Functional Servicing & Stormwater Management Report

Residential Development

958-960 East Avenue

Lakeview (Ward 1)

City of Mississauga

18 March 2020 *(Revision 0)*

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1.0 INTRODUCTION

Fabian Papa & Partners has been retained by RAW Design Studios, on behalf of the Peel Housing Corporation (PHC), to prepare this Functional Servicing & Stormwater Management Report in support of the Zoning By-law Amendment (ZBA) and Official Plan Amendment (OPA) applications from a municipal servicing perspective. This report discusses the provision of municipal services for the above captioned development proposal, including the stormwater management strategy.

Located at 958-960 East Avenue in Lakeview (Ward 1), City of Mississauga, the subject site is bounded by Lakeshore Road East to the north, East Avenue to the east, a recently constructed Peel Region Paramedic Station to the south, and both Byngmount Avenue and a residential subdivision to the west.

The subject site is approximately 7,602 m² (0.76 ha) in size, however, a re-alignment of the property line to the south is anticipated resulting in a net area of 7,578 m² (0.76 ha).

The subject site currently hosts two separate low-rise townhouse buildings, consisting of a total of 30 units. A vicinity map and aerial photograph of the subject site can be found in Appendix A.

The development proposal contemplates the construction of a 7-storey residential building with a total of 151 residential units and 1 underground level for parking, storage, and utility rooms. Architectural floor plans can be found in Appendix A for reference.

It should be noted that a significant portion of the subject site is encumbered by a multitude of easements reserved for Regional sanitary force mains and large diameter water transmission mains. The proposed building therefore will be limited to the unencumbered portions of the subject site, and servicing within the easement areas shall be minimized.

The City and Region's plan and profile drawings for the surrounding areas were obtained and reviewed as part of this assessment. Pertinent information has been incorporated into the Site Servicing & Grading Exhibits, and excerpt copies of the drawings can be found in Appendix B for reference. Furthermore, the Region of Peel's Public Works Department recently performed a servicing assessment in support of the proposed development. The report, "Water and Wastewater Servicing Analysis", henceforth referred to as the "Region's Servicing Assessment", in its entirety can be found in Appendix B for reference.

It should be noted that the City of Mississauga and the Region of Peel are currently in discussions to extend Byngmount Avenue to East Avenue. This has been taken into consideration as part of the servicing strategy and is discussed in subsequent sections.

2.0 WATER SUPPLY

Located just north of the Lakeview Water Treatment Plant, the existing municipal water infrastructure adjacent to the subject property is consists of a 300 mm diameter local distribution watermain, a 900 mm diameter transmission main, and a 1,500 mm diameter transmission main within East Avenue, a 300 mm diameter local watermain and a 600 mm diameter feedermain within Lakeshore Road East, and a 600 mm diameter feedermain (Zone 1) and a 2,100 mm diameter transmission main (Zone 2) within the easement areas. Existing municipal hydrants are located along the east side of East Avenue and the north side of Lakeshore Road East.

At the time of preparing this report, an independent hydrant flow test was not available, however, based on historical data and general knowledge of Pressure Zone 1 in this area, the following response curve has been assumed.

Assumed Flow-Pressure Response Curve

Flow (usgpm)	Flow (L/s)	**Pressure (psi)**	Pressure (kPa)
0	0	**86**	596
314	19.8	**85**	590
738	46.6	**84**	579
1297	81.8	**80**	552
1087	68.6	**82**[1]	563
5039	317.9[2]	**20**	138

Furthermore, the results of the Region's Servicing Assessment confirm that the existing 300 mm local watermain on East Avenue has sufficient capacity to support the increase in population density (assuming 150 units with an equivalent population of 405), and that there is sufficient capacity in the system under emergency conditions (i.e. fire). Please see the Region's "Water and Wastewater Servicing Analysis" which can be found in Appendix B.

