flows:

- Maximum Hour Demand = 308.1 L/min
- Maximum Day Demand = 160.2 L/min
- Fire Flow Demand (2.0 hours) = 11,000 L/min
- Maximum Day Demand plus Fire Flow Demand = **11,160 L/min** (governs)

City of Toronto design criteria dictates the following system pressure requirements:

- Average Day and Maximum Day range = 350kPa to 550kPa
- Minimum hour and peak hour range = 275kPa to 700kPa
- Minimum pressure under any non-fire demand scenario = not less than 275kPa
- Minimum residual pressure during maximum day plus fire scenarios = not less than 140kPa

A flow and pressure test was conducted (in compliance with NFPA 291) on the existing 300 mmØ watermain at a hydrant on the west side of Landmark Blvd., northeast of the subject site. Based on the flow test results the existing watermain has a static pressure of 62 psi and a residual pressure of 52 psi at a flow of 1,678 GPM. Based on these results and utilizing the accepted calculation method of the National Fire Protection Agency (NFPA 291), the available flow from this main at the minimum residual pressure allowed by City of Toronto criteria of 20 psi would be approximately 14,552 L/min. The available flow is greater than the maximum day demand plus Fire Flow Demand, therefore adequate flow and pressures can be provided in the existing system to accommodate the proposed development and no external watermain upgrades are required. Refer to **Appendix B** for the flow test results and supporting calculations.



4.0 FOUNDATION DRAINAGE

Discharge of foundation drains to municipal sewers must be in accordance with Toronto Municipal Code, Chapter 681 Sewers. The quality limits for discharge in the sewers must satisfy the limits as listed in Table 1 – Limits for Sanitary and Combined Sewer Discharge and/or Table 2 – Limits for Storm Sewer Discharge of Chapter 681. A Permit To Take Water (PTTW) from the Ontario Ministry of the Environment, Conservation, and Parks (MECP) through an online process is required for Short Term water taking between 50 m³/day and 400 m³/day. A PTTW is required for Long Term water taking from a permanent drainage system greater than 50 m³/day. A permit is required from the City of Toronto for both short-term and long-term discharges to the municipal sewer system.

A Hydrogeological Investigation was prepared by EXP Services Inc., dated April 22, 2021 for the proposed development. Refer to **Appendix C** for the main body of the Investigation.

Short-Term (Construction) – Groundwater rates are expected to be **390m³/ day (or an expected peak rate of 4.73 L/s)**. A PTTW from the MECP will be required for construction dewatering. Short-term discharge will likely be to the 250mmØ sanitary sewer northwest of the site. Temporary discharge must meet Toronto Table 1 Sanitary/Combined Sewer Discharge Limits prior to discharge to the municipal combined sewer. The Investigation indicates that the groundwater quality currently meets Table 1 – Limits for Sanitary and Combined Sewer Discharge. As such, no additional treatment is required prior to discharge. Details of Construction (short-term) dewatering will be provided by a dewatering contractor prior to construction that satisfies Toronto Municipal Code, Chapter 681 Sewers in order to obtain a short-term discharge permit from the City.

Additionally, based on the downstream sanitary sewer analysis discussed in **Section 5.2**, there is capacity in the downstream sanitary system to accommodate short-term (construction) flows from the site in dry conditions. In wet conditions, there is surcharging but the HGL is greater than 1.8m from the road elevation.



Long-Term Discharge - The Investigation indicates an estimated long-term discharge rate of **49.6 m³/day (0.57 L/s)**. However, the sanitary analysis was based on the peak pump rate of **25 US GPM (1.58 L/s)** to the 250mmØ municipal sanitary sewer northwest of the site as discussed later in **Section 5.2**. An agreement with the City of Toronto for long-term discharge will be required. The Investigation indicates that the groundwater quality currently meets Table 1 – Limits for Sanitary and Combined Sewer Discharge. Details of the sub slab foundation system, sump pit sizing, water meter location and specification and peak flow rates to be provided, by others, at the time of Long-Term Discharge Application. A sampling access point will be installed at ground level at the property line to ensure quality and quantity control is maintained.

Refer to **Drawing S1, Servicing Plan** for the location of the groundwater sampling access point and **Appendix C** for the groundwater pump letter.



5.0 SANITARY SERVICING

5.1 EXISTING SANITARY SERVICING

Existing sanitary servicing for the parking garage at 320 McCowan is likely provided by the existing 250mm \varnothing sanitary sewer located northwest of the site. Given the nature of the existing development, the existing sanitary drainage generated from the site is minimal. The existing sanitary connection for the parking garage will be abandoned to the satisfaction of Toronto Water (the exact location of the connection will be determined during Site Plan Approval). Existing sanitary flows from the existing 19 storey apartment building and the one storey building north of the subject site are directed towards the same 250 mm \varnothing sanitary sewer. The existing sanitary connections for the existing buildings will be retained to service existing residential buildings.

5.2 PROPOSED SANITARY SERVICING

Since the existing sanitary servicing for the parking garage is serviced from the 250mm \varnothing municipal sanitary sewer northwest of the existing site and a service connection to the sewer within the easement is not prohibited by the municipal easement agreement, the proposed development is to be serviced by new connections to the aforementioned existing 250mm \varnothing sanitary sewer within the easement northwest of the site to maintain the existing sanitary drainage flow pattern. Using the City of Toronto Sanitary Design criteria the equivalent population for the proposed development has been calculated to be **934 persons**. The peak sanitary flow for the proposed development has been calculated to be **11.70 L/s** which includes the peak groundwater flow of **1.58 L/s** discussed in **Section 4.0**. Refer to **Appendix C** for detailed calculations and **Appendix D** for the On-Site Dye Testing Results that confirmed the location of the existing sanitary connection.

The subject site is located within the City of Toronto Basement Flooding Study Area 52. The Class EA for this area is currently in progress (started in 2019).

As the proposed development results in an increase in sanitary flows, a downstream sanitary analysis was completed to determine if there is capacity to accommodate the increase. Based on the analysis, there is capacity in the sanitary system to accommodate the proposed development in dry conditions, however there is surcharging in proposed wet conditions (a marginal increase to the surcharging also experienced in dry wet conditions). Due to the surcharging in wet conditions, an



HGL (hydraulic grade line) analysis was completed to determine if the surcharging remained greater than 1.8m from the surface elevation as required by the City of Toronto.

The HGL analysis confirmed that surcharging remains 1.8m or greater below the surface elevation between the subject site and the trunk sewer. As such, no external sanitary upgrades are required to accommodate the proposed site. Refer to **Drawing S1, Servicing Plan** for the proposed sanitary connection locations including the sampling access point.



6.0 STORMWATER SERVICING

6.1 EXISTING CONDITIONS

The redevelopment area consists of a multi-storey above-ground parking garage at 320 McCowan Road. Based on the On-Site Dye Testing conducted by Infrastructure Intelligence Services Inc. (I2S Inc.) in April 2023, the roof drains atop of the existing parking garage and the existing catchbasin northwest of the existing parking garage is currently discharging to the control manhole located at the northwest entrance of the site, which is connected to the 250mm \varnothing municipal storm sewer adjacent to the site running north towards Eglinton Avenue East. Two catchbasins located east of the existing parking garage currently discharge flows to the 250mm \varnothing municipal storm sewer along Landmark Boulevard. It should be noted that the area contributing to the 250mm \varnothing storm sewer along Landmark Boulevard is not included as part of the allowable release rate calculation. Refer to **Figure 2 – Existing Storm Drainage Plan** and **Appendix D** for the On-Site Dye Testing Results.

As shown in **Figure 2**, the 0.88 ha area of the site comprise of existing driveway/easement, the two existing buildings and their associated landscaped areas. These areas will not be included in the stormwater calculations as this area is not being redeveloped and will remain functioning as it does in current conditions. Overland drainage from this area will not be impacted by the redevelopment area of the site.

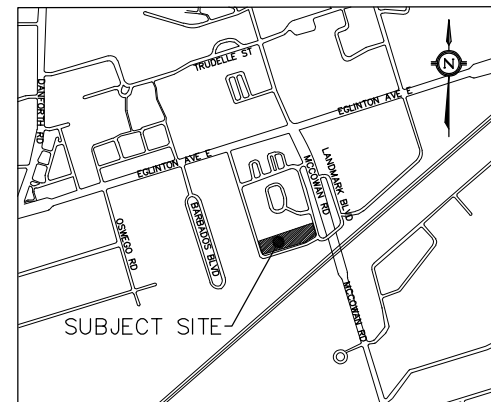
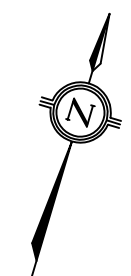
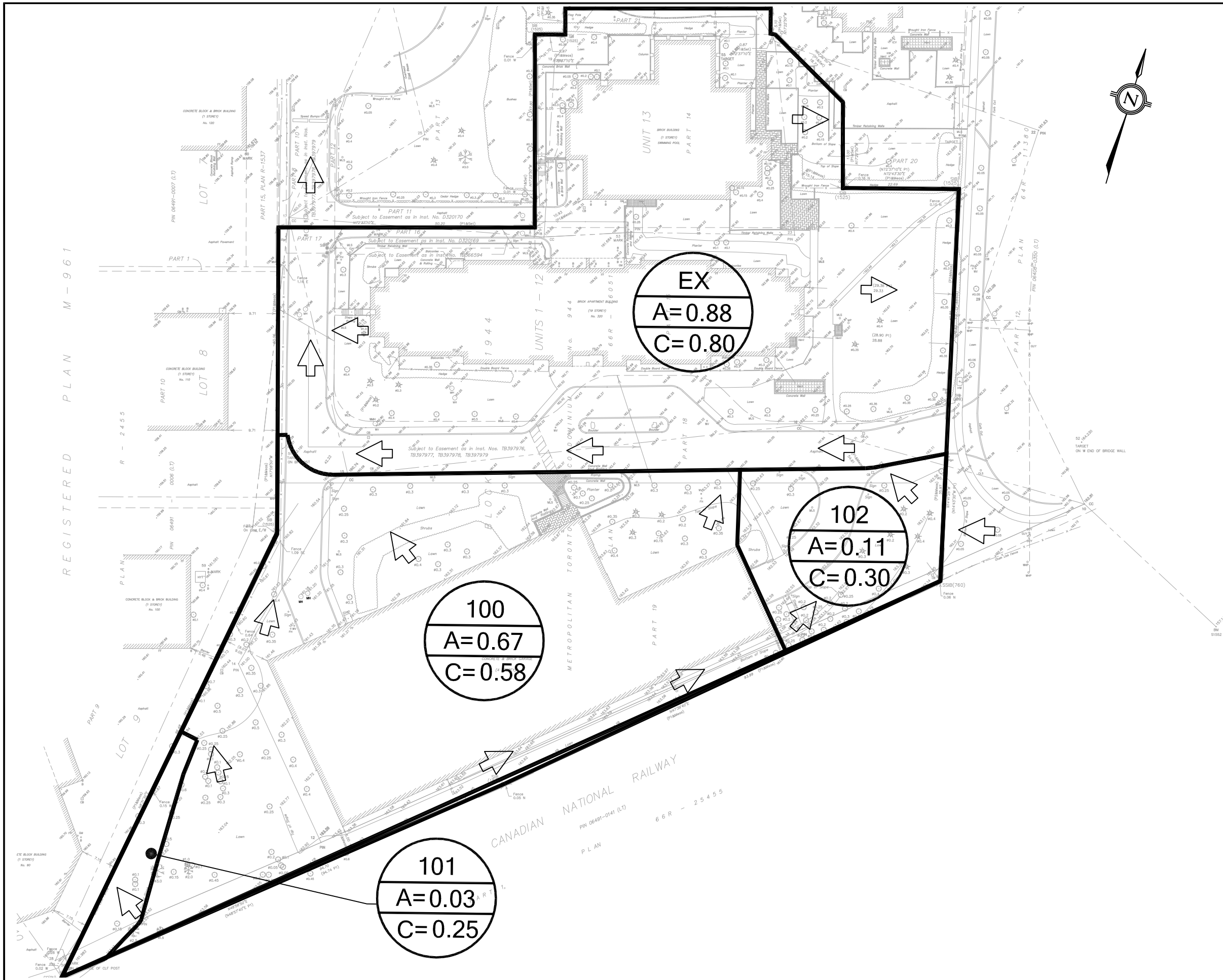
It should be noted that no storm runoff from the existing Metrolinx rail corridor is directed to the site in existing conditions and no storm runoff will be directed to the Metrolinx rail corridor upon redevelopment.

Under existing conditions, the uncontrolled flows from the 0.67 ha area to the 250mm \varnothing storm sewer adjacent the site are summarized in the below table:

Table 1 – Existing Site Uncontrolled Runoff to 250mm \varnothing Storm Sewer Northwest of the Site

Storm Event	Existing Site Uncontrolled Runoff (L/s)
2-Year	96
5-Year	143
10-Year	176
25-Year	206
50-Year	243
100-Year	271

Refer to **Appendix D** for runoff calculations.



LEGEND

- DRAINAGE AREA BOUNDARY
- AREA ID
AREA (HA)
RUNOFF COEFFICIENT
- EXISTING OVERLAND FLOW DIRECTION

counterpoint ENGINEERING

COUNTERPOINT ENGINEERING INC.
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320 MCCOWAN ROAD
RESIDENTIAL DEVELOPMENT
TORONTO, ONTARIO

EXISTING STORM DRAINAGE PLAN	
DESIGNED BY: PA	DATE: MAY 2023
CHECKED BY: RS	PROJECT No. 19066
DRAWING BY: PA	FIGURE No.
CHECKED BY: RS	2
SCALE: NTS	



6.2 ALLOWABLE RELEASE RATE

The sites imperviousness under existing conditions is higher than 50%. Based on the Wet Weather Flow Management Guidelines, the maximum value of C (Runoff Coefficient) used in calculating the pre-development peak runoff rate is limited to 0.50 for the 2-year storm event. As the existing property is greater than 50% impervious, this rule applies.

The allowable storm discharge to the existing 250mmØ storm sewer northwest of the site was calculated using the 0.67 ha area that currently discharges to the same storm sewer at a runoff coefficient of 0.50 as follows:

$$Q_A = C \times A \times i \times N \text{ (L/s)}$$

Table 2 - Allowable Release Rate

Variables	Site
A - Site Area (ha)	0.67
T_c (min)	10
C - Runoff Coefficient	0.50
i - Intensity	88.19
N – Constant	2.778
Q - Release Rate (L/s)	82.1

The allowable release rate to the 250mmØ storm sewer northwest of the site is **82.1 L/s**. Refer to **Appendix D** for allowable release rate calculations.

6.3 PROPOSED STORMWATER SERVICING

This Report has been prepared in accordance with the criteria set by the City of Toronto Weather Flow Management Guidelines (WWFMG). Since majority of the existing storm runoff from the site is capture and conveyed by the 250mmØ municipal storm sewer northwest of the existing site and service connection to sewer within an easement is not prohibited by the municipal easement agreement, the proposed development will be serviced by the aforementioned 250mmØ storm sewer northwest of the site to maintain existing storm drainage flow pattern. Refer to **Drawing S1, Servicing Plan** for the proposed stormwater sewer layout including the preliminary tank location.

Under proposed conditions the new building comprises approximately 74% of the 0.81 ha redevelopment area. It is anticipated that an area of 0.035 ha along the frontage of the development will flow uncontrolled to the easement. Refer to **Figure 3 - Proposed Storm Drainage Plan**.



FIGURE 3



6.4 QUANTITY CONTROL

The allowable site release rate for the proposed development was determined by calculating the 2-year peak flow with a maximum value of 50% impervious as per the City of Toronto Wet Weather Flow Management Guidelines. Using only the redevelopment area, the allowable release rate to the 250 mmØ municipal storm sewer northwest of the site is **82.1 L/s**.

Uncontrolled Area UNC will produce a peak runoff of **21.9 L/s** during the 100-year storm event.

Quantity control will be provided on-site by an underground storage tank in combination with an inlet control device (orifice plate to be sized during Site Plan Approval) to ensure that the 100-year post redevelopment peak flows from the site are attenuated to the 2-year allowable release rate of **82.1 L/s**. A storage volume of approximately **235 m³** will be required to control the 100-year post development flows to the allowable release rate. Refer to **Appendix D** for detailed calculations.

Table 3 – Peak Flow and Storage Summary - 100-Year Storm Event

Area ID	Area (ha)	Runoff Coefficient	t _c (min)	Storage Required (m ³)	Release Rate (L/s)	Description	Orifice Size (mm)
100	0.775	0.83	10	235	60.1	Orifice Plate	TBD at SPA
UNC	0.035	0.90	10		21.9	Uncontrolled	
	0.81				82.0		

1. On-site storage will be provided via an underground storage tank located within the building.
2. Refer to **Appendix D** for modified rational calculations.

As shown in Table 3 above, the proposed release rate of **82.0 L/s** will be less than the overall allowable release rate to the aforementioned storm sewer northwest of the site. In situations where the orifice plate is not sufficient, the at grade access lid to the underground storage tank will allow water to discharge overland to the easement adjacent to the site. The access lid is to be as per OPSD 401.010 – Type B – Open Cover. The required water reuse volume will be available below the outlet invert and is discussed in further detail later in this report in **Section 6.6**.

The design of all internal piping within the building must provide adequate capacity for full capture and conveyance of all flows generated by storms up to and including the 100-year rainfall event. All design and associated calculations for the internal storm system, including the design of the internal inlet structures, piping and mechanical appurtenances is to be completed by the Mechanical Engineer.



6.5 WATER BALANCE

The Wet Weather Flow Guidelines indicate that the minimum on-site runoff retention requires the proponent to retain all runoff from a small design rainfall event – typically 5 mm (In Toronto, storms with 24-hour volumes of 5 mm or less contribute about 50% of the total average annual rainfall volume). This runoff must be retained through infiltration, evapotranspiration or rainwater reuse.

To achieve the water balance objectives, the site was categorized by surface types: impervious asphalt/paved/roof and landscaped areas. The initial abstraction values for the impervious surfaces and pervious surfaces were 1 mm and 5 mm, respectively. The initial abstraction was determined based on percent of surface area and initial abstraction values of each surface type. Without any specific on-site retention measures, the proposed development would achieve the following levels of water balance as seen in **Table 4**.

Table 4 – Achieved Water Balance

Site Description	Fraction of Site Area		Initial Abstraction (mm)	Overall Initial Abstraction (mm)
Non-Green Roof Building Area	64%	0.500 ha	1.0	0.64
Asphalt	25%	0.202 ha	1.0	0.25
Pervious / Green Roof Area	11%	0.088 ha	5.0	0.54
Total	100%	0.810 ha		1.44

Based on **Table 4**, the site will have a shortfall of 3.57 mm (5.00 mm – 1.43 mm) of initial abstraction. This is equivalent to approximately **28.9 m³** of storage. To achieve water balance requirements, a water re-use system will be employed to provide the additional storage indicated above for irrigation demand. The 72-hour irrigation demand has been calculated to be 28.99m³. Refer to **Appendix D** for the irrigation demand calculations.

The re-use storage tank will form part of the underground stormwater storage tank provided for quantity control. The underground storage tank will outlet at an elevation such that **28.9 m³** will be available for re-use.



6.6 QUALITY CONTROL

The proposed building, landscape and non-vehicular hardscape areas covers approximately 99% of the redevelopment area of the site. The remainder of the site consists of a very small impervious vehicular driveway area. Runoff from rooftop surfaces, landscape areas and non-vehicular hardscapes is generally considered to be clean and does not require water quality treatment. However, water quality control measures may be required for the small driveway area of the site. A filtration system would then be provided within the proposed driveway to achieve the required minimum 80% TSS removal. Details of any water quality control measures that may be required will be discussed further during Site Plan Approval.

7.0 CONCLUSIONS

This Functional Servicing Report presents a site servicing strategy for the proposed development that addresses the requirements of the applicable design guidelines and provides the basis for detailed servicing design.

We trust this report sufficiently addresses the site servicing requirements and allows for approval of the proposed re-zoning and site plan approval of the subject site for the proposed use described herein. Should there be any questions or comments, please feel free to contact the undersigned.

Sincerely,

Counterpoint Engineering Inc.



Rasheed Serrao, P.Eng.
Partner, Practice Area Lead – Urban
416-629-7805
rserrao@counterpointeng.com



This Report was prepared by Counterpoint Engineering Inc. for the exclusive use of the 'Client' and in accordance with the Terms and Conditions set out in the Agreement between Counterpoint Engineering Inc. and said Client. The material contained in this Report and all information relating to this activity reflect Counterpoint Engineering's assessment based on the information made available at the time of preparation of this report and do not take into account any subsequent changes that may have occurred thereafter. It should be noted that the information included in this report and data provided to Counterpoint Engineering has not been independently verified. Counterpoint Engineering Inc. represents that it has performed services hereunder with a degree of care, skill, and diligence normally provided by similarly-situated professionals in the performance of such services in respect of projects of similar nature at the time and place those services were rendered. Counterpoint Engineering Inc. disclaims all warranties, or any other representations, or conditions, either expressed or implied. With the exception of any designated 'Approving Authorities' including the City of Toronto to whom this report was submitted to for approval by Counterpoint Engineering Inc., any reliance on this document by a third party is strictly prohibited without written permission from Counterpoint Engineering Inc.. Counterpoint Engineering Inc. accepts no responsibility for damages, if any, suffered by a third party as a result of decisions made or actions based on this Report.



*Blauson Assets Management Ltd. c/o Ling Kee Group
320 McCowan Road*

Appendix A

PLAN OF SURVEY WITH TOPOGRAPHY OF
METROPOLITAN TORONTO CONDOMINIUM
PLAN No. 944
 CITY OF TORONTO

SCALE 1 : 300

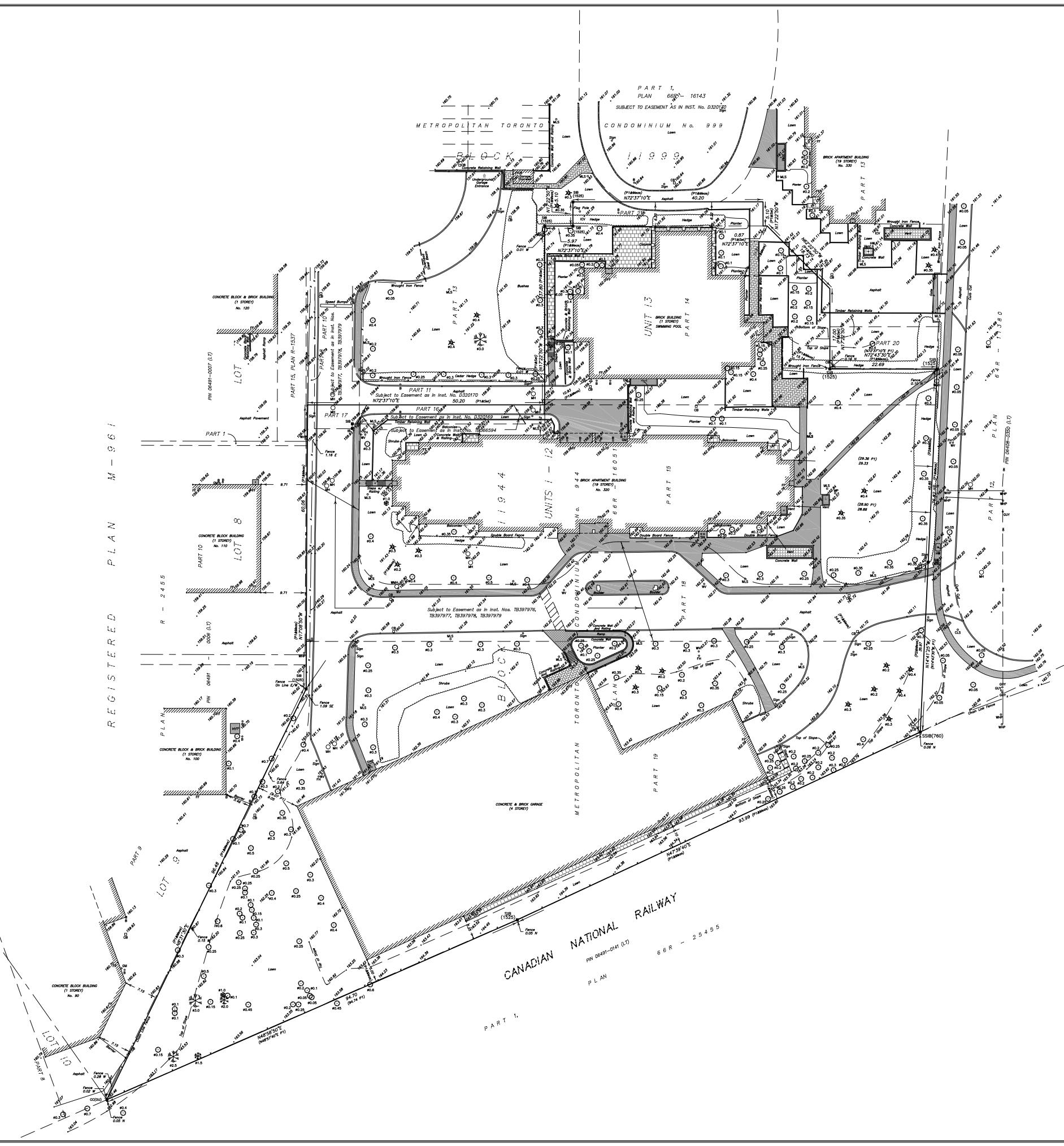


SPEIGHT, VAN NOSTRAND & GIBSON LIMITED
 ONTARIO LAND SURVEYORS
 2019

(C) THE REPRESENTATION, ALTERATION OR USE OF THIS PLAN
 IN WHOLE OR IN PART WITHOUT THE EXPRESS PERMISSION OF
 SPEIGHT, VAN NOSTRAND & GIBSON LIMITED IS STRICTLY PROHIBITED.



REGISTERED PLAN M-961



ELEVATION NOTE

ELEVATIONS ARE GEODETIC AND ARE DERIVED FROM THE CITY OF TORONTO BENCHMARK No. 1241961552, also known as S1552.

LOCATION :
 MCCOWAN ROAD AT C.N.R. MAINLINE PLATE IN WEST FACE OF EAST RETAINING WALL AT NORTH OF SOUTH END OF OVERPASS.

ELEVATION:
 PUBLISHED ELEVATION = 157.146 metres.

BEARING NOTE

BEARINGS ARE GRID BEARINGS, DERIVED FROM SPECIFIED CONTROL POINTS 00119633048 AND 02019660001, MTM ZONE 10, CENTRAL MERIDIAN 79°30'W COORDINATES TO URBAN ACCURACY PER SECTION 14 (2) OF O. REG. 216/10 LONGITUDE MAD 83 (CSRS) 1997.0.

POINT ID	NORTHING	EASTING
SCP 00119633048	4843730.215	327811.382
SCP 02019660001	4841217.947	321166.284

COORDINATES CANNOT, IN THEMSELVES, BE USED TO RE-ESTABLISH CORNERS OR BOUNDARIES SHOWN ON THIS PLAN.
 DISTANCES ARE GROUND AND CAN BE CONVERTED TO GRID BY MULTIPLYING BY THE COMBINED SCALE FACTOR OF 0.999983.

METRIC

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

LEGEND

■	DENOTES	SURVEY MONUMENT FOUND
□		SURVEY MONUMENT PLANTED
WT		WITNESS MONUMENT
SB		STANDARD IRON BAW
SSB		SHORT STANDARD IRON BAR
IB		IRON BAR
CC		CUT CROSS
N,S,E,W		NORTH, SOUTH, EAST, WEST
OU		ORIGIN UNKNOWN
S		S. BARNED LIMITED, O.L.S.
760		MCCONNELL & LAUGHAN, O.L.S.
R		R. AND SURVEYING INC., O.L.S.
1255		METROPOLITAN TORONTO CONDOMINIUM PLAN No. 944
P1		PLAN 66R-16051
P2		PLAN 66R-16051
B		BOLLARD
CB		CATCH BASIN
CLS		CONCRETE LIGHT STANDARD
FF		FINISHED FLOOR
FF		FINISHED FLOOR
PH		FIRE HYDRANT
QM		QAS METER
GUY		GUY WIRE/POLE
HW		HAND WELL
HT		HYDRO TRANSFORMER
ICV		IRRIGATION CONTROL VALVE
MH		MANHOLE
MCS		METAL LIGHT STANDARD
MP		METAL POLE
CH		CROSSHEAD WIRE
SA		SHAMBLE CONNECTION
WHP		WOODEN HYDRO POLE
WMH		WATER MANHOLE
WV		WATER VALVE
DT		DECIDUOUS TREE
CT		CONIFEROUS TREE
CON		CONCRETE
CON		CONCRETE TILES
BRK		BRICK
GR		METAL GRATE

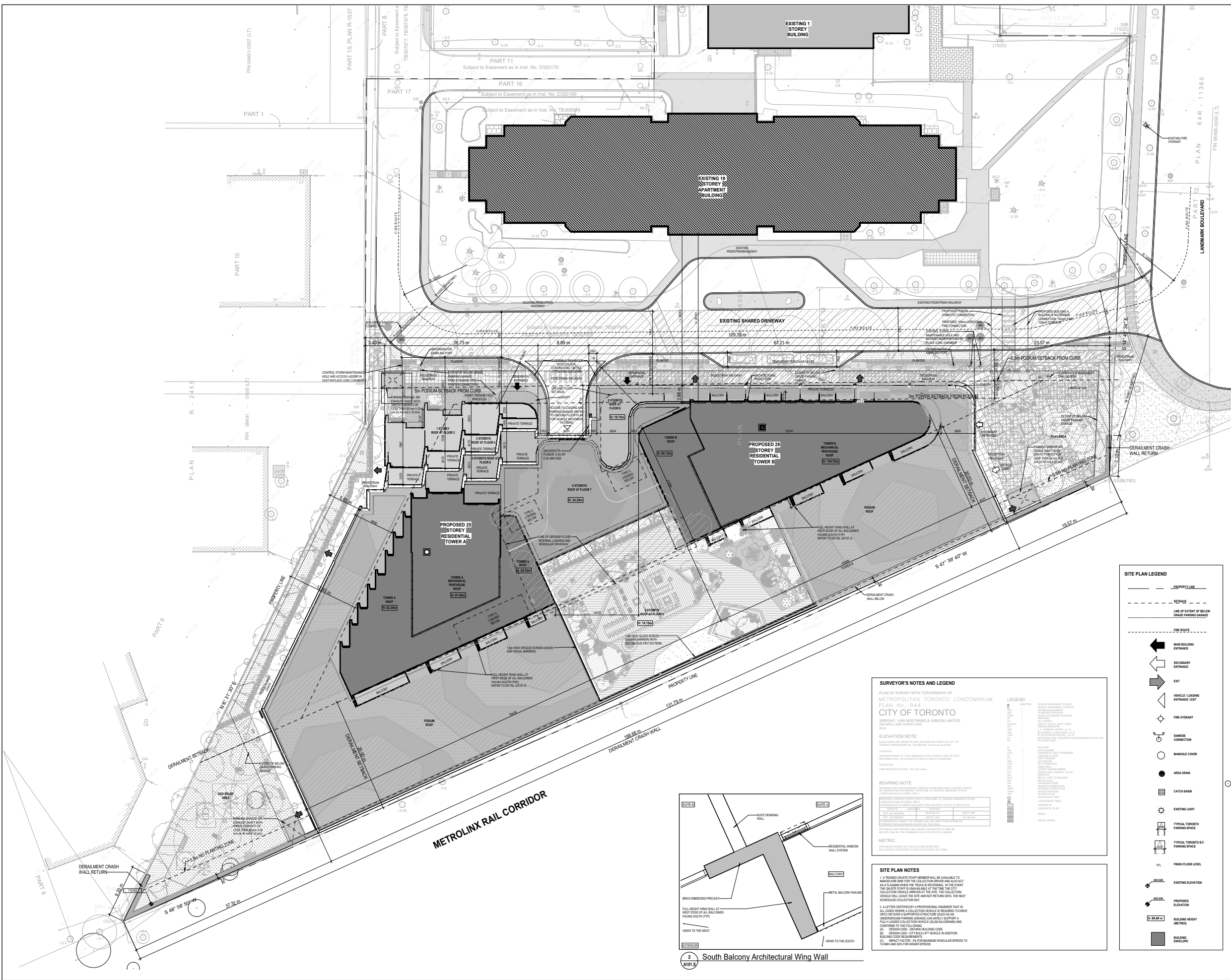
SURVEYOR'S CERTIFICATE

I CERTIFY THAT:
 1. THIS SURVEY AND PLAN ARE CORRECT AND IN ACCORDANCE WITH THE SURVEYS ACT,
 THE SURVEYORS ACT AND THE REGULATIONS MADE UNDER THEM.
 2. THE SURVEY WAS COMPLETED ON OCTOBER 8, 2019.

DATE: _____
 D. A. WILSON
 ONTARIO LAND SURVEYOR

SPEIGHT, VAN NOSTRAND & GIBSON LIMITED
 ONTARIO LAND SURVEYORS
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 TORONTO, ONTARIO M3N 2Z4
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DRAWN: F. P. B./M. M.	FILE NAME: A1900291.DWG
CHECKED: D. A. W.	PLOT SCALE: MET. 1=0.40
JOB No.: 190-0291	PLOTTED: Oct. 8, 2019
REF. No.: T-344 Metro Condo Plan	UPDATED:



SURVEYOR'S NOTES AND LEGEND
 PLAN OF SURVEY WITH TOPOGRAPHY OF
 METROPOLITAN TORONTO CONDOMINIUM
 PLAN No. 944
CITY OF TORONTO
 SPEIGHT, VAN NOSTRAND & GIBSON LIMITED
 ONTARIO LAND SURVEYORS
 2015

ELEVATION NOTE
 ELEVATIONS ARE GIVEN TO AND FROM THE CITY OF TORONTO MEAN SEA LEVEL DATUM, 1985.

BEARING NOTE
 BEARINGS ARE GIVEN TO AND FROM THE CITY OF TORONTO MEAN SEA LEVEL DATUM, 1985.

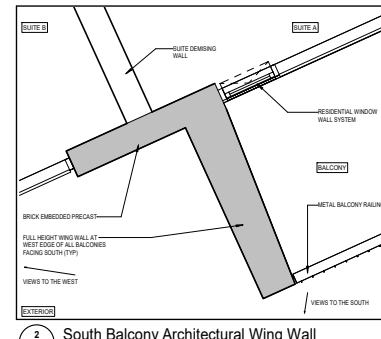
METRIC
 DISTANCES SHOWN ON THIS PLAN ARE IN METERS AND DECIMALS THEREOF TO THE NEAREST MILLIMETER.

LEGEND

PROPERTY LINE	PROPERTY LINE
SETBACK	SETBACK
LINE OF EXTENT OF BELOW GRADE PARKING GARAGE	LINE OF EXTENT OF BELOW GRADE PARKING GARAGE
FIRE ROUTE	FIRE ROUTE
MAIN BUILDING ENTRANCE	MAIN BUILDING ENTRANCE
SECONDARY ENTRANCE	SECONDARY ENTRANCE
EXIT	EXIT
VEHICLE LOADING ENTRANCE / EXIT	VEHICLE LOADING ENTRANCE / EXIT
FIRE HYDRANT	FIRE HYDRANT
BARRELS CONNECTION	BARRELS CONNECTION
MANHOLE COVER	MANHOLE COVER
AREA DRAIN	AREA DRAIN
CATCH BASIN	CATCH BASIN
EXISTING LIGHT	EXISTING LIGHT
TYPICAL TORONTO PARKING SPACE	TYPICAL TORONTO PARKING SPACE
TYPICAL TORONTO B.F. PARKING SPACE	TYPICAL TORONTO B.F. PARKING SPACE
FINISH FLOOR LEVEL	FINISH FLOOR LEVEL
EXISTING ELEVATION	EXISTING ELEVATION
PROPOSED ELEVATION	PROPOSED ELEVATION
BUILDING HEIGHT (METRES)	BUILDING HEIGHT (METRES)
BUILDING ENVELOPE	BUILDING ENVELOPE

SITE PLAN NOTES

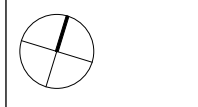
- A TRAINED ON-SITE STAFF MEMBER WILL BE AVAILABLE TO MANEUVER VEHICLES FOR THE COLLECTION DRIVER AND SUBJECT AS A FLAGMAN WHEN THE TRUCK IS REVERSING. IN THE EVENT THE TRUCK STAFF IS UNAVAILABLE AT THE TIME THE CITY COLLECTION VEHICLE ARRIVES AT THE SITE, THE COLLECTION VEHICLE WILL LEAVE THE SITE AND NOT RETURN AT THE NEXT SCHEDULED COLLECTION DAY.
- A LETTER CERTIFIED BY A PROFESSIONAL ENGINEER THAT IN ALL CASES WHERE A COLLECTION VEHICLE IS REQUIRED TO COME ONTO THE SITE TO SUPPORT THE COLLECTION OF DEBRIS FROM AN UNDERGROUND PARKING GARAGE CAN SAFELY SUPPORT A FULLY LOADED COLLECTION VEHICLE (15,000 KG GROSS) AND CONFORM TO THE FOLLOWING:
 - (A) DESIGN LOAD - CITY BUILDING CODE
 - (B) DESIGN LOAD - CITY BUILDING CODE IN ADDITION BUILDING CODE REQUIREMENTS
 - (C) IMPACT FACTOR - 2/3 FOR MAXIMUM VEHICULAR SPEEDS TO 15 MPH AND 3/4 FOR HIGHER SPEEDS.



REVISION RECORD

Date	No.	Description
2023-05-03		Rezoning Resubmission
2021-07-09		Site Plan Approval Submission
2021-04-30		Rezoning Resubmission
2020-02-14		Rezoning Application

ISSUE RECORD



BDP. Quadrangle

Quadrangle Architects Limited
 881 King Street West, Suite 707 Toronto, ON M5V 3P5
 416 598 1240 www.bdpquadrangle.com

320 McCowan Road
 Toronto, Ontario
 for Blauson Assets Management Ltd. c/o Ling Kee Group

18005 As indicated JT RL
 PROJECT SCALE DRAWN REVIEWED

Site Plan

A101.S

Note: This drawing is the property of the Architect and may not be reproduced or used without the expressed consent of the Architect. The Contractor is responsible for checking and verifying all levels and dimensions and shall report all discrepancies to the Architect and obtain clarification prior to commencing work.

2 South Balcony Architectural Wing Wall
 A101.S



*Blauson Assets Management Ltd. c/o Ling Kee Group
320 McCowan Road*

Appendix B

Counterpoint Engineering Inc.

Water Demand Design Calculations

Project: 320 McCowan Road
 Project No: 19066
 Location: Toronto, Ontario
 Site Area: 0.810 ha

Population

Bachelor / 1 Bedroom	1.4	ppu
2 Bedroom	2.1	ppu
3 Bedroom	3.1	ppu
Commercial / Retail	1.1	persons/100m ²
Offices	3.3	persons/100m ²

	Residential Units					Retail
	Bach	1 Bedroom	2 Bedroom	3 Bedroom	Total Units	Area (m ²)
Level 1	-	7	0	0	7	-
Level 2	-	1	8	3	12	-
Level 3	-	2	8	2	12	-
Level 4	-	2	9	1	12	-
Level 5	-	2	9	1	12	-
Level 6	-	5	2	1	8	-
Tower A Level 7-25	19	57	95	19	190	-
Tower B Level 7-29	-	92	115	23	230	-
TOTAL UNITS / AREA (m²)	19	168	246	50	483	0

	Population 1 Bedroom	Population 2 Bedroom	Population 3 Bedroom	TOTAL POPULATION
Residential Building	262	517	155	934
Retail/Commercial				0
Total Equivalent Population				934

City of Toronto Watermain Guidelines

Per Capita Demand

Single Family	310	(L/capita/day)
Multi-Unit	190	(L/capita/day)

Peaking Factors

Land Use	Minimum Hour	Maximum Hour	Maximum Day
Residential	0.80	2.25	1.50
Commercial	0.84	1.20	1.10
Industrial	0.84	1.90	1.10
Institutional	0.84	1.90	1.10
Apartment	0.84	2.50	1.30

*Values used for Residential Land Use

Water Demand based on Equivalent Population

Land Use	Population	Minimum Hour (L/min)	Maximum Hour (L/min)	Maximum Day (L/min)	Fire Flow Required (L/min)	Fire Flow Duration (hr)	Max Day + Fire Flow (L/min)
Residential (Multi-Unit)	934	103.5	308.1	160.2	-	-	
Totals	934	103.5	308.1	160.2	11,000	2.00	11,160

Counterpoint Engineering Inc.