2.1 Supply Demands

The domestic water demand for the subject site was calculated based on the Region of Peel demand criteria. The detailed demand calculations, which can be found in Appendix C, is summarized in the following table:

Domestic Water Supply Demands

Building Use	Units / Area	Population	Avg. Domestic Demand, ADD (L/s)	Peak Hour Demand, PHD (L/s)	Peak Day Demand, MDD (L/s)
Residential	151 units	334	1.1	3.3	2.2

The recommended fire flow demand has been calculated using the criteria outlined in the Water Supply for Public Fire Protection Manual, 1999, by the Fire Underwriters Survey (FUS). Appropriate reductions and increases have been applied to the calculations as follows:

Fire Underwriters Survey Coefficients

Construction Coefficient	0.6 (fire resistive)
Building Occupancy	-15% (limited-combustible)
Fire Suppression System	-30% (automatic sprinkler)
Exposure / Proximity	+10%

As the building is sprinkled, the floor area is calculated as follows:

Area (A) = Area of largest floor plus 25% of two adjoining floors

$A = 2{,}156\ m^2 + [(2{,}156\ m^2 + 2{,}156\ m^2) \times 0.25] = 3{,}234\ m^2$

[1] Interpolated residual pressure for design fire flow based on assumed response curve.

[2] Theoretical Flow predicted at 20 psi residual pressure and calculated per NFPA 291

The detailed fire flow calculations are as follows:

$F = 220 \times C \times A^{0.5} = 220 \times 0.6 \times (3{,}234\ m^2)^{0.5} = 7{,}507$ L/min

F = 8,000 L/min (rounded to nearest 1,000)

F1 = 8,000 L/min x 0.85 = 6,800 L/min

F2 = F1 × 0.50 = 6,800 L/min × 0.30 = 2,040 L/min

F3 = F1 × 0.10 = 6,800 L/min × 0.10 = 680 L/min

Fire Flow = F1 – F2 + F3 = 6,800 – 2,040 + 680 = 5,440 L/min

Fire Flow = 5,000 L/min (rounded to nearest 1,000)

Fire Flow = 83.3 L/s

The design flows applied in the design of the service connections to the proposed building are calculated as follows:

- Domestic Supply Line (PHD) = 3.3 L/s
- Total Fire Flow (Fire + MDD) = 83.3 L/s + 2.2 L/s = 85.5 L/s

Refer to Appendix C for the detailed calculations.

2.2 Water Service Connections

Based on the above demands, a new 150 mm diameter PVC service connection is proposed to be connected to the existing 300 mm diameter PVC watermain within East Avenue. The service connection will branch from the 150 mm diameter fire line to create a 100 mm diameter domestic supply line at the property per Peel standard drawing 1-6-4. The fire service detector check valve will be installed in the northeast corner of the property line per Peel standard drawing 1-3-1. The domestic service water meter and back-flow preventer will be installed within a mechanical room within the underground parking level.

The Ontario Building Code (clause 3.2.9.7.4) requires that any building above 84 m in height (measured from grade to the ceiling of the upper most occupied floor) be protected by two separate fire service connections separated by an isolation valve. Furthermore, NFPA 14 (clause 7.12.2) classifies buildings greater than 23 m as high-rise and requires a second siamese connection. As previously mentioned, the proposed building is ±27 m in height, therefore one fire service is sufficient, however, a second siamese connection will be required.

To service the siamese connections, a new private hydrant shall be located within 45 m of the proposed siamese connections and the proposed residential entrance to the building. Furthermore, the two existing municipal hydrants adjacent to the site satisfy the 90 m of coverage requirement to building faces with municipal frontage.

The location of the hydrants, service connections, and siamese connections are shown on the Site Servicing & Grading Exhibits which can be found in Appendix F.

2.3 Domestic and Fire Flow Analysis

The pressure at the building face for each connection is calculated as the residual pressure at the main less the head loss in the supply line. Based on the assumed static pressure and response curve at the existing main, and using the Hazen-Williams formula to determine the head losses in the lines, the resulting residual pressures are summarized in the following table (refer to Appendix C for the detailed calculations):

Hazen Williams Formula: $Q = 0.278 \times C \times D^{2.63} \times (H_f / L)^{0.54}$