REQUIRED FIRE FLOW WORKSHEET - PROPOSED DEVELOPMENT

Fire Underwriters Survey

Project : 320 McCowan Road

Project No: 19066

Guide for Determination of Required Flow Copyright I.S.O

$$F = 220C\sqrt{A^*}$$

where

- F = the required fire flow in litres per minute.
- C = coefficient related to the type of construction.
 - = 1.5 for wood frame construction (structure essentially all combustible).
 - = 1.0 for ordinary construction (brick or other masonry walls, combustible floor and interior).
 - = 0.8 for non-combustible construction (unprotected metal structural components, masonry or metal walls).
 - = 0.6 for fire-resistive construction (fully protected frame, floors, roof).
- A = The total floor area in square metres (including all storeys, but excluding basements at least 50 percent below grade) in the building being considered.

Type of Construction	Class Factor	
WF	Wood Frame	1.5
OC	Ordinary Construction	1.0
NC	Non-Combustible	0.8
FC	Fire-Resistive	0.6

Area Notes for Fire Resistive Buildings (from FUS manual, 1999):

If Vertical Openings are inadequately protected (less than 1-hour fire rating): Area is the total of the two largest adjoining floors (above ground level) plus 50% of the area of each of the next 8 adjoining floors above that.

Contents	% Reduction	
NC	Non-Combustible	25
LC	Limited Combustible	15
C	Combustible	0
FB	Free Burning	15
RB	Rapid Burning	25

If Vertical Openings are adequately protected (at least 1-hour fire rating): Area is the total of the largest floor (above ground level) plus 25% of the area of each of the next 2 immediately adjoining floors above that.

1) Fire Flow

Type of Construction:

FC

C=

0.60

A*=

19439 m²

F=

18,000 L/min (rounded to nearest 1,000 L/min)

Note: Vertical Openings are not adequately protected and building is fire-resistive as required by OBC for multi-unit buildings

Area is the GBA of the second floor and third floor plus 50% of Floors 4-11. Only GBA of one tower is included in calculations for Floor 7-11 (750m²/ floor)

2) Occupancy Reduction/Surcharge

Contents Factor:

LC

Reduction/Surcharge of

-15%

= -2,700 L/min

F=

18000L/min + -2700 L/min = 15,300 L/min

3) System Type Reduction

NFPA 13 Sprinkler:

YES 30%

Standard Water Supply:

YES 10%

Fully Supervised:

YES 10%

Total

50%

Reduction of

50% L/min = 7,650 L/min

F=

15300L/min - 7,650 L/min = 7,650 L/min

4) Separation Charge

Building Face

Dist(m) Charge

North

26.2 10%

East

49.1 0%

South

49.1 0%

West

20.5 10%

Total

20% of 15300 L/min = 3,060 L/min

(max exposure charge can be 75%)

Separation	Charge	Separation	Charge
0 to 3m	25%	20.1 to 30 m	10%
3.1 to 10m	20%	30.1 to 45m	5%
10.1 to 20m	15%		

F= 7650L/min + 3060L/min = 10,710 L/min (2,000L/min<F<45,000L/min)

F=	11,000	L/min	(round to the nearest 1,000L/min)
F=	183	L/s	
F=	2,906	gpm	

counterpoint engineering

NFPA Theoretical Flow Calculations

Project Name: 320 McCowan Road

Project Number: 19066

Based on National Fire Protection Association Guidelines, the available flow at the minimum residual pressure of 20psi can be calculated based on the observed flow at the observed pressure readings, as follows:

$$Q_F = 29.83 \times c \times d^2 \times p^{0.5}, \text{ where}$$

Q_F = observed flow (US GPM)

c = hydrant nozzle coefficient (0.90 - 0.95)

d = nozzle diameter (in)

p = observed pitot pressure

$$Q_R = Q_F \times h_F^{0.54} / h_R^{0.54}, \text{ where}$$

Q_R = available flow

Q_F = observed flow (US GPM)

h_F = drop from measured static to desired baseline pressure

h_R = drop from measured static to measured residual pressure

Based on flow test results obtained by *Lozzi Aqua Check, November 5, 2020, 2pm.*

$c =$ 0.95
 $d =$ 2.5 in
number of ports = 2
 $p =$ 25

$Q_F =$ 1771 US GPM

Measured Static Pressure = 62 psi
Measured Residual Pressure = 52 psi
Desired Residual Pressure = 20 psi, minimum per City of Toronto design criteria

$Q_R =$ 3844 US GPM per fire connection
14,552 L/min

Lozzi Aqua Check

4820 18th Sideroad
Schomberg, Ontario
LOG-1T0

Massimo Lozzi Cell: 416 990-2131
E-mail: lozziaquacheck@gmail.com

Hydrant Flow Test Form

Job Location: 320 McCowan Rd ,Toronto

Date: November 5,2020

Test Date

Time of Test: 2:00 pm

Location of Flow Hydrant: in front of 320 Landmark Blvd.

Residual hydrant: the next hydrant north in front of 330 Landmark Blvd.

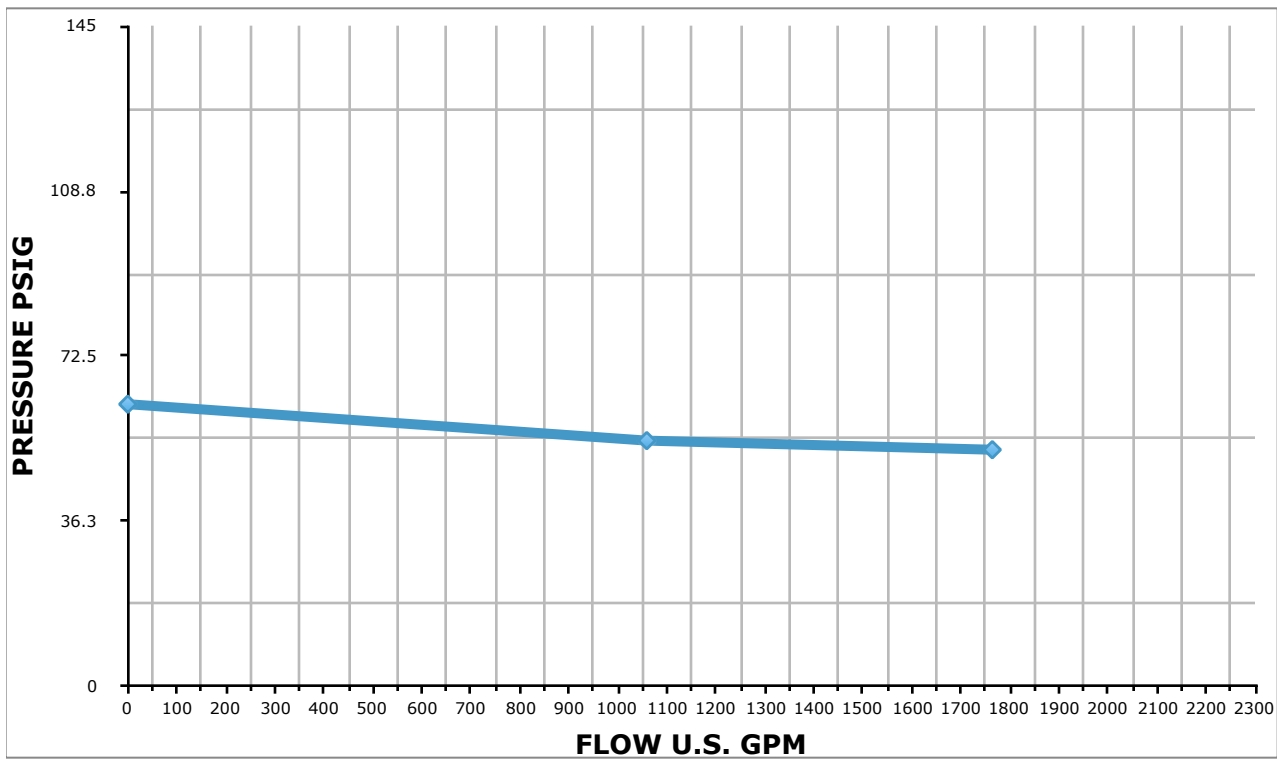
Main Size: 300 mm

Static Pressure: 62 psi

Theoretical GPM at 20 psi - 3633 gpm

	Number of Outlets & Orifice Size	Pitot Pressure (psi)	Flow (U.S. G.P.M.)	Residual Pressure (psi)
1.	Static	0	0	62
2.	1 x 2 ½	40	1059	54
3.	2 x 2 ½	25	1674	52

Note :Flow test conducted in accordance with NFPA Std 291





*Blauson Assets Management Ltd. c/o Ling Kee Group
320 McCowan Road*

Appendix C

Counterpoint Engineering Inc.

Project: 320 McCowan Road
 Project No: 19066
 Location: Toronto
 Redevelopment Site Area: 0.810 ha

Proposed Sanitary Flow Calculations

As per Design Criteria for Sewers and Watermains - First Edition November 2009 City of Toronto

Design flow = average daily dry weather flow x peaking factor + infiltration

Persons Per Unit and per Land Use

Townhouse	2.7	ppu
Bachelor / 1 Bedroom	1.4	ppu
2 Bedroom	2.1	ppu
3 Bedroom	3.1	ppu
Commercial / Retail	1.1	persons/100m ²
Offices	3.3	persons/100m ²

	Residential Units				Retail
	Bach & 1 Bedroom	2 Bedroom	3 Bedroom	Total Units	Area (m ²)
TOTAL UNITS / AREA (m ²)	187	246	50	483	0

	Population 1BR / 1B + D	Population 2BR / 2BR + D	Population 3BR / 3BR + D	TOTAL POPULATION
Residential Building	262	517	155	934
Retail/Commercial				0
Total Equivalent Population				934

Peak flow Design Parameters

Residential Average flow	240	litres/person/day
Commercial Average flow	180,000	litres/ha/day
Infiltration	0.26	litres/second/ha

Harmon Peaking Factor

$$PF = 1 + (14/(4+(P/1000)^{1/2}))$$

Residential Population	Harmon Peak Factor
934	3.82

Residential Flow	9.91	L/s	*based on population
Retail/Commercial Flow	0.00	L/s	*based on floor area
Infiltration	0.21	L/s	
Groundwater Flows	1.58	L/s	*based on a 25gpm pump.

Total Peak Flow	11.70	L/s
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Existing Dry Conditions Sanitary Flow

320 McCowan Road

Project No.: 19066

Composite population density per unit Single - 3.5 Semi & Townhouse - 2.7	Population density-Commercial 1.1 p / 100 m ² Population density-Office 3.3 p / 100 m ² Population density-Institutional 86 p / hectare	Page 1 of 1
P = population = sum p A site	A = area ha	Date: 19-May-23 Designed by: PA Checked by: RS
$M = 1 + (14)/(4+(P/10^3))^{0.5}$ I = 0.26 x A gross L/s q = 240 L/person/day (For the analysis of existing sewers)	Pflow = M q P / 86400 = population flow L/s Q TOTAL = P flow + I L/s	

STREET	MANHOLE		AREA ha	EQUIV. POP P	CUMULATIVE			A GROSS ha	I L/s	Q TOTAL L/s	LENGTH (m)	SLOPE S %	D (mm)	TYPE OF PIPE	ROUGH. COEFF.	Q FULL (L/s)	Velocity Full (m/s)	Capacity (%)
	FROM	TO			POP. P	M	PEAK FLOW L/s											
Easement Sewer (site)	MH26	MH25	2.38	189	189	4.20	2.2	2.38	0.62	2.8	42.50	0.50	250	CONC	0.013	43.9	0.87	6%
Easement Sewer	MH25	MH24	0.60	174	363	4.00	4.0	2.98	0.77	4.8	89.20	0.62	250	CONC	0.013	48.8	0.96	10%
Easement Sewer	MH24	MH23	0.00	0	363	4.00	4.0	2.98	0.77	4.8	9.00	0.39	250	CONC	0.013	38.7	0.76	12%
Input: Eglinton Avenue East		MH23	8.14	1005	1368	3.70	14.1	11.12										
Eglinton Ave E	MH23	MH22	0.48	9	1377	3.70	14.2	11.60	3.02	17.2	39.93	0.88	250	CONC	0.013	58.2	1.15	30%
Eglinton Ave E	MH22	MH21	0.00	0	1377	3.70	14.2	11.60	3.02	17.2	7.01	1.56	250	CONC	0.013	77.5	1.53	22%
Eglinton Ave E	MH21	MH20	0.16	0	1377	3.70	14.2	11.76	3.06	17.2	62.79	1.14	250	CONC	0.013	66.2	1.31	26%
Input: Barbados Blvd E		MH20	2.55	74	1451			14.31										
Eglinton Ave E	MH20	MH19	0.00	0	1451	3.70	14.9	14.31	3.72	18.6	12.19	1.48	250	CONC	0.013	75.5	1.49	25%
Input: Barbados Blvd W		MH19	156.80	4451	5902			171.11										
North of Barbados Blvd	MH19	MH18	0.04	0	5902	3.20	52.5	171.15	44.50	97.0	29.87	0.22	600	CONC	0.013	301.0	1.03	32%
Input: Eglinton E North			5.50	114	6016			176.65										
North of Barbados Blvd	MH18	MH17	5.02	519	6535	3.10	56.3	181.67	47.23	103.5	98.45	0.38	600	CONC	0.013	395.6	1.35	26%
North of Barbados Blvd	MH17	MH16	0.25	0	6535	3.10	56.3	181.92	47.30	103.6	87.17	0.42	600	CONC	0.013	415.9	1.42	25%
North of Trudelle St	MH16	MH15	0.00	0	6535	3.10	56.3	181.92	47.30	103.6	4.00	0.31	600	CONC	0.013	357.3	1.22	29%
Input: Trudelle St W		MH15	5.28	661	7196			187.20										
North of Trudelle St	MH15	MH14	0.00	0	7196	3.10	62.0	187.20	48.67	110.6	23.00	0.31	600	CONC	0.013	357.3	1.22	31%
Input: Trudelle St E			12.11	2378	9573			199.31										
West Highland Creek	MH14	MH13	0.00	0	9573	3.00	79.8	199.31	51.82	131.6	60.17	0.31	600	CONC	0.013	357.3	1.22	37%
West Highland Creek	MH13	MH12	0.00	0	9573	3.00	79.8	199.32	51.82	131.6	42.98	0.31	600	CONC	0.013	357.3	1.22	37%
West Highland Creek	MH12	MH11	0.00	0	9573	3.00	79.8	199.32	51.82	131.6	111.25	0.35	600	CONC	0.013	379.6	1.30	35%
West Highland Creek	MH11	MH10	0.00	0	9573	3.00	79.8	199.32	51.82	131.6	63.09	0.40	600	CONC	0.013	405.8	1.39	32%
West Highland Creek	MH10	MH9	0.00	0	9573	3.00	79.8	199.33	51.82	131.6	72.54	0.37	600	CONC	0.013	390.3	1.34	34%
Input: Danforth Rd North		MH9	5.40	1034	10607			204.73										
West Highland Creek	MH9	MH8	0.00	0	10607	2.90	85.4	204.73	53.23	138.7	26.52	0.40	600	CONC	0.013	405.8	1.39	34%
Input: Danforth Rd south		MH8	1.06	1078	11685			205.79										
West Highland Creek	MH8	MH7	0.00	0	11685	2.90	94.1	205.79	53.51	147.6	22.25	0.74	600	CONC	0.013	552.0	1.89	27%
Input: Strode Dr		MH7	2.05	386	12071			207.84										
West Highland Creek	MH7	MH6	0.00	0	12071	2.90	97.2	207.85	54.04	151.3	28.96	0.50	600	CONC	0.013	453.7	1.55	33%
West Highland Creek	MH6	MH5	0.00	0	12071	2.90	97.2	207.85	54.04	151.3	65.23	0.49	600	CONC	0.013	449.2	1.54	34%
Input: Graylee Ave		MH5	1.64	84	12155			209.49										
West Highland Creek	MH5	MH4	0.00	0	12155	2.90	97.9	209.49	54.47	152.4	70.41	0.51	600	CONC	0.013	458.3	1.57	33%
West Highland Creek	MH4	MH3	0.00	0	12155	2.90	97.9	209.49	54.47	152.4	76.81	0.44	600	CONC	0.013	425.6	1.46	36%
West Highland Creek	MH3	MH2	0.00	0	12155	2.90	97.9	209.50	54.47	152.4	74.98	0.46	600	CONC	0.013	435.2	1.49	35%
West Highland Creek	MH2	MH1	0.00	0	12155	2.90	97.9	209.50	54.47	152.4	83.82	0.60	600	CONC	0.013	497.0	1.70	31%
West Highland Creek	MH1	Trunk	0.00	0	12155	2.90	97.9	209.50	54.47	152.4	17.37	1.04	600	CONC	0.013	654.4	2.24	23%

Existing Wet Conditions Sanitary Flow

320 McCowan Road

Project No.: 19066

Composite population density per unit	Single - 3.5 Semi & Townhouse - 2.7	Population density-Commercial 1.1 Population density-Office 3.3 Population density-Institutional 86	p / 100 m ² p / 100 m ² p / hectare
P = population = sum p A site		A = area ha	Page 1 of 1
M = 1 + (14)/(4+(P/10 ³)) ^{0.5}		Pflow = M q P / 86400 = population flow L/s	Date: 19-May-23
I = 0.26 x A gross L/s		Q TOTAL = P flow + I L/s	Designed by: PA Checked by: RS
q = 240 L/person/day (For the analysis of existing sewers)			