Head Loss & Residual Pressure Summary Table

Service Connections	Flow (L/s)	Head Loss (psi)	Head Loss (kPa)	Residual Pressures	
				@ Main (psi/kPa)	@ Building (psi/kPa)
100 mm Domestic Line (PHD)	3.3	0.0	0.2	86.0/593	86.0/593
150 Fire Line (MDD+Fire)	85.5	4.9	33.6	79.5/548	74.9/514

As shown above, the residual pressures at the building face are expected to be well above the Region's minimum acceptable pressures of 40 psi (275 kPa) under both PHD conditions and Fire + MDD conditions. Furthermore, and as previously mentioned, the Region has recently performed a Servicing Assessment for the subject site in order to ensure that the existing municipal water system can accommodate the proposed increase in population. It was concluded that the existing water infrastructure can support the intensification.

3.0 SANITARY DRAINAGE

Local sanitary infrastructure consists of an existing 250 mm diameter sanitary sewer within Lakeshore Boulevard East, and an existing 250 mm diameter sanitary sewer within Byngmount Avenue. The subject site is currently serviced via an existing 250 mm diameter sanitary service which conveys flows from the existing buildings by gravity in a southwesterly direction and connects to the 250 mm diameter sanitary sewer within Byngmount Avenue. Sanitary flows are then conveyed in a westerly direction to Montbeck Crescent ultimately discharging to the Beach Street Wastewater Pumping Station. Flows are then pumped to the 1,650 mm diameter trunk sewer on Lakeshore Road East via parallel 450 mm and 500 mm diameter forcemains which are located within the aforementioned easements on the west side of the subject site.

3.1 Sanitary Design Criteria

Sanitary design flows for the subject property have been calculated using the Region's design criteria for sanitary sewers. The relevant criteria are summarized below.

- Design Flow: 302.8 Lpcd (residential)
- Peaking Factor: Calculated using the Harmon Formula
- Infiltration Flow: 0.2 L/s/ha
- Population Densities: Varies. See design sheets.

3.2 Foundation Drainage

Per the hydrogeological assessment by WSP Global Inc. (WSP), total suspended solids (TSS), Total Kjeldahl Nitrogen (TKN), total manganese (Mn), and total zinc (Zn) are in exceedance of the Region's limit for discharge to storm sewers whereas only total suspended solids (TSS) exceeds the Region's limit for discharge to sanitary sewers (per the Region of Peel's Sewer By-Law). It is therefore recommended that all dewatering activities be discharged to the sanitary sewer system with pre-treatment for TSS. Please see Appendix D for an excerpt copy of the hydrogeological assessment.

Short Term Groundwater Discharge (construction dewatering)

The anticipated temporary discharge has been calculated by WSP to be 101,000 L/day (1.17 L/s) if no caisson walls are utilized during construction, and 24,000 L/day (0.28 L/s) if caisson walls are chosen. As the method of construction shall be determined at a later date, the worst case construction method (no caisson walls) shall be considered. Furthermore, a dewatering pump will be sized at a later date. It is therefore assumed that a dewatering pump would operate for 6 hours per day, resulting in a maximum groundwater pumping rate of 4.7 L/s.

Long Term Groundwater Discharge (foundation drainage)

The anticipated long-term groundwater discharge has been calculated by WSP to be 23,700 L/day (0.27 L/s). At the time of this report, a groundwater pumping confirmation letter was not made available. It was therefore assumed that the groundwater pump would operate for 4 hours per day resulting in a maximum groundwater pumping rate of 1.6 L/s. The anticipated groundwater pumping rate has been incorporated into the post-development design sanitary flow which is discussed in the subsequent section.

3.3 Sanitary Design Flows

The pre-development flow for the subject site is calculated as follows:

$$Q_{\text{Pre-Dev.}} = \left(\frac{302.8\text{ Lpcd} \times 108\text{ pp} \times 4.21_{\text{P.F.}}}{86400\text{ s/day}}\right) + (0.20\text{ L/s/ha} \times 0.7588\text{ ha}) = 2.1\text{ L/s}$$

The post-development flow for the subject site is calculated as follows:

$$Q_{POST} = \left(\frac{302.8\text{ Lpcd} \times 334\text{ pp} \times 4.06_{\text{P.F.}}}{86400\text{ s/day}}\right) + (0.20\text{ L/s/ha} \times 0.7588\text{ ha}) + 1.6\text{ L/s}^{3} = 6.5\text{ L/s}$$

Based on the above, the increase in sanitary flow is calculated to be 4.4 L/s (6.5 L/s – 2.1 L/s). Please refer to Appendix D for the detailed sanitary design sheet.