STREET	MANHOLE		AREA ha	EQUIV. POP P	CUMULATIVE			A GROSS ha	I L/s	Q TOTAL L/s	LENGTH (m)	SLOPE S %	D (mm)	TYPE OF PIPE	ROUGH. COEFF.	Q FULL (L/s)	Velocity Full (m/s)	Capacity (%)
	FROM	TO			POP. P	M	PEAK FLOW L/s											
Easement Sewer (site)	MH26	MH25	2.38	189	189	4.20	2.2	2.38	23.80	26.0	42.50	0.50	250	CONC	0.013	43.9	0.87	59%
Easement Sewer	MH25	MH24	0.60	174	363	4.00	4.0	2.98	29.80	33.8	89.20	0.62	250	CONC	0.013	48.8	0.96	69%
Easement Sewer	MH24	MH23	0.00	0	363	4.00	4.0	2.98	29.80	33.8	9.00	0.39	250	CONC	0.013	38.7	0.76	87%
Input: Eglinton Avenue East		MH23	8.14	1005	1368	3.70	14.1	11.12										
Eglinton Ave E	MH23	MH22	0.48	9	1377	3.70	14.2	11.60	58.00	72.2	39.93	0.88	250	CONC	0.013	58.2	1.15	124%
Eglinton Ave E	MH22	MH21	0.00	0	1377	3.70	14.2	11.60	58.00	72.2	7.01	1.56	250	CONC	0.013	77.5	1.53	93%
Eglinton Ave E	MH21	MH20	0.16	0	1377	3.70	14.2	11.76	58.80	73.0	62.79	1.14	250	CONC	0.013	66.2	1.31	110%
Input: Barbados Blvd E		MH20	2.55	74	1451			14.31										
Eglinton Ave E	MH20	MH19	0.00	0	1451	3.70	14.9	14.31	71.56	86.5	12.19	1.48	250	CONC	0.013	75.5	1.49	115%
Input: Barbados Blvd W		MH19	156.80	4451	5902			171.11										
North of Barbados Blvd	MH19	MH18	0.04	0	5902	3.20	52.5	171.15	513.45	565.9	29.87	0.22	600	CONC	0.013	301.0	1.03	188%
Input: Eglinton E North			5.50	114	6016			176.65										
North of Barbados Blvd	MH18	MH17	5.02	519	6535	3.10	56.3	181.67	545.01	601.3	98.45	0.38	600	CONC	0.013	395.6	1.35	152%
North of Barbados Blvd	MH17	MH16	0.25	0	6535	3.10	56.3	181.92	545.76	602.0	87.17	0.42	600	CONC	0.013	415.9	1.42	145%
North of Trudelle St	MH16	MH15	0.00	0	6535	3.10	56.3	181.92	545.76	602.0	4.00	0.31	600	CONC	0.013	357.3	1.22	169%
Input: Trudelle St W		MH15	5.28	661	7196			187.20										
North of Trudelle St	MH15	MH14	0.00	0	7196	3.10	62.0	187.20	561.60	623.6	23.00	0.31	600	CONC	0.013	357.3	1.22	175%
Input: Trudelle St E			12.11	2378	9573			199.31										
West Highland Creek	MH14	MH13	0.00	0	9573	3.00	79.8	199.31	597.94	677.7	60.17	0.31	600	CONC	0.013	357.3	1.22	190%
West Highland Creek	MH13	MH12	0.00	0	9573	3.00	79.8	199.32	597.95	677.7	42.98	0.31	600	CONC	0.013	357.3	1.22	190%
West Highland Creek	MH12	MH11	0.00	0	9573	3.00	79.8	199.32	597.96	677.7	111.25	0.35	600	CONC	0.013	379.6	1.30	179%
West Highland Creek	MH11	MH10	0.00	0	9573	3.00	79.8	199.32	597.97	677.7	63.09	0.40	600	CONC	0.013	405.8	1.39	167%
West Highland Creek	MH10	MH9	0.00	0	9573	3.00	79.8	199.33	597.98	677.8	72.54	0.37	600	CONC	0.013	390.3	1.34	174%
Input: Danforth Rd North		MH9	5.40	1034	10607			204.73										
West Highland Creek	MH9	MH8	0.00	0	10607	2.90	85.4	204.73	614.19	699.6	26.52	0.40	600	CONC	0.013	405.8	1.39	172%
Input: Danforth Rd south		MH8	1.06	1078	11685			205.79										
West Highland Creek	MH8	MH7	0.00	0	11685	2.90	94.1	205.79	617.38	711.5	22.25	0.74	600	CONC	0.013	552.0	1.89	129%
Input: Strode Dr		MH7	2.05	386	12071			207.84										
West Highland Creek	MH7	MH6	0.00	0	12071	2.90	97.2	207.85	623.54	720.8	28.96	0.50	600	CONC	0.013	453.7	1.55	159%
West Highland Creek	MH6	MH5	0.00	0	12071	2.90	97.2	207.85	623.54	720.8	65.23	0.49	600	CONC	0.013	449.2	1.54	160%
Input: Graylee Ave		MH5	1.64	84	12155			209.49										
West Highland Creek	MH5	MH4	0.00	0	12155	2.90	97.9	209.49	628.47	726.4	70.41	0.51	600	CONC	0.013	458.3	1.57	159%
West Highland Creek	MH4	MH3	0.00	0	12155	2.90	97.9	209.49	628.48	726.4	76.81	0.44	600	CONC	0.013	425.6	1.46	171%
West Highland Creek	MH3	MH2	0.00	0	12155	2.90	97.9	209.50	628.49	726.4	74.98	0.46	600	CONC	0.013	435.2	1.49	167%
West Highland Creek	MH2	MH1	0.00	0	12155	2.90	97.9	209.50	628.50	726.4	83.82	0.60	600	CONC	0.013	497.0	1.70	146%
West Highland Creek	MH1	Trunk	0.00	0	12155	2.90	97.9	209.50	628.51	726.4	17.37	1.04	600	CONC	0.013	654.4	2.24	111%

Proposed Dry Conditions Sanitary Flow

320 McCowan Road

Project No.: 19066

Composite population density per unit Single - 3.5 Semi & Townhouse - 2.7	Population density-Commercial 1.1 p / 100 m ² Population density-Office 3.3 p / 100 m ² Population density-Institutional 86 p / hectare	Page 1 of 1
P = population = sum p A site	A = area ha	Date: 19-May-23 Designed by: PA Checked by: RS
$M = 1 + (14)/(4 + (P/10^3))^{0.5}$ I = 0.26 x A gross L/s q = 240 L/person/day (For the analysis of existing sewers)	Pflow = M q P / 86400 = population flow L/s Q TOTAL = P flow + I L/s	

STREET	MANHOLE		CUMULATIVE					A GROSS ha	I L/s	Q TOTAL L/s	LENGTH (m)	SLOPE S %	D (mm)	TYPE OF PIPE	ROUGH. COEFF.	Q FULL (L/s)	Velocity Full (m/s)	Capacity (%)
	FROM	TO	AREA ha	EQUIV. POP P	POP. P	M	PEAK FLOW L/s											
Easement Sewer (site)	MH26	MH25	2.38	1088	1088	3.80	11.5	2.38	0.62	12.1	42.50	0.50	250	CONC	0.013	43.9	0.87	28%
Easement Sewer	MH25	MH24	0.60	174	1262	3.70	13.0	2.98	0.77	13.7	89.20	0.62	250	CONC	0.013	48.8	0.96	28%
Easement Sewer	MH24	MH23	0.00	0	1262	3.70	13.0	2.98	0.77	13.7	9.00	0.39	250	CONC	0.013	38.7	0.76	35%
Input: Eglinton Avenue East	MH23	MH22	8.14	1005	2267	3.50	22.0	11.12										
Eglinton Ave E	MH23	MH22	0.48	9	2276	3.50	22.1	11.60	3.02	25.1	39.93	0.88	250	CONC	0.013	58.2	1.15	43%
Eglinton Ave E	MH22	MH21	0.00	0	2276	3.50	22.1	11.60	3.02	25.1	7.01	1.56	250	CONC	0.013	77.5	1.53	32%
Eglinton Ave E	MH21	MH20	0.16	0	2276	3.50	22.1	11.76	3.06	25.2	62.79	1.14	250	CONC	0.013	66.2	1.31	38%
Input: Barbados Blvd E	MH20	MH19	2.55	74	2350			14.31										
Eglinton Ave E	MH20	MH19	0.00	0	2350	3.50	22.8	14.31	3.72	26.6	12.19	1.48	250	CONC	0.013	75.5	1.49	35%
Input: Barbados Blvd W	MH19	MH18	156.80	4451	6801			171.11										
North of Barbados Blvd	MH19	MH18	0.04	0	6801	3.10	58.6	171.15	44.50	103.1	29.87	0.22	600	CONC	0.013	301.0	1.03	34%
Input: Eglinton E North			5.50	114	6915			176.65										
North of Barbados Blvd	MH18	MH17	5.02	519	7434	3.10	64.0	181.67	47.23	111.2	98.45	0.38	600	CONC	0.013	395.6	1.35	28%
North of Barbados Blvd	MH17	MH16	0.25	0	7434	3.10	64.0	181.92	47.30	111.3	87.17	0.42	600	CONC	0.013	415.9	1.42	27%
North of Trudelle St	MH16	MH15	0.00	0	7434	3.10	64.0	181.92	47.30	111.3	4.00	0.31	600	CONC	0.013	357.3	1.22	31%
Input: Trudelle St W	MH15	MH14	5.28	661	8095			187.20										
North of Trudelle St	MH15	MH14	0.00	0	8095	3.00	67.5	187.20	48.67	116.1	23.00	0.31	600	CONC	0.013	357.3	1.22	33%
Input: Trudelle St E			12.11	2378	10472			199.31										
West Highland Creek	MH14	MH13	0.00	0	10472	2.90	84.4	199.31	51.82	136.2	60.17	0.31	600	CONC	0.013	357.3	1.22	38%
West Highland Creek	MH13	MH12	0.00	0	10472	2.90	84.4	199.32	51.82	136.2	42.98	0.31	600	CONC	0.013	357.3	1.22	38%
West Highland Creek	MH12	MH11	0.00	0	10472	2.90	84.4	199.32	51.82	136.2	111.25	0.35	600	CONC	0.013	379.6	1.30	36%
West Highland Creek	MH11	MH10	0.00	0	10472	2.90	84.4	199.32	51.82	136.2	63.09	0.40	600	CONC	0.013	405.8	1.39	34%
West Highland Creek	MH10	MH9	0.00	0	10472	2.90	84.4	199.33	51.82	136.2	72.54	0.37	600	CONC	0.013	390.3	1.34	35%
Input: Danforth Rd North	MH9	MH8	5.40	1034	11506			204.73										
West Highland Creek	MH9	MH8	0.00	0	11506	2.90	92.7	204.73	53.23	145.9	26.52	0.40	600	CONC	0.013	405.8	1.39	36%
Input: Danforth Rd south	MH8	MH7	1.06	1078	12584			205.79										
West Highland Creek	MH8	MH7	0.00	0	12584	2.90	101.4	205.79	53.51	154.9	22.25	0.74	600	CONC	0.013	552.0	1.89	28%
Input: Strode Dr	MH7	MH6	2.05	386	12970			207.84										
West Highland Creek	MH7	MH6	0.00	0	12970	2.80	100.9	207.85	54.04	154.9	28.96	0.50	600	CONC	0.013	453.7	1.55	34%
West Highland Creek	MH6	MH5	0.00	0	12970	2.80	100.9	207.85	54.04	154.9	65.23	0.49	600	CONC	0.013	449.2	1.54	34%
Input: Graylee Ave	MH5	MH4	1.64	84	13054			209.49										
West Highland Creek	MH5	MH4	0.00	0	13054	2.80	101.5	209.49	54.47	156.0	70.41	0.51	600	CONC	0.013	458.3	1.57	34%
West Highland Creek	MH4	MH3	0.00	0	13054	2.80	101.5	209.49	54.47	156.0	76.81	0.44	600	CONC	0.013	425.6	1.46	37%
West Highland Creek	MH3	MH2	0.00	0	13054	2.80	101.5	209.50	54.47	156.0	74.98	0.46	600	CONC	0.013	435.2	1.49	36%
West Highland Creek	MH2	MH1	0.00	0	13054	2.80	101.5	209.50	54.47	156.0	83.82	0.60	600	CONC	0.013	497.0	1.70	31%
West Highland Creek	MH1	Trunk	0.00	0	13054	2.80	101.5	209.50	54.47	156.0	17.37	1.04	600	CONC	0.013	654.4	2.24	24%

Proposed Wet Conditions Sanitary Flow

320 McCowan Road

Project No.: 19066

Composite population density per unit Single - 3.5 Semi & Townhouse - 2.7	Population density-Commercial 1.1 p / 100 m ² Population density-Office 3.3 p / 100 m ² Population density-Institutional 86 p / hectare	Page 1 of 1
P = population = sum p A site	A = area ha	Date: 19-May-23 Designed by: PA Checked by: RS
$M = 1 + (14)/(4+(P/10^3))^{0.5}$	Pflow = M q P / 86400 = population flow L/s	
I = 0.26 x A gross L/s	Q TOTAL = P flow + I L/s	
q = 240 L/person/day (For the analysis of existing sewers)		

STREET	MANHOLE		AREA ha	EQUIV. POP P	CUMULATIVE			A GROSS ha	I L/s	Q TOTAL L/s	LENGTH (m)	SLOPE S %	D (mm)	TYPE OF PIPE	ROUGH. COEFF.	Q FULL (L/s)	Velocity Full (m/s)	Capacity (%)
	FROM	TO			POP. P	M	PEAK FLOW L/s											
Easement Sewer (site)	MH26	MH25	2.38	1088	1088	3.80	11.5	2.38	23.80	35.3	42.50	0.50	250	CONC	0.013	43.9	0.87	80%
Easement Sewer	MH25	MH24	0.60	174	1262	3.70	13.0	2.98	29.80	42.8	89.20	0.62	250	CONC	0.013	48.8	0.96	88%
Easement Sewer	MH24	MH23	0.00	0	1262	3.70	13.0	2.98	29.80	42.8	9.00	0.39	250	CONC	0.013	38.7	0.76	110%
Input: Eglinton Avenue East		MH23	8.14	1005	2267	3.50	22.0	11.12										
Eglinton Ave E	MH23	MH22	0.48	9	2276	3.50	22.1	11.60	58.00	80.1	39.93	0.88	250	CONC	0.013	58.2	1.15	138%
Eglinton Ave E	MH22	MH21	0.00	0	2276	3.50	22.1	11.60	58.00	80.1	7.01	1.56	250	CONC	0.013	77.5	1.53	103%
Eglinton Ave E	MH21	MH20	0.16	0	2276	3.50	22.1	11.76	58.80	80.9	62.79	1.14	250	CONC	0.013	66.2	1.31	122%
Input: Barbados Blvd E		MH20	2.55	74	2350			14.31										
Eglinton Ave E	MH20	MH19	0.00	0	2350	3.50	22.8	14.31	71.56	94.4	12.19	1.48	250	CONC	0.013	75.5	1.49	125%
Input: Barbados Blvd W		MH19	156.80	4451	6801			171.11										
North of Barbados Blvd	MH19	MH18	0.04	0	6801	3.10	58.6	171.15	513.45	572.0	29.87	0.22	600	CONC	0.013	301.0	1.03	190%
Input: Eglinton E North			5.50	114	6915			176.65										
North of Barbados Blvd	MH18	MH17	5.02	519	7434	3.10	64.0	181.67	545.01	609.0	98.45	0.38	600	CONC	0.013	395.6	1.35	154%
North of Barbados Blvd	MH17	MH16	0.25	0	7434	3.10	64.0	181.92	545.76	609.8	87.17	0.42	600	CONC	0.013	415.9	1.42	147%
North of Trudelle St	MH16	MH15	0.00	0	7434	3.10	64.0	181.92	545.76	609.8	4.00	0.31	600	CONC	0.013	357.3	1.22	171%
Input: Trudelle St W		MH15	5.28	661	8095			187.20										
North of Trudelle St	MH15	MH14	0.00	0	8095	3.00	67.5	187.20	561.60	629.1	23.00	0.31	600	CONC	0.013	357.3	1.22	176%
Input: Trudelle St E			12.11	2378	10472			199.31										
West Highland Creek	MH14	MH13	0.00	0	10472	2.90	84.4	199.31	597.94	682.3	60.17	0.31	600	CONC	0.013	357.3	1.22	191%
West Highland Creek	MH13	MH12	0.00	0	10472	2.90	84.4	199.32	597.95	682.3	42.98	0.31	600	CONC	0.013	357.3	1.22	191%
West Highland Creek	MH12	MH11	0.00	0	10472	2.90	84.4	199.32	597.96	682.3	111.25	0.35	600	CONC	0.013	379.6	1.30	180%
West Highland Creek	MH11	MH10	0.00	0	10472	2.90	84.4	199.32	597.97	682.3	63.09	0.40	600	CONC	0.013	405.8	1.39	168%
West Highland Creek	MH10	MH9	0.00	0	10472	2.90	84.4	199.33	597.98	682.3	72.54	0.37	600	CONC	0.013	390.3	1.34	175%
Input: Danforth Rd North		MH9	5.40	1034	11506			204.73										
West Highland Creek	MH9	MH8	0.00	0	11506	2.90	92.7	204.73	614.19	706.9	26.52	0.40	600	CONC	0.013	405.8	1.39	174%
Input: Danforth Rd south		MH8	1.06	1078	12584			205.79										
West Highland Creek	MH8	MH7	0.00	0	12584	2.90	101.4	205.79	617.38	718.7	22.25	0.74	600	CONC	0.013	552.0	1.89	130%
Input: Strode Dr		MH7	2.05	386	12970			207.84										
West Highland Creek	MH7	MH6	0.00	0	12970	2.80	100.9	207.85	623.54	724.4	28.96	0.50	600	CONC	0.013	453.7	1.55	160%
West Highland Creek	MH6	MH5	0.00	0	12970	2.80	100.9	207.85	623.54	724.4	65.23	0.49	600	CONC	0.013	449.2	1.54	161%
Input: Graylee Ave		MH5	1.64	84	13054			209.49										
West Highland Creek	MH5	MH4	0.00	0	13054	2.80	101.5	209.49	628.47	730.0	70.41	0.51	600	CONC	0.013	458.3	1.57	159%
West Highland Creek	MH4	MH3	0.00	0	13054	2.80	101.5	209.49	628.48	730.0	76.81	0.44	600	CONC	0.013	425.6	1.46	172%
West Highland Creek	MH3	MH2	0.00	0	13054	2.80	101.5	209.50	628.49	730.0	74.98	0.46	600	CONC	0.013	435.2	1.49	168%
West Highland Creek	MH2	MH1	0.00	0	13054	2.80	101.5	209.50	628.50	730.0	83.82	0.60	600	CONC	0.013	497.0	1.70	147%
West Highland Creek	MH1	Trunk	0.00	0	13054	2.80	101.5	209.50	628.51	730.0	17.37	1.04	600	CONC	0.013	654.4	2.24	112%

Proposed Construction Wet Conditions Sanitary Flow

320 McCowan Road

Project No.: 19066

Composite population density per unit Single - 3.5 Semi & Townhouse - 2.7	Population density-Commercial 1.1 p / 100 m ² Population density-Office 3.3 p / 100 m ² Population density-Institutional 86 p / hectare
P = population = sum p A site	A = area ha
M = 1 + (14)/(4+(P/10 ³)) ^{0.5} I = 0.26 x A gross L/s q = 240 L/person/day (For the analysis of existing sewers)	Pflow = M q P / 86400 = population flow L/s Q TOTAL = P flow + I L/s
Page 1 of 1	
Date: 19-May-23 Designed by: PA Checked by: RS	