[3] Anticipated long term groundwater pumping rate.

3.4 Receiving Sanitary Sewer Capacity

As the subject development results in an increase in dry weather flow; it is prudent to check if there are any impacts to the existing downstream sewer system. For the purposes of confirming capacity in the downstream receiving sanitary sewer, typically the "worst case" of following two conditions is considered:

- Short term condition (construction dewatering) = 4.7 L/s
- Long term condition (sanitary flows + foundation drainage) = 6.5 L/s

In this case, the long term condition governs.

As previously mentioned, the Region has prepared a Servicing Assessment for the subject property in order to ensure that the existing municipal sanitary system can accommodate the proposed development. For the purposes of the Servicing Assessment, the Region assumed a maximum 180 units (an increase of 150 units) with an equivalent post-development population of 486 people (an increase of 405 people) based on 2.7 people / unit.

The corresponding increase in flow per the Region's Servicing Assessment is calculated as follows:

$$Q_{REGION} = \left(\frac{302.8\text{ Lpcd} \times 486\text{ pp} \times 3.98_{\text{P.F.}}}{86400\text{ s/day}}\right) + (0.20\text{ L/s/ha} \times 0.7588\text{ ha}) = 6.9\text{ L/s}$$

As shown above, the post-development sanitary flow (including foundation drainage) is less than the flow considered in the Servicing Assessment therefore it is not expected that there will be any constraints to development. The Region, as part of this ZBA/OPA review, will confirm that there are no upgrades required to the receiving sewer system.

The Servicing Assessment noted that the Region's model for the sanitary system has potential for surcharging conditions. That said, the model shows that flows being discharged from the Lakeview Plant Backwash to the Montbeck Creek sewer were lower than expected. Therefore there is capacity for the proposed development.

3.5 Sanitary Service Connection

As previously mentioned, sanitary flows from the subject site are conveyed to the existing 250 mm diameter sanitary sewer within Byngmount Avenue via an existing 250 mm sanitary service. It is proposed however to abandon the existing 250 mm service, and extend the 300 mm sanitary sewer as a new municipal sewer for the potential extension of Byngmount Avenue.

Derived from City plan and profile drawings, the invert of the 250 mm diameter sanitary sewer at the existing manhole is approximately 77.62 m (3.0 m± below surface elevation).

It is proposed to install 84.0 m of 250 mm PVC sanitary sewer at a 1.0% slope, and place a new sanitary manhole. From the new manhole, 7.3 m of 200 mm PVC sanitary service shall be placed at a 2.0% slope resulting in an invert elevation of 78.72 m at the building's control manhole.

As shown below, the proposed service and sewer will easily convey the post-development sanitary flow, operating at only 10.5% (or less) of full flow capacity.

Sanitary Sewer and Service Performance Table

From	To	Pipe Size (mm)	Pipe Slope	Peak Flow (L/s)	Capacity (L/s)	Percent of Full Flow
Building	Prop.MH	200	2.0%	6.5	48.4	13.4%
Prop.MH	Ex.SAN.MH	250	1.0%	6.5	62.0	10.5%

A 1.2 m x 1.2 m cast-in-place control manhole shall be constructed within the underground footprint of the underground parking level. The control manhole lid shall be accessible 24 hours/day for inspection and monitoring, thus satisfying the Region of Peel's Wastewater Bylaw. The sewer invert at the control manhole is adequately deep to service the building by gravity, however, it is noted that the underground level of the building will require pumps to discharge to the sanitary service.

4.0 STORM DRAINAGE

4.1 Overview

Local exclusive storm infrastructure consists of an existing 525 mm diameter storm sewer within Lakeshore Boulevard East, a 450 mm diameter storm sewer within East Avenue, and an existing 375 mm storm sewer within Byngmount Avenue.