A GROSS - Cumulative Residential + Non-Residential Area

STREET	MANHOLE		AREA ha	EQUIV. POP P	CUMULATIVE			A GROSS ha	I L/s	Q TOTAL L/s	LENGTH (m)	SLOPE S %	D (mm)	TYPE OF PIPE	ROUGH. COEFF.	Q FULL (L/s)	Velocity Full (m/s)	Capacity (%)
	FROM	TO			POP. P	M	PEAK FLOW L/s											
Easement Sewer (site)	MH26	MH25	2.38	154	154	4.20	1.8	2.38	28.53	30.3	42.50	0.50	250	CONC	0.013	43.9	0.87	69%
Easement Sewer	MH25	MH24	0.60	174	328	4.10	3.7	2.98	34.53	38.3	89.20	0.62	250	CONC	0.013	48.8	0.96	78%
Easement Sewer	MH24	MH23	0.00	0	328	4.10	3.7	2.98	34.53	38.3	9.00	0.39	250	CONC	0.013	38.7	0.76	99%
Input: Eglinton Avenue East	MH23	MH23	8.14	1005	1333	3.70	13.7	11.12										
Eglinton Ave E	MH23	MH22	0.48	9	1342	3.70	13.8	11.60	62.73	76.5	39.93	0.88	250	CONC	0.013	58.2	1.15	131%
Eglinton Ave E	MH22	MH21	0.00	0	1342	3.70	13.8	11.60	62.73	76.5	7.01	1.56	250	CONC	0.013	77.5	1.53	99%
Eglinton Ave E	MH21	MH20	0.16	0	1342	3.70	13.8	11.76	63.53	77.3	62.79	1.14	250	CONC	0.013	66.2	1.31	117%
Input: Barbados Blvd E	MH20	MH20	2.55	74	1416			14.31										
Eglinton Ave E	MH20	MH19	0.00	0	1416	3.70	14.6	14.31	76.29	90.8	12.19	1.48	250	CONC	0.013	75.5	1.49	120%
Input: Barbados Blvd W	MH19	MH19	156.80	4451	5867			171.11										
North of Barbados Blvd	MH19	MH18	0.04	0	5867	3.20	52.1	171.15	518.18	570.3	29.87	0.22	600	CONC	0.013	301.0	1.03	189%
Input: Eglinton E North			5.50	114	5981			176.65										
North of Barbados Blvd	MH18	MH17	5.02	519	6500	3.10	56.0	181.67	549.74	605.7	98.45	0.38	600	CONC	0.013	395.6	1.35	153%
North of Barbados Blvd	MH17	MH16	0.25	0	6500	3.10	56.0	181.92	550.49	606.5	87.17	0.42	600	CONC	0.013	415.9	1.42	146%
North of Trudelle St	MH16	MH15	0.00	0	6500	3.10	56.0	181.92	550.49	606.5	4.00	0.31	600	CONC	0.013	357.3	1.22	170%
Input: Trudelle St W	MH15	MH15	5.28	661	7161			187.20										
North of Trudelle St	MH15	MH14	0.00	0	7161	3.10	61.7	187.20	566.33	628.0	23.00	0.31	600	CONC	0.013	357.3	1.22	176%
Input: Trudelle St E			12.11	2378	9538			199.31										
West Highland Creek	MH14	MH13	0.00	0	9538	3.00	79.5	199.31	602.67	682.2	60.17	0.31	600	CONC	0.013	357.3	1.22	191%
West Highland Creek	MH13	MH12	0.00	0	9538	3.00	79.5	199.32	602.68	682.2	42.98	0.31	600	CONC	0.013	357.3	1.22	191%
West Highland Creek	MH12	MH11	0.00	0	9538	3.00	79.5	199.32	602.69	682.2	111.25	0.35	600	CONC	0.013	379.6	1.30	180%
West Highland Creek	MH11	MH10	0.00	0	9538	3.00	79.5	199.32	602.70	682.2	63.09	0.40	600	CONC	0.013	405.8	1.39	168%
West Highland Creek	MH10	MH9	0.00	0	9538	3.00	79.5	199.33	602.71	682.2	72.54	0.37	600	CONC	0.013	390.3	1.34	175%
Input: Danforth Rd North	MH9	MH9	5.40	1034	10572			204.73										
West Highland Creek	MH9	MH8	0.00	0	10572	2.90	85.2	204.73	618.92	704.1	26.52	0.40	600	CONC	0.013	405.8	1.39	173%
Input: Danforth Rd south	MH8	MH8	1.06	1078	11650			205.79										
West Highland Creek	MH8	MH7	0.00	0	11650	2.90	93.8	205.79	622.11	716.0	22.25	0.74	600	CONC	0.013	552.0	1.89	130%
Input: Strode Dr	MH7	MH7	2.05	386	12036			207.84										
West Highland Creek	MH7	MH6	0.00	0	12036	2.90	97.0	207.85	628.27	725.2	28.96	0.50	600	CONC	0.013	453.7	1.55	160%
West Highland Creek	MH6	MH5	0.00	0	12036	2.90	97.0	207.85	628.27	725.2	65.23	0.49	600	CONC	0.013	449.2	1.54	161%
Input: Graylee Ave	MH5	MH5	1.64	84	12120			209.49										
West Highland Creek	MH5	MH4	0.00	0	12120	2.90	97.6	209.49	633.20	730.8	70.41	0.51	600	CONC	0.013	458.3	1.57	159%
West Highland Creek	MH4	MH3	0.00	0	12120	2.90	97.6	209.49	633.21	730.8	76.81	0.44	600	CONC	0.013	425.6	1.46	172%
West Highland Creek	MH3	MH2	0.00	0	12120	2.90	97.6	209.50	633.22	730.9	74.98	0.46	600	CONC	0.013	435.2	1.49	168%
West Highland Creek	MH2	MH1	0.00	0	12120	2.90	97.6	209.50	633.23	730.9	83.82	0.60	600	CONC	0.013	497.0	1.70	147%
West Highland Creek	MH1	Trunk	0.00	0	12120	2.90	97.6	209.50	633.24	730.9	17.37	1.04	600	CONC	0.013	654.4	2.24	112%

Counterpoint Engineering Inc.

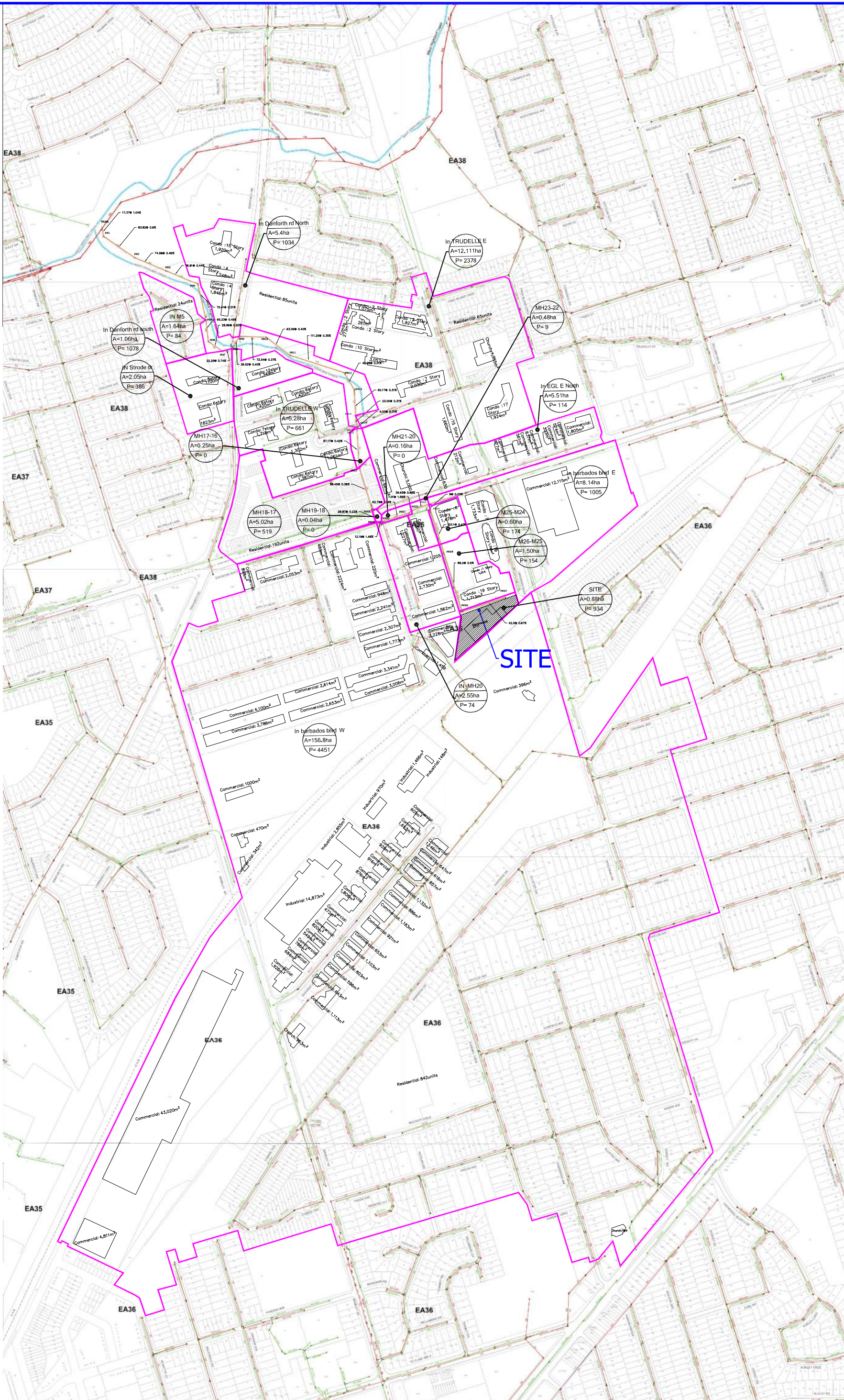
Wet Weather HGL - Proposed

Project: 320 MacCowan Road
 Project No: 19066
 Prepared by: PA
 Checked by: RS
 Date: 28-Apr-21

Trunk invert= 147.89
 Trunk invert= 147.1 mm
 Diameter 750.0 mm

Street	MANHOLES U/S	INVERT ELEV U/S (m)	D/S (m)	GROUND U/S (m)	COVER U/S (m)	PIPE Diam (mm)	Length (m)	'n'	Population San Flow (cms)	Infiltration/ Inflow Flow (cms)	TOTAL FLOW (cms)	Q-cap (cms)	Qin/ Qcap	Surch. (U/S) (m)	HGL(U/S) (m) *	HGL(D/S) (m) <- outlet	HEIGHT ABOVE OBV. (U/S)	Distance betw HGL & Ground (m)
--------	-----------------	---------------------------	------------	----------------------	---------------------	----------------------	---------------	-----	---------------------------------	---------------------------------------	------------------------	----------------	--------------	------------------------	-------------------	------------------------------	-------------------------------	--------------------------------------

West Highland Creek	MH1	147.41	147.23	0.18	152.48	4.466	600	17.37	0.013	0.10	0.54	0.642	0.625	1.03	0.0741	148.085	147.890	0.07	4.392
West Highland Creek	MH2	147.94	147.44	0.50	152.70	4.161	600	17.37	0.013	0.10	0.54	0.642	1.045	0.61	-0.0228	148.521	148.105	-0.02	4.184
West Highland Creek	MH3	148.36	148.02	0.34	152.83	3.868	600	83.82	0.013	0.10	0.54	0.642	0.394	1.63	0.0281	148.993	148.620	0.03	3.840
West Highland Creek	MH4	148.71	148.38	0.34	153.61	4.292	600	74.98	0.013	0.10	0.54	0.642	0.412	1.56	0.1235	149.438	149.073	0.12	4.169
West Highland Creek	MH5	149.13	148.77	0.36	154.02	4.295	600	76.81	0.013	0.10	0.54	0.642	0.420	1.53	0.1780	149.908	149.518	0.18	4.117
West Highland Creek	MH6	149.46	149.14	0.32	154.60	4.539	600	65.23	0.013	0.10	0.54	0.641	0.430	1.49	0.2719	150.334	149.988	0.27	4.267
West Highland Creek	MH7	149.69	149.54	0.15	154.57	4.277	600	65.23	0.013	0.10	0.54	0.641	0.291	2.21	0.2816	150.572	150.414	0.28	3.995
West Highland Creek	MH8	149.87	149.71	0.16	155.17	4.704	600	22.25	0.013	0.10	0.53	0.632	0.528	1.20	0.3598	150.830	150.652	0.36	4.344
West Highland Creek	MH9	150.02	149.91	0.11	155.16	4.539	600	26.52	0.013	0.09	0.53	0.623	0.389	1.60	0.4088	151.025	150.910	0.41	4.130
West Highland Creek	MH10	150.34	150.07	0.27	155.22	4.286	600	72.54	0.013	0.08	0.52	0.605	0.373	1.62	0.4589	151.395	151.105	0.46	3.827
West Highland Creek	MH11	150.65	150.40	0.25	156.04	4.789	600	83.09	0.013	0.08	0.52	0.605	0.386	1.57	0.4953	151.746	151.475	0.50	4.294
West Highland Creek	MH12	151.06	150.67	0.39	156.12	4.466	600	111.25	0.013	0.08	0.52	0.605	0.364	1.66	0.5888	152.248	151.826	0.59	3.877
West Highland Creek	MH13	151.33	151.20	0.13	156.42	4.490	600	42.98	0.013	0.08	0.52	0.605	0.343	1.76	0.5426	152.473	152.328	0.54	3.948
West Highland Creek	MH14	151.72	151.45	0.27	156.65	4.335	600	60.17	0.013	0.08	0.52	0.605	0.412	1.47	0.5289	152.846	152.553	0.53	3.806
North of Trudelle St	MH15	151.76	151.74	0.02	156.97	4.612	600	23.00	0.013	0.07	0.50	0.568	0.189	3.01	0.5894	152.950	152.926	0.59	4.022
North of Trudelle St	MH16	151.72	151.71	0.00	156.65	4.335	600	4.00	0.013	0.06	0.49	0.555	0.189	2.94	0.7165	153.034	153.030	0.72	3.618
North of barbados blvd	MH17	152.10	151.74	0.37	156.97	4.268	600	87.17	0.013	0.06	0.49	0.555	0.398	1.39	0.8050	153.509	153.114	0.81	3.463
North of barbados blvd	MH18	152.55	152.18	0.37	158.24	5.091	600	98.45	0.013	0.06	0.48	0.545	0.377	1.44	0.8392	153.992	153.589	0.84	4.251
North of barbados blvd	MH19	152.71	152.58	0.13	158.50	5.185	600	29.87	0.013	0.06	0.46	0.519	0.402	1.29	0.8992	154.210	154.072	0.90	4.286
Eglinton Ave E)	MH20	153.23	153.05	0.18	157.58	4.099	250	12.19	0.013	0.02	0.06	0.084	0.072	1.16	1.0025	154.485	154.290	1.00	3.097
Eglinton Ave E)	MH21	154.45	153.77	0.68	157.58	2.883	250	62.79	0.013	0.02	0.06	0.083	0.062	1.33	0.6048	155.303	154.565	0.60	2.279
Eglinton Ave E)	MH22	154.62	154.45	0.17	157.89	3.017	250	7.01	0.013	0.02	0.06	0.083	0.092	0.90	0.6817	155.551	155.383	0.68	2.336
Eglinton Ave E)	MH23	154.98	154.92	0.06	158.73	3.500	250	39.93	0.013	0.02	0.06	0.083	0.023	3.59	0.4656	155.696	155.631	0.47	3.034
McCowan Road	MH24	155.24	155.21	0.03	158.83	3.340	250	9.00	0.013	0.00	0.02	0.024	0.034	0.70	0.3126	155.803	155.776	0.31	3.027
McCowan Road	MH25	155.84	155.36	0.48	159.61	3.520	250	103.10	0.013	0.00	0.02	0.024	0.041	0.59	0.1814	156.271	155.883	0.18	3.339
McCowan Road	MH26	156.41	155.96	0.45	161.70	5.036	250	89.20	0.013	0.00	0.01	0.012	0.042	0.29	-0.0772	156.587	156.351	-0.08	5.113
McCowan Road.(site)	MH27	156.84	156.58	0.26	160.20	3.110	250	42.50	0.013	0.00	0.01	0.010	0.047	0.22	0.0000	157.090	156.830	0.00	3.110



SANITARY DRAINAGE PLAN

MIXED USE HIGHRISE CONDO
320 McCowan
TORONTO, ONTARIO

DRAWING BY: MA
CHECKED BY: RS
SCALE: NTS

DATE: APRIL 2021
PROJECT No. 19066
FIGURE No. 5



320 McCowan Road, Scarborough, Ontario

M1J 3N2

Supplementary Hydrogeological Investigation

Client:

*Blauson Assets Management Ltd. c/o Ling Kee Group
#106-320 McCowan Road, Scarborough, Ontario*

Attention: Allen Au

Type of Document:

FINAL

Project Name:

320 McCowan Road, Scarborough, Ontario

Project Number:

BRM-00255009-D0

EXP Services Inc.
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Date Submitted:

2021-04-22

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1 Introduction

1.1 Project Description

EXP Services Inc. (EXP) was retained by Blauson Assets Management Ltd. c/o Ling Kee Group. to prepare a Supplementary Hydrogeological Investigation Report associated with the proposed development located at 320 McCowan Road, Scarborough, Ontario (hereinafter referred to as the 'Site').

Based on the information provided to EXP, it is understood that the proposed development plan is to replace the existing parking garage with a twenty nine (29) storey high-rise structure and a twenty five (25) storey high-rise structure with a common seven (7) level parking structure (five (5) levels above ground and two (2) levels below ground). The Site location plan is shown on Figure 1.

EXP had recently completed the Preliminary Geotechnical and Hydrogeological Investigations along with Environmental Site Assessment to address design for high-rise development with one (P1) level of underground parking (EXP, 2020). It is EXP understanding that updated plans include increase in underground levels of parking structure to two levels (P2). Therefore, the supplementary geotechnical investigation (reported under separate cover) was conducted in conjunction with this supplementary hydrogeological investigation onsite to support new designs. The pertinent information gathered from the noted investigations is utilized for this report.

1.2 Project Objectives

The main objectives of the Supplementary Hydrogeological Investigation are as follows:

- Establish and refine the local hydrogeological settings within the Site;
- Assess construction dewatering flow rates and potential impacts;
- Evaluate long term foundation sub-drain discharge volumes;
- Assess groundwater quality;
- Prepare a Supplementary Hydrogeological Investigation Report; and,
- Prepare a Hydrological Review Summary Form (as per City's requirements).

1.3 Scope of Work

To achieve the investigation objectives, EXP has completed the following scope of work:

- Reviewed available geological and hydrogeological information for the Site;
- Drilled and installed three (3) 50 mm monitoring wells including two (2) wells to 8.8 and 9.0 mbgs and one (1) well to 20.0 mbgs in conjunction with geotechnical investigation;
- Developed all new monitoring wells and conducted Single Well Response Tests (SWRT) to assess hydraulic properties of the saturated soils at the Site;
- Conducted three (3) months of water level monitoring at all monitoring wells on site (total of 9), as per City's requirements.

- Collected one (1) groundwater sample for the City of Toronto Sanitary and Storm Sewer Use By-Law parameters at selected monitoring well location;
- Evaluated the information collected during the field investigation program, including borehole geological information, SWRT results, groundwater level measurements and groundwater water quality;
- Prepared site plans, cross sections and groundwater contour mapping for the Site;
- Provided preliminary recommendations on the requirements for construction and long-term dewatering;
- Assessed potential impacts and provided recommendations on the Ministry of Environment, Conservation and Parks (MECP) Water Taking Permits and City of Toronto Sewer Discharge Agreements (SDA) for the construction and post-construction phases;
- Prepared a Supplementary Hydrogeological Investigation Report; and
- Prepared a City of Toronto Hydrological Review Summary Form.

The Supplementary Hydrogeological Investigation was prepared in accordance with the Ontario Water Resources Act, Ontario Regulation 387/04, and Toronto Municipal Code 681-Sewers. The scope of work outlined above was made to assess dewatering and did not include a review of Environmental Site Assessments (ESA).

1.4 Review of Previous Reports

The following reports were reviewed as part of this Supplementary Hydrogeological Investigation:

- EXP Services Inc. (February 2020), Preliminary Geotechnical Investigation, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.
- EXP Services Inc. (February 2020), Hydrogeological Investigation, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.
- EXP Services Inc. (February 2020), Phase One Environmental Site Assessment, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.
- EXP Services Inc. (March 2020), Phase Two Environmental Site Assessment, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.
- EXP Services Inc. (February 2021), Supplementary Geotechnical Investigation, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.

2 Hydrogeological Setting

2.1 Regional Setting

2.1.1 Regional Physiography

The Site is within a physiographic region named as the South Slope. The physiographic landform is known as the (Drumlinized) Till Plains. The Peel Plain lies to the north of the South Slope (Chapman & Putnam, 2007).

The South Slope is a sloping plain that extends along the southern boundary of the Peel Plain to the shoreline of former Lake Iroquois. The Till Plains were created by glacial movement and is characterized by gently sloping topography, with a gradual downward slope toward Lake Ontario.

2.1.2 Regional Geology and Hydrogeology

The surficial geology can be described as stone-poor, carbonate-derived silty to sandy till (Ministry of Northern Development and Mines, 2012). The surficial geology of the Site and surrounding areas is shown on Figure 2.

Based on the available regional geology maps, the subsurface stratigraphy of the Site from top to bottom is summarized in Table 2-1 (TRCA, 2008 and Oak Ridge Moraine Groundwater Program, 2018). The overburden thickness is approximately 114 meters (m).

Table 2-1: Summary of Subsurface Stratigraphy

Stratigraphic Unit	General Description	Top Elevation of Stratigraphic Unit
Halton Till or Equivalent (Aquitard)	This lithologic unit typically consists of sandy silt to clayey silt till interbedded with silt, clay, sand and gravel.	162
Oak Ridges Moraine or Equivalent (Aquifer)	This geology unit mainly consists of interbedded fine-grained sand and silt deposits where coarse-grained sand and gravel along with clay laminae are locally reported.	N/A
Newmarket Till (Aquitard)	This lithologic unit mainly consist of a massive and dense silty sand unit.	155
Thornccliffe Formation (Aquifer)	This geology formation generally consists of glaciofluvial (sand, silty sand) or glaciolacustrine deposits (silt, sand, pebbly silt and clay).	150
Sunnybrook Formation (Aquitard)	This lithologic unit was deposited near an ice sheet. It predominately consists of silt and clay.	N/A
Scarborough Formation (Aquifer)	This geology unit is interpreted as deposits of a fluvial-deltaic system fed by large braided melt-water rivers draining from an ice sheet. It consists of peat sand overlaying silt and clay deposits.	120

Stratigraphic Unit	General Description	Top Elevation of Stratigraphic Unit
Georgian Bay Formation	Bedrock primarily consists of interbedded shale, limestone, dolostone and siltstone. It belongs to the Upper Ordovician, (Ministry of Northern Development and Mines, 2012).	48

Regional groundwater across the area flows southeast, towards Lake Ontario (Oak Ridge Moraine Groundwater Program, 2018). Local deviation from the regional groundwater flow pattern may occur in response to changes in topography and/or soils, as well as the presence of surface water features and/or existing subsurface infrastructure.

2.1.3 Existing Water Well Survey

Water Well Records (WWRs) were compiled from the database maintained by the Ministry of the Environment, Conservation and Parks (MECP) and reviewed to determine the number of water wells documented within a 500-m radius of the Site boundaries. The locations of the MECP WWRs within 500 m of the Site are shown on Figure 3. A summary of the WWR is included in Appendix A.

The MECP WWR database indicates that one hundred and thirteen (113) records within a 500 m radius from the Site centroid (Figure 3 and Appendix A). All well records are reportedly located off-site.

The database indicates that all off-site wells are at an approximate distance of seventy-one (71) m or greater from the Site centroid. All offsite wells were reportedly identified as monitoring and observation wells (14), test holes (54), water supply well (1), abandoned wells (41) and/or listed with unknown use (3).

The Well Identification Number (Well ID No.) of the only off-site water supply well is 6905249 where it is reportedly located 269 m from the Site centroid.

The reported water levels ranged from depths of 1.5 m to 6.0 meters below ground surface (mbgs).

Based on the date of installation of the water supply well (March 11, 1963) and since the area is municipally serviced, it is unlikely that the noted water supply well is still active.

2.2 Site Setting

2.2.1 Site Topography

The Site is located in an urbanized land use setting.

The topography is considered relatively flat, with an overall gradual downwards slope to the south towards Lake Ontario.

As indicated on the borehole logs included in Appendix B, the surface elevation of the Site ranges between approximately 161.11 to 163.78 meters above sea level (masl).

2.2.2 Local Surface Water Features

The Site is within the Highland Creek watershed. No surface water features exist onsite. The nearest surface water feature is West Highland Creek, a tributary of the Highland Creek, located approximately 550 m northwest of the Site. Lake Ontario is approximately 2.2 km from the Site boundary to the southeast.

2.2.3 Local Geology and Hydrogeology

A summary of subsurface soil stratigraphy is provided in the following paragraphs. The soil descriptions are based on the preliminary and supplementary geotechnical investigation reports (EXP, 2020 and 2021) and they are summarized for the hydrogeological interpretations. As such, the information provided in this section shall not be used for the construction design purposes.

The detailed soil profiles encountered in each borehole and the results of moisture content determinations are presented on the attached borehole logs (Appendix B) from the preliminary and supplementary geotechnical investigations (Boreholes 1, 2, 3, 4, 5, 201 and 202). It should be noted that the soil boundaries indicated on the borehole logs are inferred from non-continuous sampling and observations during drilling. These boundaries are intended to reflect approximate transition zones for the purpose of Supplementary Hydrogeological Investigation and shall not be interpreted as exact planes of geological change.

Boreholes 201 and 202 were advanced without sampling to a depth of about 6.1 m below existing grade, corresponding to approximate Elevations 155.0 to 157.7 m. For stratigraphy to these elevations reference is made to the boreholes drilled for the previous geotechnical investigation.

The "Notes on Sample Description" preceding the borehole logs form an integral part of and should be read in conjunction with this report. The following is a brief description of the soil conditions encountered during the investigation.

The stratigraphy of the site, as revealed in the boreholes, is variable and generally comprised pavement structure underlain by fill followed by native deposits of clayey sandy silt till, sandy silt till, silty sand till and silty sand.

Based on the results of the geotechnical investigations, the general subsurface soil stratigraphy consist of the following units from top to maximum depth of investigation onsite as follows:

Pavement Structure

A flexible pavement structure, comprising 80 to 100 mm asphalt over about 250 mm granular material, was encountered in Boreholes 1 and 3.

Fill

Fill was encountered surficially in Boreholes 2, 4 and 5, and below the pavement structure in Boreholes 1 and 3. The fill extended to depths of about 2.3 to 4.5 m below existing grade.

The fill generally comprised clayey silt with some sand, trace gravel and trace organics. Locally, sandy silt with trace gravel was encountered in Borehole 2. Pieces of concrete and brick were also encountered within the fill in Boreholes 2 and 5. Moisture contents varied from about 10 to 29%, indicating a moist condition and the presence of organics.

Clayey Sandy Silt Till

A clayey sandy silt till deposit was encountered below the fill in all boreholes, and extended to depths of about 7.1 to 11.7 m below existing grade.

The clayey sandy silt till was brown and grey in colour, and contained trace to some gravel and occasional cobbles. Wet sand seams and layers were also noted in the clayey sandy silt till. Based on the pocket penetrometer test, SPT 'N' values and the shear vane test, the consistency of the clayey sandy silt till was found to be very stiff to hard and became soft to firm with depth. Moisture contents varied from about 10 to 26%, indicating a moist to wet condition.

Sandy Silt Till

A sandy silt till deposit was encountered below the clayey sandy silt till in all boreholes, and extended to depths of about 10.8 to 14.7 m below existing grade.

The sandy silt till was grey in colour and contained trace to some clay and trace gravel. Occasional sand/silt seams and cobbles were noted in the sandy silt till deposit. Based on the SPT 'N' values, the sandy silt till existed in a very dense state of compactness. The sandy silt till was generally in a moist condition with moisture contents ranging from about 7 to 14%, indicating a moist condition.

Silty Sand Till

Silty sand till deposit was encountered below the sandy silt till in Boreholes 201 and 202, and extended to the depths of about 19.1 to 19.3 m below existing grade. The silty sand till deposit was grey in colour and contained trace clay and gravel, and occasional wet sand seams and cobbles and boulders. Occasional wet gravelly sand layers were also noted within the lower portion of the deposit in both boreholes. Based on the obtained SPT 'N' values, the silty sand till deposit was in a dense to very dense state of compactness. Moisture contents of the silty sand till deposit ranged from about 8 to 19%, indicating moist to wet conditions.

Silty Sand

Silty sand deposit was encountered below the silty sand till in Boreholes 201 and 202, and extended to the termination depths of about 19.9 to 20.2 m below existing grade. The silty sand deposit was grey in colour and contained trace gravel. Based on the obtained SPT 'N' values, the silty sand deposit was in a very dense state of compactness. Moisture contents of the silty sand deposit ranged from about 8 to 15%, indicating a wet condition.

The borehole and monitoring well locations are shown on Figure 4. Geological cross-sections were generated based on the available borehole logs completed as part of the current investigations and shown on Figure 5A (Cross section A-A') and on Figure 5B (Cross section B-B'). Borehole logs used to generate both cross-sections are provided in Appendix B.

3 Results

3.1 Monitoring Well Details

The monitoring well network was installed as part of the previous Hydrogeological, Geotechnical and Environmental and current Supplementary Hydrogeological and Geotechnical Investigations at the Site. It consists of the following:

- Five (5) shallow overburden monitoring wells (BH 1, BH 2A, BH 3, BH 4 and BH 5) varying in depths from 6.2 to 7.7 mbgs;
- Three (3) intermediate overburden monitoring wells (BH 2, BH 203 and BH 204) varying in depths from 8.8 to 13.8 mbgs;
- One (1) deep overburden monitoring well (BH 201) was installed to approximately 20.0 mbgs.

All EXP wells were installed between 2019 and 2021. The diameter of all monitoring wells is 50 mm. All wells were installed with a 3 m long screens and completed with flush mount protective casings.

Borehole logs and monitoring well installation details are provided in Appendix B. The monitoring well locations are shown on Figure 4.

3.2 Water Level Monitoring

As part of the previous Hydrogeological and current Supplementary Hydrogeological Investigation, static water levels in the monitoring wells were recorded in number of monitoring events (total of 15) between November 26 of 2019 and April 20 of 2021.

The groundwater elevation recorded in the shallow wells ranged from 157.89 masl (4.20 mbgs at BH 3 on November 28, 2019) to 161.30 masl (2.33 mbgs at BH 5 on April 19, 2020). The groundwater elevation recorded in the intermediate wells ranged from 156.18 masl (5.71 mbgs at BH 203 on January 21, 2021) to 160.47 masl (2.36 mbgs at BH 204 on March 30, 2021). The groundwater elevation recorded for the deep well (BH 201) ranged from 159.54 masl (1.57 mbgs on February 2 and 25, 2021) to 159.80 masl (1.31 mbgs on April 20, 2021).

A summary of all static water level data as it relates to the elevation survey is given in Table 3-1 below.

Table 3-1: Summary of Measured Groundwater Elevations

Monitoring Well ID	Ground Surface Elevation (masl)	Approximate Full Well Depth (mbgs)	Depth	26-Nov-2019	28-Nov-2019	05-Dec-2019	09-Dec-2019	06-Jan-2020	20-Jan-2020	04-Feb-2020	19-Apr-2020	19-Jan-2021	21-Jan-2021	02-Feb-2021	25-Feb-2021	17-Mar-2021	30-Mar-2021	20-Apr-2021
BH 1	161.12	6.2	mbgs	1.50	1.51	1.68	1.58	1.51	1.07	1.06	1.20	1.70	1.70	1.80	1.89	1.63	1.49	1.46
			masl	159.63	159.62	159.44	159.54	159.62	160.05	160.06	159.92	159.42	159.42	159.32	159.23	159.49	159.63	159.66
BH 2A	163.78	6.4	mbgs	3.17	3.22	3.40	4.00	3.23	2.93	2.84	2.97	3.30	3.23	3.16	3.48	3.27	3.01	3.02
			masl	160.61	160.57	160.38	159.78	160.56	160.85	160.94	160.82	160.48	160.55	160.62	160.30	160.51	160.77	160.76
BH 2	163.58	13.8	mbgs	3.89	4.44	3.88	4.92	4.27	3.15	3.14	3.20	4.36	4.10	3.93	4.22	3.98	3.71	3.67
			masl	159.69	159.14	159.70	158.66	159.31	160.44	160.44	160.38	159.22	159.48	159.65	159.36	159.60	159.87	159.91
BH 3	162.09	6.2	mbgs	3.60	4.20	1.92	2.05	2.25	2.35	2.02	2.23	2.65	2.47	2.44	2.52	1.79	2.25	2.24
			masl	158.50	157.89	160.17	160.04	159.84	159.74	160.08	159.86	159.44	159.62	159.65	159.57	160.30	159.84	159.85
BH 4	163.09	6.3	mbgs	2.46	2.48	2.67	2.48	2.24	2.00	2.17	2.34	2.49	2.48	2.57	2.75	2.55	2.31	2.33
			masl	160.63	160.61	160.42	160.61	160.85	161.09	160.92	160.75	160.60	160.61	160.52	160.34	160.54	160.78	160.76
BH 5	163.63	7.7	mbgs	3.05	3.81	3.35	2.69	2.52	2.48	2.46	2.33	2.76	2.80	2.87	3.10	2.78	2.50	2.60
			masl	160.58	159.82	160.28	160.94	161.11	161.14	161.17	161.30	160.87	160.83	160.76	160.53	160.85	161.13	161.03
BH 201	161.11	20.0	mbgs	ND	ND	ND	ND	ND	ND	ND	ND	1.50	1.40	1.57	1.57	1.47	1.40	1.31
			masl										159.61	159.72	159.54	159.54	159.64	159.71
BH 203	161.89	9.0	mbgs	ND	ND	ND	ND	ND	ND	ND	ND	5.63	5.71	5.62	5.66	5.58	5.22	5.15
			masl										156.26	156.18	156.27	156.23	156.31	156.67
BH 204	162.83	8.8	mbgs	ND	ND	ND	ND	ND	ND	ND	ND	3.87	2.50	2.72	2.79	2.54	2.36	2.43
			masl										158.96	160.33	160.11	160.04	160.29	160.47

Notes:
 mbgs- meters below ground surface;
 masl- meters above sea level;
 ND- not drilled.



One (1) map was created for the Site to show groundwater contour of the intermediate water-bearing zone (Figures 6). The groundwater flow direction in this zone is interpreted to be northeast of the Site.

For the design of foundations without perimeter and foundation drainage systems, shallow wells need to be considered to evaluate the shallow groundwater table. The hydrogeologist needs to be consulted during the design process.

Groundwater levels are expected to show seasonal fluctuations and vary in response to prevailing climate conditions. This may also affect the direction and rate of flow. It is recommended to conduct seasonal groundwater level measurements to provide more information on seasonal groundwater level fluctuations.

3.3 Hydraulic Conductivity Testing

Six (6) Single Well Response Tests (SWRT's) were completed on monitoring wells BH 1, BH 2, BH 2A, BH 3, BH 4, and BH 5 on November 28, 2019. Additional six (6) Single Well Response Tests (SWRT's) were completed on monitoring wells BH 201, BH 203, and BH 204 on January 19 and 21, 2021. All tests were completed to estimate the saturated hydraulic conductivity (K) of the soils at the well screen depths utilizing data transducers (data loggers, preprogrammed on one (1) second intervals).

The static water level within each monitoring well was measured prior to the start of testing. In advance of performing SWRTs, each monitoring well underwent development to remove fines introduced into the screens following construction. The development process involved purging of the monitoring wells to induce the flow of fresh formation water through the screen. Each monitoring well was permitted to fully recover prior to performing SWRTs.

The SWRT tests was conducted using a Solinst® Model 3001 Levellogger Edge, preprogrammed on one second intervals, and a Solinst® Water Level Tape.

Hydraulic conductivity values were calculated from the SWRT and constant rate test data as per Hvorslev's solution included in the Aqtesolv Pro. V.4.5 software package. The semi-log plots for normalized drawdown versus time are included in Appendix C.

A summary of the hydraulic conductivity (K) values estimated from the SWRTs are provided in Table 3-2.

Table 3-2: Summary of Hydraulic Conductivity Testing

Monitoring Well ID	Well Depth (mbgs)	Screen Interval (mbgs)	Predominant Formation Screened	Estimated Hydraulic Conductivity (m/s) Falling Head SWRT	Estimated Hydraulic Conductivity (m/s) Rising Head SWRT
BH 1	6.2	3.2 - 6.2	Clayey Sandy Silt Till	-	4.4E-08
BH 2A	6.4	4.8 - 6.4	Clayey Sandy Silt Till	-	4.9E-07
BH 2	13.8	10.8 - 13.8	Clayey Sandy Silt Till/ Sandy Silt Till	4.4E-08	-
BH 3	6.2	4.7 - 6.2	Clayey Sandy Silt Till	-	8.9E-09
BH 4	6.3	4.8 - 6.3	Clayey Sandy Silt Till	-	8.9E-07
BH 5	7.7	6.1 - 7.7	Clayey Sandy Silt Till	-	3.5E-08
BH 201	20.0	17 - 20	Silty Sand Till	1.4E-06	1.2E-06
BH 203	9.0	6 - 9	Clayey Sandy Silt Till/ Sandy Silt Till	1.1E-06	3.3E-07

Monitoring Well ID	Well Depth (mbgs)	Screen Interval (mbgs)	Predominant Formation Screened	Estimated Hydraulic Conductivity (m/s) Falling Head SWRT	Estimated Hydraulic Conductivity (m/s) Rising Head SWRT
BH 204	8.8	5.8 - 8.8	Clayey Sandy Silt Till/ Sandy Silt Till	1.5E-06	9.3E-07
Highest K Value Estimated for the clayey sandy silt till				9.3E-07	
Geometric Mean of the K Values Estimated for clayey sandy silt till				6.6E-08	

SWRTs provide K-estimates of the geological formation surrounding the well screens and may not be representative of bulk formation hydraulic conductivity. As shown in Table 3-2, the highest K-value of clayey sandy silt till is 9.3E-07 m/s, and the geometric mean of the K values is 6.6E-08 m/s.

3.4 Groundwater Quality

To assess the suitability for discharging pumped groundwater into the sewers owned by the City of Toronto during dewatering activities, one (1) groundwater sample was collected from monitoring well BH 1 on November 28, 2019 during previous hydrogeological investigation and one (1) groundwater sample was collected from monitoring well BH 203 on January 21, 2021 during current investigation.

All groundwater samples were collected using a peristaltic pump. Prior to collecting the noted water sample, approximately three (3) standing well volumes of groundwater were purged from the referred well. The samples were collected unfiltered and placed into pre-cleaned laboratory-supplied vials and/or bottles provided with analytical test group specific preservatives, as required. Dedicated nitrile gloves were used during sample handling. The groundwater samples were submitted for analysis to Bureau Veritas Laboratory, a CALA certified independent laboratory in Mississauga, Ontario. Analytical results are provided in Appendix D.

Table 3-3 summarizes exceedance(s) of the Sanitary (Table 1) and Storm (Table 2) Sewer Use By-Law parameters.

When comparing the chemistry of the collected groundwater samples to the City of Toronto Sanitary and Combined Sewer Discharge Criteria (Table 1), there were no parameter exceedances reported.

When comparing the chemistry of the collected groundwater samples to the City of Toronto Storm Sewer Discharge Criteria (Table 2) the following parameters reported an exceedance: Total Manganese (BH 1 and BH 203), Total Suspended Solids (BH 203), Total Zinc (BH 203).

Reporting detection limits (RDLs) were below the Sewer Use By-Law parameter criteria of Tables 1 and 2.

Table 3-3: Summary of Analytical Results

Parameter	Units	City of Toronto Sanitary and Combined Sewer Discharge Limit (Table 1)	City of Toronto Storm Sewer Discharge Limit (Table 2)	Concentration BH 1 November 28, 2019	Concentration BH 203 January 21, 2021
Total Suspended Solids (TSS)	mg/L	350	15	15	33

Parameter	Units	City of Toronto Sanitary and Combined Sewer Discharge Limit (Table 1)	City of Toronto Storm Sewer Discharge Limit (Table 2)	Concentration BH 1 November 28, 2019	Concentration BH 203 January 21, 2021
Total Manganese (Mn)	µg/L	5,000	50	82	180
Total Zinc (Zn)	µg/L	2,000	40	<5.0	76

Bold – Exceeds City of Toronto Storm Sewer Discharge Limit (Table 2).

For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented (filtration or decantation facilities and/ or any other applicable treatment system) during construction dewatering activities to discharge to the applicable sewer system. The specifications of the treatment system will need to be adjusted to the reported water quality results by the treatment contractor/process engineer.

For the long-term dewatering discharge to the sanitary sewer system (post-development phase) and based on the water quality test results, the water is suitable to be discharged without a treatment system.

For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

The water quality results presented in this report may not be representative of the long-term condition of groundwater quality onsite. As such, regular water quality monitoring is recommended for the post-construction phase, as required by the City.

An agreement to discharge into the sewers owned by the City of Toronto will be required prior to releasing dewatering effluent.

The Phase Two Environmental Site Assessment Report (EXP, 2020) has been reviewed for more information on the soil and groundwater quality conditions at the Site.

Based on the analytical results completed, levels of electrical conductivity (EC) and sodium adsorption ratio (SAR) in soil as well as sodium and chloride in groundwater were found to be above the applicable Site Condition Standards in locations tested. EC and SAR as well as sodium and chloride parameters are typically associated with winter road salt applications. Most of these soil and groundwater samples were taken from boreholes/monitoring wells located on or in close proximity to pedestrian walkways, paved parking areas and driveways on Site or on adjacent properties. As such, it was determined that the source of the elevated EC and SAR in soil as well as sodium and chloride in groundwater was a result of winter road salt applications over walkways, parking areas and driveways on Site or adjacent properties for vehicular and pedestrian safety. No other potential sources of EC and SAR were identified on Site. Based on Section 49.1 of Ontario Regulation 153/04 (as amended), the applicable Site Conditions are deemed not to be exceeded under these conditions.