It should be noted that while a small portion of the subject site drains towards East Avenue, the majority of the site drains in a southwesterly direction towards an existing drainage swale, which in turn is conveyed to the 375 mm storm sewer within Byngmount Avenue. A copy of the City's drainage area map and associated storm sewer design sheet for the existing adjacent residential development confirms that the subject site was taken into account as part of the storm sewer design within Byngmount Avenue. Therefore, in order to maintain existing drainage patterns, storm flows from the subject site will be directed to Byngmount Avenue. A copy of the drainage area map and design sheet can be found in Appendix E for reference.

4.2 Design Criteria

The stormwater management servicing strategy for the subject development has been prepared in accordance with the City and Credit Valley Conservation Authority (CVC) design standards and criteria for the Cawthra Creek Sub-watershed (CCSW). The relevant criteria are summarized below:

<u>Water Quantity Management</u>

- The CCSW requires that the maximum allowable release rate from the site during all storm events, up to and including the 100-year event, must not exceed the flow equivalent to the peak runoff rate generated during a 2-year storm event.
- Discharge to a storm sewer shall not exceed capacity constraints.
- The pre-development runoff coefficient shall not exceed 0.50.
- An overland flow route shall be provided within the developed site to direct runoff in excess of the 100-year storm runoff to an approved overland flow outlet with a maximum of 300 mm ponding within paved areas.

<u>Water Quality Management</u>

- All runoff from the site shall achieve a long-term average removal of 80% of Total Suspended Solids (TSS) on an annual loading basis.

<u>Water Balance Management</u>

- The first 5mm of runoff shall be retained on-site and managed by way of infiltration, evapotranspiration or re-use. Low Impact Development (LID) practices should be explored for implementation.

4.3 Pre-Development Conditions

As previously mentioned, the allowable release rate shall be based on the 2-year pre-development release rates. The 2-Year return period design rainfall intensity is calculated as follows:

$$I_2 = \frac{610}{(T+4.6)^{0.78}} = \frac{610}{(15+4.6)^{0.78}} = 59.9 \text{ mm / hr}$$

The corresponding allowable peak flows from the subject site are calculated as follows:

$$Q_{\text{2-Year Pre-EAST}} = \frac{(A \times R) * I_2}{360} = \frac{(0.1540 \text{ ha} \times 0.50) \times 59.9 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 12.8 \text{ L/s}$$

$$Q_{\text{2-Year Pre-WEST}} = \frac{(A \times R) * I_2}{360} = \frac{(0.6038 \text{ ha} \times 0.35) \times 59.9 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 35.3 \text{ L/s}$$

The allowable site discharge from the subject site to the sewer system within East Avenue shall be limited to **12.8 L/s**, and the allowable site discharge from the subject site to the sewer system within Byngmount Avenue shall be limited to **35.3 L/s**. Please refer to the detailed storm sewer design sheet which can be found in Appendix E.

It is noted that the storm drainage plan and corresponding storm design sheet (both sourced from the City) demonstrate that the receiving storm sewer within Byngmount Avenue was sized to accommodate storm flows from and area of 0.67 ha with a runoff coefficient of 0.60. As such the allowable discharge from the subject site to the Byngmount Avenue storm sewer is calculated as follows:

$$Q_{\text{2-Year Pre-WEST}} = \frac{(A \times R) * I_2}{360} = \frac{(0.6038 \text{ ha} \times 0.60) \times 59.9 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 60.3 \text{ L/s}$$

As this exceeds the 2-year pre-development allowable discharge rate, the 2-year pre-development rate shall govern.

As shown on the pre-development storm drainage plan, there is an area north of the subject site which is conveyed through the property via an existing 400 mm storm culvert which daylights to a swale along the future Byngmount Avenue extension to East Avenue. It is proposed that this external area continue to be conveyed via the existing culvert, therefore it is not considered in the stormwater management calculation for this development.