4 Construction Dewatering Assessment

It is our understanding that the proposed development plan is to replace the existing parking garage with a twenty nine (29) storey high-rise structure and a twenty five (25) storey high-rise structure with a common seven (7) level parking structure including two (2) levels below ground (P2).

It should be noted that the building construction and shoring drawings were not available at the time of this report. For this assessment, it was assumed that the proposed construction plans include an excavation with a soldier piles and lagging shoring system extending to the boundaries of the proposed construction. EXP should be retained to review the assumptions outlined in this section, when the designs for proposed shoring system is available.

Table 4-1 presents the assumptions used to calculate the dewatering rate for the Site.

Table 4-1 Dewatering Estimate Assumptions

Input Parameter	Assumption	Units	Notes
Ground Surface Elevation	162.15	masl	Based on Building Section A451.S, by Quadrangle Architects Limited
Top of Slab Elevation (P2)	154.15	masl	Based on Building Section A451.S, by Quadrangle Architects Limited
Lowest Footing Elevation	152.15	masl	Assumed to be approximately 2.0 m below the P2 Level
Dewatered Elevation Target	151.15	masl	Assumed to be approximately 1.0 m below the lowest footing elevation
Length of Excavation Area	127	m	Approximate length of Site for the proposed development
Width of Excavation Area	48	m	Approximate width of Site for the proposed development
Groundwater elevation	161.30	masl	The highest recorded groundwater elevation measured across the site (BH5 as measured on April 19, 2020).
Hydraulic Conductivity (K)	9.3×10^{-7}	m/s	Highest K values estimated for the clayey sandy silt till
Hydraulic Conductivity (K)	6.6×10^{-8}	m/s	Geometric mean K values estimated for the clayey sandy silt till

4.1 Dewatering Flow Rate Estimate and Zone of Influence

The Dupuit equation for steady linear flow to both sides of an excavation through an unconfined aquifer resting on a horizontal impervious surface was used to obtain a flow rate estimate. Dewatering flow rate is expressed as follows:

$$Q_w = xK(H^2 - h^2)/L_o$$

Where:

- Q_w = Rate of pumping (m³/sec)
- X = Length of excavation (m)
- K = Hydraulic conductivity (m/sec)
- H = Hydraulic head beyond the influence of pumping (static groundwater elevation) (m)
- h = Hydraulic head above the base of aquifer in an excavation (m)
- L_o = Distance of influence (m)

It is expected that the initial dewatering rate will be higher in order to remove groundwater from within the overburden formation. The dewatering rates are expected to decrease once the target water level is achieved in the excavation footprint as groundwater will have been removed, primarily from storage resulting in lower seepage rates into the excavation.

4.2 Sichardt's Radius of Influence

The radius of influence (R_s) for the construction dewatering was calculated based on Sichardt's equation. This equation is used to predict the distance at which the drawdown resulting from pumping is negligible. This empirical formula was developed to provide representative flow rates using the steady state flow dewatering equations, as discussed below.

The estimated radius of influence due to pumping is based on Sichardt's formula as follows:

$$R_s = C(H - h)\sqrt{K}$$

Where:

- R_s = Estimated Sichardt's radius of influence (m)
- H = Hydraulic head in aquifer (static water level or saturated depth) (m)
- h = Dynamic water level (m)
- K = Hydraulic conductivity (m/sec)
- C = Constant (3,000)

Based on Sichardt's formula and the highest K-value, the calculated distance of influence (L_o = R_o/2) is provided in Appendix E.

4.3 Stormwater

Additional pumping capacity may be required to maintain dry conditions within the excavation during and following significant precipitation events. Therefore, the dewatering rates at the Site should also include removing stormwater from the excavation.

A 15 mm precipitation event was utilized for estimating the stormwater volume. The calculation of the stormwater volume is included in Appendix E.

The estimate of the stormwater volume only includes direct precipitation into the excavation. The dimensions of the excavation are considered in the dewatering calculations. Runoff from outside of the excavation's footprint is excluded and it should be directed away from the excavation.

During precipitation events greater than 15 mm (ex: 100-year storm), measures should be taken by the contractor to retain stormwater onsite in a safe manner to not exceed the allowable water taking and discharge limits, as necessary. A two (2) and a one hundred (100) year storm event over a 24-hour period are 56.2 and 124.7 mm, respectively, which would correspond to 343 and 760 m³ of water from direct precipitation.

4.4 Results of Construction Dewatering Rate Estimate

For this assessment, it was assumed that the proposed construction plans include an excavation with shoring extending to the Site boundaries. EXP should be retained to review the assumptions outlined in this section, should the assumed shoring design change.

Short-term (construction) dewatering calculations are presented in Appendix E.

Pits (elevator, sump pits) are assumed to have the same excavation depth and dewatering target as the main excavation; deeper pits may require localized dewatering and revised dewatering estimates.

Based on the assumptions provided in this report, the results of the dewatering rate estimate can be summarized as follows:

Table 4-2 Summary of Dewatering Flow Rate Estimate

Location	Dewatering Flow Rate Without Safety Factor (including rainwater collection) (m ³ /day)	Dewatering Flow Rate With Safety Factor 2.0 (including rainwater collection) (m ³ /day)
P2 Extent	240 (rounded)	390 (rounded)

This peak dewatering flow rates does not account for flow from utility beddings and variations in hydrogeological properties beyond those encountered during this investigation.

Local dewatering may be required for pits (elevator pits, sump pits), if these extend deeper than the dewatering target. Local dewatering is not considered to be part of this assessment. Dewatering estimates should be reviewed once the pit dimensions are available.

All grading around the perimeter of the Site should be graded away from the shoring the systems. The dewatering assumptions are based on using shoring system without open cuts.

The maximum flow rate calculated with a high K value, provides a conservative estimate to account for higher than expected flow rates during the construction dewatering.

If caisson walls are installed, these should be designed for full hydrostatic pressure for shallow and deep water levels, without dewatering on the outside. Soldier pile and lagging and caisson wall systems should be designed to account for shallow groundwater conditions and take into consideration that dewatering systems may not provide fully dewatered conditions.

If caisson walls are used for decreasing long-term dewatering rates, these should be designed as permanent structures to allow for groundwater cutoff in the long-term. All perforations should be sealed permanently (ex: tiebacks, breaches and cold joints) with no leakages and inspected. Fillers should extend into low permeability deposits (ex: sound bedrock or till) to cutoff groundwater from water bearing zones. Inspections should be conducted to confirm the depth of low permeability deposits along shoring system and that fillers are keyed into low permeability soil deposits.

All grading around the perimeter of the construction Site should be graded away from the shoring the system.

The contractor is responsible to ensure that dry conditions are always maintained within the excavation at all costs.

4.5 Construction MECP Water Taking Permit

In accordance with the Ontario Water Resources Act, if the water taking for the construction dewatering is more than 50 m³/day but less than 400 m³/day, then an online registration in the Environmental Activity and Sector Registry (EASR) with the MECP will be required. If groundwater dewatering rates onsite exceed 400 m³/day, a Category 3 Permit to Take Water (PTTW) will be required from the MECP.

It is recognized that the maximum flow estimate calculated with a high K-value, provides a conservative estimate to account for higher than expected flow rates during the construction dewatering. Based on the dewatering estimate of approximately 390 m³/day for this project, an EASR would be required to facilitate the construction dewatering program of the Site.

A Discharge Plan (dewatering sketch, sewer discharge agreement) must be developed and applied for any discharges from the Site. The Discharge Plan and monitoring for both water quantity and water quality must be carried at the Site during the entire construction dewatering phase. The daily water taking records must be maintained onsite for the entire construction dewatering phase.

The EASR, Discharge Plan, hydrogeological investigation report, and geotechnical assessment of settlements must always also be available at the construction Site for the entire construction dewatering. EXP should be notified immediately about any changes to the construction dewatering schedule or design, since EASR will need to be updated to reflect these modifications. The hydrogeological report, EASR, Discharge Plan and geotechnical assessment constitutes the Water Taking Plan which needs to be available onsite for the duration of construction dewatering.

5 Sub-Drain Discharge Estimate

5.1 Long-Term Dewatering Rate Assessment

It is our understanding that the development plan includes a permanent foundation sub-drain system that will ultimately discharge to the municipal sewer system, if conventional footings are installed.

To estimate the groundwater flow to the future sub-drain, the Dupuit equation for steady linear flow to both sides of a partially-penetrating excavation through an unconfined aquifer resting on a horizontal impervious surface was used to obtain a flow rate estimate, which is expressed as follows:

$$Q_w = \left[0.73 + 0.23 \left(\frac{P}{H} \right) \right] x K (H^2 - h^2) / L_o$$

Where:

- Q_w = Rate of pumping (m³/sec)
- x = Length of excavation (m)
- P = Depth of penetration of drainage (m)
- K = Hydraulic conductivity (m/sec)
- H = Head beyond the influence of pumping (static groundwater elevation) (m)
- h = Head above base of the water-bearing zone at the excavation (m)
- L_o = Distance of influence (Ro/2) (m)

The calculation for the estimated flow to the future sub-drain system (with no cut-off walls) is provided in Appendix F. The dewatering target for the foundation drainage system is taken at 0.5 m below the lowest slab elevation.

The foundation drain analysis provides a preliminary flow rate estimate. Actual flow rate measurements of the sump discharge, once it is built, will be required to confirm these flow rates.

5.2 Sub-Drain Flow Rate Estimate

Based on the assumptions provided in this report and utilizing the geometric mean k-value for clayey sandy silt till (6.6×10^{-8} m/sec), the estimated sub-drain discharge volumes are summarized in Appendix F. Seasonal and daily fluctuations are expected. These estimates may be affected by hydrogeological conditions beyond those encountered at this time, fluctuations in groundwater regimes, surrounding Site alterations, and existing and future infrastructures.

Table 5-1 Long-Term Discharge Flow Rates

Location	Long-Term Dewatering Flow Rate (m ³ /day)	
	Without Safety Factor	With Safety Factor
P2 extent	40 (rounded)	49.6

For the design of water foundations without perimeter and foundation drainage systems, shallower wells need to be considered to evaluate the shallow groundwater table. The hydrogeologist needs to be consulted during the design process.

Intermittent cycling of sump pumps and seasonal fluctuation in groundwater regimes should be considered for pump specifications. A safety factor of 1.3 was applied to the flow rate to account for water level fluctuations due to seasonal changes.

These estimates assume that pits (elevator and/or sump pits) are made as watertight structures (without drainage), if their depths extend below the dewatering target, as previously stated. The dewatering assumptions are based on using shoring system without open cuts. Open cuts can act as preferential groundwater pathways in the long-term and cause foundation drainage volumes to increase.

If caisson walls are used for decreasing long-term dewatering rates, these should be designed as permanent structures to allow for groundwater cutoff in the long-term. All perforations should be sealed permanently (ex: tiebacks, breaches and cold joints) with no leakages and inspected. Fillers should extend into low permeability deposits (ex: sound bedrock or till) to cutoff groundwater from water bearing zones. Inspections should be conducted to confirm the depth of low permeability deposits along shoring system and that the fillers are keyed into low permeability soil deposits.

The sub-drain rate estimate is based on the assumptions outlined in this report. Any variations in hydrogeological conditions beyond those encountered as part of this investigation may significantly influence the sub-drain discharge volumes. Once the sub-drain system is in place, it is recommended to install a flow meter at the sump(s) to record daily discharge volumes during the commissioning stage of the system.

5.3 Post-Development MECP Water Taking Permit

In accordance with the Ontario Water Resources Act, if the water taking for the construction dewatering is more than 50 m³/day, then an application for a Category 3 Permit to Take Water (PTTW) will be required from the MECP.

It is recognized that the maximum flow rate calculated with a high K-value, provides a conservative estimate to account for higher than expected flow rates during the post-development dewatering. Based on the dewatering estimate of approximately 49.6 m³/day for this project, a Category 3 Permit to Take Water (PTTW) would not be required to facilitate the post-development phase.

6 Environmental Impact

6.1 Surface Water Features

No surface water features are located onsite. The nearest surface water feature is West Highland Creek, a tributary of the Highland Creek, located approximately 550 m northwest of the Site. Lake Ontario is approximately 2.2 km from the Site boundary to the southeast.

Due to the limited extent of zone of influence and the distance of the nearest surface water feature, no impacts to surface water features are expected during construction activities.

6.2 Groundwater Sources

Well Records from the MECP Water Well Record (WWR) Database were reviewed to determine the number of water supply wells present within a 500 m radius of the Site boundaries. Given that the dewatering zone of influence is limited, no dewatering related impact is expected on the water wells in the area.

6.3 Geotechnical Considerations

As per the MECP technical requirement for PTTW and EASRs, the geotechnical assessment of the stability of the soils due to water taking (ex: settlement, soil loss, subsidence, etc.) is required. The water taking should not have unacceptable interference on soils and underground structures (foundations, utilities, etc.).

A letter related to geotechnical issues as it pertains to the Site is required to be completed under a separate cover.

6.4 Groundwater Quality

It is our understanding that the potential effluent from the dewatering system during the construction will be released to the municipal sewer system. As such, the quality of groundwater discharge is required to conform the City of Toronto Sewer Use By-Law.

For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented (filtration or decantation facilities and/ or any other applicable treatment system) during construction dewatering activities to discharge to the applicable sewer system. The specifications of the treatment system will need to be adjusted to the reported water quality results by the treatment contractor/process engineer.

For the long-term dewatering discharge to the sanitary sewer system (post-development phase), and based on the water quality test results, the water is suitable to be released without a treatment system.

Dewatering (short and long-term) may induce migration of contaminants within the zone of influence and beyond due to changing hydraulic gradients, hydrogeological conditions beyond Site boundaries and preferential pathways in utility beddings etc. The water quality sampling conducted as part of this assessment was performed under static conditions. As a result, monitoring may be required during dewatering activities (short and long-term) to monitor potential migration, and this should be performed more frequently during early dewatering stages.

For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, it is recommended to implement a suitable pre-treatment, as required.

The water quality results presented in this report may not be representative of the long-term condition of groundwater quality onsite. As such, regular water quality monitoring is recommended for the post-construction phase as required by the City.

An agreement to discharge into the sewers owned by the City of Toronto will be required prior to releasing dewatering effluent.

The Phase Two Environmental Site Assessment Report (EXP, 2020) has been reviewed for more information on the soil and groundwater quality conditions at the Site.

Based on the analytical results completed, levels of electrical conductivity (EC) and sodium adsorption ratio (SAR) in soil as well as sodium and chloride in groundwater were found to be above the applicable Site Condition Standards. EC and SAR as well as sodium and chloride parameters are typically associated with winter road salt applications. Most of these soil and groundwater samples were taken from boreholes/monitoring wells located on or in close proximity to pedestrian walkways, paved parking areas and driveways on Site or on adjacent properties. As such, it was determined that the source of the elevated EC and SAR in soil as well as sodium and chloride in groundwater was a result of winter road salt applications over walkways, parking areas and driveways on Site or adjacent properties for vehicular and pedestrian safety. No other potential sources of EC and SAR were identified on Site. Based on Section 49.1 of Ontario Regulation 153/04 (as amended), the applicable Site Conditions are deemed not to be exceeded under these conditions.

6.5 Well Decommissioning

In conformance with Regulation 903 of the Ontario Water Resources Act, the installation and eventual decommissioning of any dewatering system wells or monitoring wells must be completed by a licensed well contractor. This will be required for all wells that are no longer in use.

7 Conclusions and Recommendations

Based on the findings of the Supplementary Hydrogeological Investigation, the following conclusions and recommendations are provided:

- When comparing the chemistry of the collected groundwater samples to the City of Toronto Sanitary and Combined Sewer Discharge Criteria (Table 1), there were no parameter exceedances reported.
- When comparing the chemistry of the collected groundwater samples to the City of Toronto Storm Sewer Discharge Criteria (Table 2) the following parameters reported an exceedance: Total Manganese (BH 1 and BH 203), Total Suspended Solids (BH 203), Total Zinc (BH 203).
- As required by the City of Toronto, this report was updated to include the results of the three months bi-weekly water level monitoring (15 events) completed between November 26, 2019 and April 20, 2021. The groundwater elevation recorded in the shallow wells ranged from 157.89 masl (4.20 mbgs at BH 3 on November 28, 2019) to 161.30 masl (2.33 mbgs at BH 5 on April 19, 2020). The groundwater elevation recorded in the intermediate wells ranged from 156.18 masl (5.71 mbgs at BH 203 on January 21, 2021) to 160.47 masl (2.36 mbgs at BH 204 on March 30, 2021). The groundwater elevation recorded for the deep well (BH201) ranged from 159.54 masl (1.57 mbgs on February 2 and 25, 2021) to 159.80 masl (1.31 mbgs on April 20, 2021).
- Based on the assumptions outlined in this report, the estimated peak dewatering pumping rate for proposed construction activities is approximately 390 m³/day. As the dewatering flow rate estimate exceeds 50 m³/day, an EASR would be required to facilitate the construction dewatering program for the Site.
- The long-term flow rate of the foundation sub-drain is estimated to be approximately 49.6 m³/day. It is recommended that once the sub-drain system is in place, a flow meter be installed at the sump(s) to record daily discharge volumes during the commissioning stage of the system. Regular maintenance/cleaning of the sub-drain system is recommended to ensure its proper operation. A PTTW will not be required for post construction phase.
- The construction dewatering and long-term estimate of sub-drain discharge volumes is based on the assumptions outlined in this report. Any variations in hydrogeological conditions beyond those encountered as part of this preliminary investigation may significantly influence the discharge volumes.
- For the short-term dewatering system (construction phase), it is anticipated that TSS levels and some other parameters (for example, Total Metals) in the pumped groundwater may become elevated and exceed both, Sanitary and Storm Sewer Use By-Law limits. To control the concentration of TSS and associated metals, it is recommended that a suitable treatment method be implemented (filtration or decantation facilities and/ or any other applicable treatment system) during construction dewatering activities to discharge to the applicable sewer system. The specifications of the treatment system will need to be adjusted to the reported water quality results by the treatment contractor/process engineer.
- For the long-term dewatering discharge to the sanitary sewer system (post-development phase) and based on the water quality test results, the water is suitable to discharge without a treatment system.
- For the long-term dewatering discharge to the storm sewer system (post-development phase) and based on the water quality results, it is recommended to implement a suitable pre-treatment as required.
- As per the MECP technical requirement for PTTW and EASRs, the geotechnical assessment of the stability of the soils due to water taking (ex: settlement, soil loss, subsidence etc.) is required. The water taking should not have unacceptable interference on soils and underground structures (foundations, utilities etc.). A letter related to geotechnical issues as it pertains to the Site is required to be completed under a separate cover.
- An agreement to discharge into the sewers owned by the City of Toronto will be required prior to releasing dewatering effluent.

- The EASR registration allows construction dewatering discharge of up to 400 m³/day. A Discharge Plan (dewatering sketch, sewer discharge agreement) must be developed and applied for any discharges from the Site. The Discharge Plan and monitoring for both water quantity and water quality must be carried at the Site during the entire construction dewatering phase. The daily water taking records must be maintained onsite for the entire construction dewatering phase. The EASR, Discharge Plan, hydrogeological investigation report, and geotechnical assessment of settlements must always also be available at the construction Site for the entire construction dewatering. EXP should be notified immediately about any changes to the construction dewatering schedule or design, since EASR will need to be updated to reflect these modifications. The hydrogeological report, EASR, Discharge Plan and geotechnical assessment constitutes the Water Taking Plan which needs to be available onsite for the duration of construction dewatering.
- In conformance with Regulation 903 of the Ontario Water Resources Act, the installation and eventual decommissioning of any dewatering system wells or monitoring wells must be completed by a licensed well contractor. This will be required for all wells that are no longer in use.

The conclusions and recommendations provided above should be reviewed in conjunction with the entirety of the report. They assume that the present design concept described throughout the report will proceed to construction. This report is solely intended for the construction and long-term dewatering assessments. Any changes to the design concept may result in a modification to the recommendations provided in this report.

8 Limitations

This report is based on a limited investigation designed to provide information to support an assessment of the current hydrogeological conditions within the study area. The conclusions and recommendations presented within this report reflect Site conditions existing at the time of the assessment. EXP must be contacted immediately, if any unforeseen Site conditions are experienced during construction activities. This will allow EXP to review the new findings and provide appropriate recommendations to allow the construction to proceed in a timely and cost-effective manner.

Our undertaking at EXP, therefore, is to perform our work within limits prescribed by our clients, with the usual thoroughness and competence of the geoscience/engineering profession. No other warranty or representation, either expressed or implied, is included or intended in this report.

This report was prepared for the exclusive use of Blauson Assets Management Ltd. c/o Ling Kee Group. This report may not be reproduced in whole or in part, without the prior written consent of EXP, or used or relied upon in whole or in part by other parties for any purposes whatsoever. Any use which a third party makes of this report, or any part thereof, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. EXP Services Inc. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report.

We trust that this information is satisfactory for your purposes. Should you have any questions or comments, please do not hesitate to contact this office.

Sincerely,

EXP Services Inc.



Nataliya Tkach, P. Geo., PMP, P.Eng.
Senior Hydrogeologist
Environmental Services



Francois Chartier, M.Sc., P. Geo.
Head of Hydrogeology Group
Environmental Services

9 References

Cashman and Preene (2013) Groundwater Lowering in Construction, 2nd Edition.

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EXP Services Inc. (March 2020), Phase Two Environmental Site Assessment, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.

EXP Services Inc. (February 2021), Supplementary Geotechnical Investigation, 320 McCowan Road, Scarborough, ON, prepared for Blauson Assets Management Ltd. c/o Ling Kee Group.



UNITED ENGINEERING INC
Mechanical and Electrical Consulting Engineers

3645 Keele St. #105
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Email: info@u-eng.ca
T: (416) 398-1999
F: (416) 398-1933

July 29, 2021

Attention: Executive Director, Engineering and Construction Services
c/o Manager, Development Engineering

Cc: General Manager, Toronto Water
c/o Manager, Environmental Monitoring Protection Unit
30 Dee Ave, Toronto ON M9N 1S9

Dear Sir or Madam,

This letter is to confirm that groundwater from the Private Water Drainage System will be collected and discharged into the sanitary control manhole of the Site located 320 McCowan Road, Scarborough.

The groundwater sump pumps will be sized at 94.8 L/min (groundwater peak flow rate) and are expected to run approximately 8.7 hours per day.

This peak flow rate will be used for assessing capacity for the peak discharge flow into the City's sanitary sewer system.

Once the proposed groundwater peak flow rate of 94.8 L/min is approved by Engineering Construction Services (ECS), City of Toronto at the Re-Zoning stage, the property owner will not be allowed to amend this flow rate in the future. Should there be any amendment to the peak flow rate of 94.8 L/min in future; the property owner shall re-submit either the updated pump schedule or a revised letter to ECS. In addition, the sewer capacity will need to be re-assessed.

Robert Hou

Name (printed)

Signature



Stamp



*Blauson Assets Management Ltd. c/o Ling Kee Group
320 McCowan Road*

Appendix D

counterpoint engineering

Predevelopment Flows

Project Name: 320 McCowan Road
 Project Number: 19066
 Drainage Area ID: 100

Composite RC Value	Area [ha]	RC	RC * Area
Landscaped Area	0.327	0.25	0.082
Conventional Roof and Paved	0.343	0.90	0.309
Total:	0.670		0.390
Divided by Total Area =			0.58

Rational Method - 2 Year Predevelopment

Event: **2** years

ABC's: A
 C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i = A/(T)^c mm/hr

Flow Q = CiA/360 Q m³/s
 L/s

Rational Method - 5 Year Predevelopment

Event: **5** years

ABC's: A
 C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i = A/(T)^c mm/hr

Flow Q = CiA/360 Q m³/s
 L/s

Rational Method - 10 Year Predevelopment

Event: **10** years

ABC's: A
 C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i = A/(T)^c mm/hr

Flow Q = CiA/360 Q m³/s
 L/s

Rational Method - 25 Year Predevelopment

Event: **25** years

ABC's: A
 C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i = A/(T)^c mm/hr

Flow Q = CiA/360 Q m³/s
 L/s

Rational Method - 50 Year Predevelopment

Event: **50** years

ABC's: A
 C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i = A/(T)^c mm/hr

Flow Q = CiA/360 Q m³/s
 L/s

Rational Method - 100 Year Predevelopment

Event: **100** years

ABC's: A
 C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i = A/(T)^c mm/hr

Flow Q = CiA/360 Q m³/s
 L/s

counterpoint engineering

Allowable Release Rate

Project Name: 320 McCowan Road
Project Number: 19066

Rational Method - 2 Year Predevelopment

Event: years

ABC's: A
C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i mm/hr
 $i=A/(T)^c$

Flow Q m³/s
Q=CiA/360 L/s

counterpoint engineering

Rational Method - Uncontrolled Areas (Post Development) To Servicing Easement

Project Name: 320 McCowan Road
Project No: 19066

Event: years

ABC's: A
C

Time of Concentration: t min

Runoff Coefficient: C

Site Area A ha

Intensity i mm/hr
 $i=A/(T)^c$

Flow Q m³/s
Q=CiA/360 L/s

counterpoint engineering

Project Name: 320 McCowan Road
 Project Number: 19066

Rainfall Data			
Location:	Toronto	a	59.7
Event	100 Year	b	0
		c	0.8

Allowable Release Rate: 82 L/s

Area ID	Area (ha)	Runoff Coefficient	t _c (min)	Storage Required (m ³)	Release Rate (L/s)	Description	Orifice Plate Size (mm)	Release Rate (L/s)
100	0.775	0.83	10	235	60.1	Orifice Plate	TBD	60.1
UNC	0.035	0.90	10		21.9	Uncontrolled		-
	0.810				82.0			

NOTES: On-site storage will be provided via an underground storage tank located within the building

AREA ID: 100

Composite RC Value	Area [ha]	RC	RC * Area
Landscape/Pervious	0.088	0.25	0.02
Green Roof	0.000	0.35	0.00
Building/Impervious Area	0.687	0.90	0.62

0.775 Total 0.64
 Divided by Total Area = 0.83

AREA ID: UNC

Composite RC Value	Area [ha]	RC	RC * Area
Landscape/Pervious	0.000	0.25	0.00
Building/Impervious Area	0.035	0.90	0.03

0.035 Total 0.03
 Divided by Total Area = 0.90

counterpoint engineering

**Modified Rational
Residential Tower**

Area: 100

Project Name: 320 McCowan Road
Project Number: 19066

Rainfall Data			
Location:	Toronto	a	59.700
Event	100 Year	b	0.000
		c	0.800

Site Data	
Area	0.775 ha
Runoff Coefficient	0.83
AC	0.64
Tc	10
Time Increment	10
Release Rate	60.1 L/s
Storage Required	235 m ³

Time	Rainfall Intensity	Storm Runoff	Runoff Volume	Released Volume	Storage Volume	
(min)	(mm/hr)	(m ³ /s)	(m ³)	(m ³)	(m ³)	
10	250	0.45	267	36	231	
20	144	0.26	307	72	235	*****
30	104	0.19	333	108	225	
40	83	0.15	353	144	209	
50	69	0.12	369	180	189	
60	60	0.11	383	216	166	
70	53	0.09	395	252	142	
80	47	0.08	405	288	117	
90	43	0.08	415	324	91	
100	40	0.07	424	360	63	
110	37	0.07	432	396	35	
120	34	0.06	439	432	7	
130	32	0.06	447	468	-22	
140	30	0.05	453	505	-51	
150	29	0.05	460	541	-81	
160	27	0.05	465	577	-111	
170	26	0.05	471	613	-141	
180	25	0.04	477	649	-172	
190	24	0.04	482	685	-203	
200	23	0.04	487	721	-234	
210	22	0.04	491	757	-265	
220	21	0.04	496	793	-297	
230	20	0.04	501	829	-328	

Counterpoint Engineering

Water Balance

Project Name: 320 McCowan Road

Project Number: 19066

City of Toronto's Green Standard Tier 1

Section QW 2.2

Initial Abstraction Asphalt, I	1	mm
Initial Abstraction Pervious, P	5	mm
Initial Abstraction Roof, R	1	mm
Toronto's small design rainfall event has 5mm excess rainfall		

Type of Area	Area	Units	% Redevelopment Area
Non-Green Roof Area	0.520	ha	64%
Asphalt	0.202	ha	25%
Pervious / Green Roof Area	0.088	ha	11%
Total Area	0.810	ha	100%

Initial Abstraction= Percent Impervious (Roof) *R + Percent Impervious (Asphalt)* I + Percent Pervious Green Roof * P

$$\text{Initial Abstraction} = 0.64 \times 1\text{mm} + 0.25 \times 1\text{mm} + 0.11 \times 5\text{mm}$$

Initial Abstraction (credit)= 1.43 mm

Required Development Retention = (Excess Rainfall- Initial Abstraction) * (Total Development Area)

$$\text{Required Development Retention} = (5\text{mm} - 1.4 \text{ mm}) \times (0.810 \text{ ha})$$

Required Development Retention (debit)= 28.9 m³

320 McCowan

Irrigation Water Requirement

<u>Planting Description</u>	<u>Area (m2)</u>	<u>Species Factor</u> (Ks)	<u>Density Factor</u> (Kd)	<u>Micriclimate Factor</u> (Kmc)	<u>Kl</u> (Kl=KsxKdxKmc)	<u>ETI (mm)/ Day</u> (Etl=ET0xKl)	<u>Water Req (m3)</u> per day	<u>Irrig. Eff (%)</u> MP Rotors	<u>Gross Water Req</u> (m3) per day
Ground Floor									
Shrub Planting	646.54	0.5	1.0	1.0	0.5	2.3	1.45	90	1.62
Deciduous Trees Large (25 x 28.3)	707.50	0.5	1.0	1.0	0.5	2.3	1.59	90	1.77
Deciduous Trees Small (19 x 7.1)	134.90	0.5	1.0	1.0	0.5	2.3	0.30	90	0.34
6th Floor Amenity Terrace									
Shrub Planting	310.77	0.5	1.0	1.3	0.65	2.9	0.91	90	1.01
Deciduous Trees Large (10 x 28.3)	283.00	0.5	1.0	1.4	0.7	3.2	0.89	90	0.99
Deciduous Trees Small (20 x 7.1)	142.00	0.5	1.0	1.4	0.7	3.2	0.45	90	0.50
Green Roofs									
6th Level Semi-Intensive Green Roof	1991.82	0.5	1.0	1.2	0.6	2.7	5.38	90	5.98
7th Level Semi-Intensive Green Roof	368.25	0.5	1.0	1.2	0.6	2.7	0.99	90	1.10

Total Water Requirement For July: 4584.78

13.30

ET0 is the evapotranspiration rate for peak period (Month of July in Toronto). This value is 138.2 mm for the month @ 4.5 mm/ day

Seasonal Water Requirement (M3)

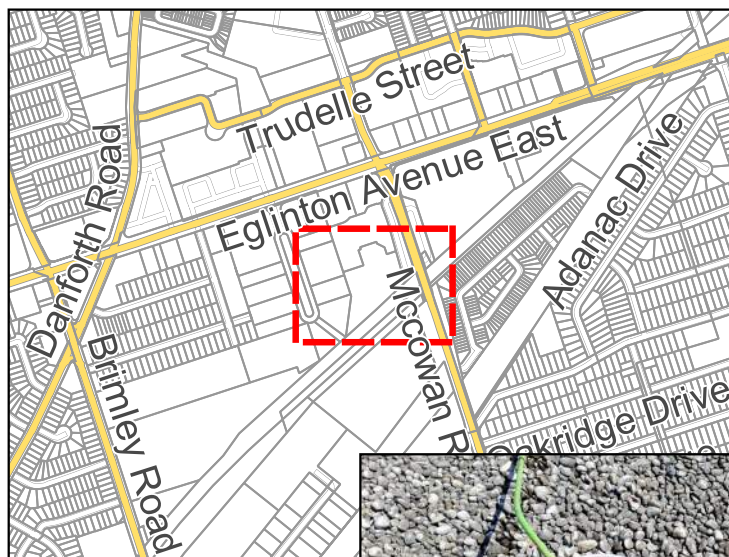
<u>Month</u>	<u>Evapotranspiration Factor</u>	<u>Water Req/Day (M3)</u>	<u>Water Req./72 Hours (M3)</u>
May	74%	9.84	29.53
June	90%	11.97	35.91
July	100%	13.30	39.90
August	80%	10.64	31.92
September	52%	6.92	20.75
October	40%	5.32	15.96

Seasonal Average/ 72 hours:

28.99

On-Site Dye Testing Results 320 McCowan Road, Scarborough

Study Area



I2S Ref #: 2023-6971
April 25, 2023

April 25, 2023

Counterpoint Engineering Inc.
8395 Jane Street, Suite 100
Vaughan, ON
L4K 5Y2



Att: Rasheed Serrao

On-Site Dye Testing Results

320 McCowan Road, Scarborough

Infrastructure Intelligence Services Inc. (I2S Inc.) is pleased to submit the attached results of on-site drainage inspections completed for the above noted property. These inspections were completed to determine the connectivity of stormwater features to the municipal sewers. A summary investigation map is presented in **Appendix A**. The findings are as follows:

- Seven (7) roof drains were dye tested atop of the parking garage and confirmed discharging to a control storm manhole (CTRL STM MH), which then continues to a municipal storm sewer by the northwest entrance further north towards Eglinton Avenue East.
- One (1) catch basin (CB03), found on the northwest side of the parking garage, was confirmed discharging to a storm control MH at the northwest entrance. This MH connects to the municipal storm sewer which runs north on the private road towards Eglinton Avenue East.
- Two (2) catch basins (CB01 & CB02) located on the east side of the property were confirmed discharging to the municipal storm sewer on Landmark Boulevard.

The videos and photographs from the investigation have been provided via download link.

Please feel free to contact the undersigned if you have questions/comments.

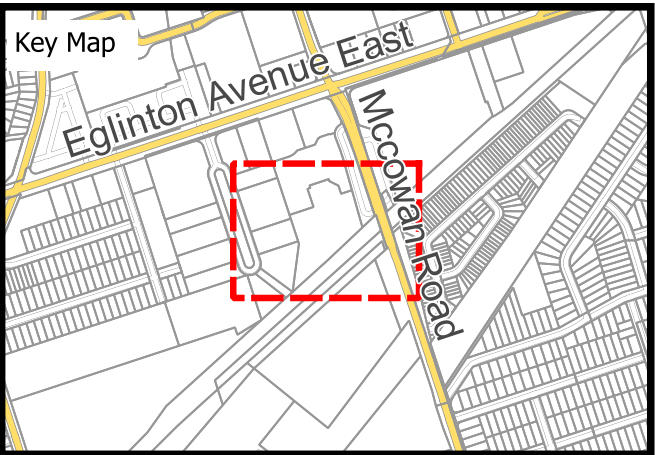
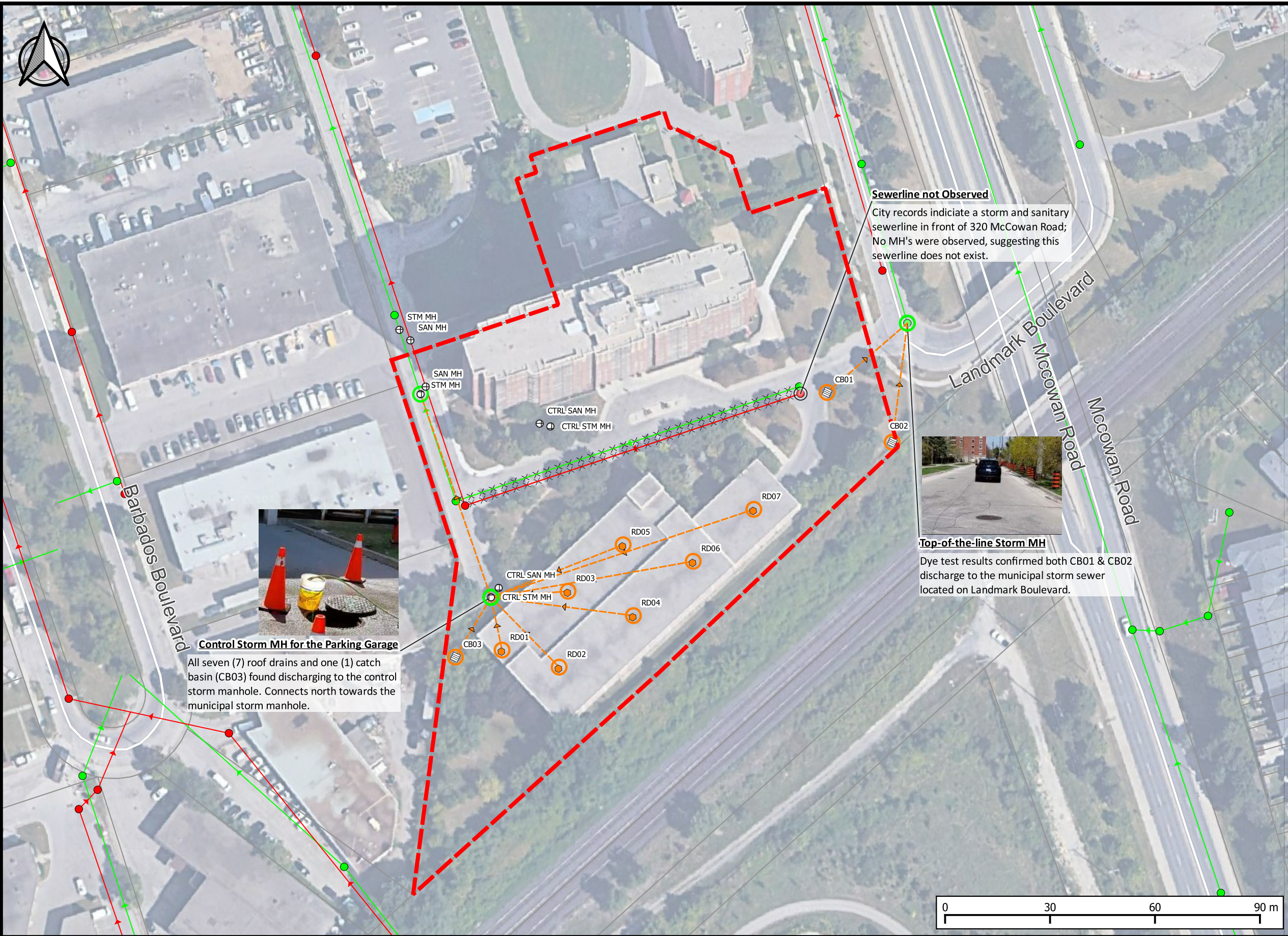
Sincerely,

Niall Quinlan
Manager, I&I Services
Tel.: 647-229-9175
Email: niall.quinlan@i2sinc.ca

Attached: On-Site Dye Results Map

Appendix A

Investigation Map

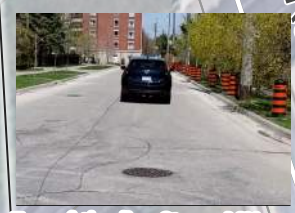


General Notes

Map Legend

- Extent
- Catch Basin
- Manhole
- Roof Drain
- Municipal Sewers**
- > Sanitary
- > Storm
- x-x-x Not Observed Sanitary
- x-x-x Not Observed Storm
- Dye Test Results**
- Dye Injection Point
- > Likely Dye Path
- Dye Observation Point

Sewerline not Observed
 City records indicate a storm and sanitary sewerline in front of 320 McCowan Road; No MH's were observed, suggesting this sewerline does not exist.



Top-of-the-line Storm MH
 Dye test results confirmed both CB01 & CB02 discharge to the municipal storm sewer located on Landmark Boulevard.



Control Storm MH for the Parking Garage
 All seven (7) roof drains and one (1) catch basin (CB03) found discharging to the control storm manhole. Connects north towards the municipal storm manhole.

Sources:

Projection System: NAD83 UTM ZONE 17N

Prepared By: Associated With:



Drawn: B. Lee Apr 25, 2023
Checked: M. Skerlan Apr 25, 2023
Approved: N. Quinlan Apr 25, 2023

Revision History

Project Location: 320 McCowan Road, Scarborough
Project # 2023-6971 - 320 McCowan Road

Appendix B

Digital Package of Files

Videos and Photos