4.4 Water Quantity Management

The post-development hydrologic conditions for the site were established using the City's design standards, which include the 2-year and 100-year IDF data, a time of concentration of 15 minutes, and the following storm drainage run-off coefficients:

	Runoff Coefficients
Bare Roof	0.90
Green Roof	0.45
Landscaped Areas	0.25
Permeable Pavers	0.55
Hard Surfaces	0.90

These design parameters are used in the subsequent sections to determine the on-site storage requirements to meet the target release rate for the site.

The 100-Year return period design rainfall intensity is calculated per City standards as follows:

$$I_{100} = \frac{1450}{(T+4.9)^{0.78}} = \frac{1450}{(15+4.9)^{0.78}} = 140.7 \text{ mm / hr}$$

Due to grading constraints, a portion of the subject site will continue to be conveyed to East Avenue under post-development conditions. The corresponding 100-Year post-development peak un-attenuated flows from the site are calculated as follows:

$$Q_{100\text{-EAST}} = \frac{(A \times R) \times I_{100}}{360} = \frac{(0.0176 \text{ ha} \times 0.50) \times 140.7 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) \times 1.25_{\text{Adjust}} = 4.3 \text{ L/s}$$

$$Q_{100\text{-WEST}} = \frac{(A \times R) \times I_{100}}{360} = \frac{(0.7402 \text{ ha} \times 0.65) \times 140.7 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) \times 1.25_{\text{Adjust}} = 234.4 \text{ L/s}$$

As shown above, the 100-year post-development design flow to East Avenue (4.3 L/s) is less than the allowable 2-year pre-development flow (12.8 L/s), therefore no further action is required.

The 100-year post-development design flow to Byngmount Avenue (234.4 L/s) is significantly larger than the allowable 2-year pre-development flow (35.3 L/s), therefore on-site storage will be required.

To attenuate flows, typically a combination of roof top, surface and/or underground storage can used to achieve the required volumes. In this case, all methods shall be implemented in order to contain flows generated from the subject site, up to the 100-Year level.

It should be noted that due to grading constraints, a portion of the site along the western property will be allowed to be discharged to the Byngmount Avenue without attenuation. This will act to effectively lower the allowable discharge from the stormwater management tank. All other areas shall be directed via gravity to the underground stormwater tank system, and be released at a controlled rate at or below the calculated allowable discharge, and then be discharged by gravity to the municipal sewer via the proposed storm service connection.

The un-controlled discharge is calculated as follows:

$$Q_{\text{Un-Controlled}} = \frac{(A \times R) \times I_{100}}{360} = \frac{(0.00752 \text{ ha} \times 0.25) \times 140.7 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) \times 1.25_{\text{Adjust}} = 9.2 \text{ L/s}$$

The corresponding allowable stormwater management release rate is therefore **26.1 L/s** (35.3 L/s – 9.2 L/s).

4.4.1 Rooftop Storage

Pre-manufactured rooftop drainage hoppers are proposed to be installed on various roof levels to help control the storm runoff. All roof control drains are proposed to be 'Control-Flo' (Product No. ZCF-121) manufactured by Zurn Industries Limited (or approved equal), with a discharge release rate of 5 gal/min/inch depth of storage (1 notch design).

The following calculations and a copy of the detailed specification from Zurn Industries Limited can be found in Appendix E for reference.

Using the above noted City's rainfall intensity for the 100-year storm, the total required storage and resulting roof controlled discharge rates at each roof level are summarized as follows:

Roof Area

- Roof Area: 2,209 m^2
- 100-Yr Storage required: 107.6 m^3
- 100-Yr Storage depth: 54 mm
- No. of Drains 5
- Roof Control drain release rate: 2.7 L/s

$$Q_{100\text{-Roof Drains}} = \frac{(5 \text{ gal/min} \times 3.785 \text{ L/gal})}{60 \text{ sec/min}} \times \frac{50 \text{ mm}}{25.4 \text{ mm/inch}} \times 4 \text{ Drains} = 2.7 \text{ L/s}$$

The total rooftop controlled release rate from all roof areas is calculated to be 2.7 L/s which will be directed to (and further attenuated by) the stormwater management tank.

Rooftop scuppers should be installed at a height of at least 100 mm, however, it is noted that there is some flexibility in this specification should conditions warrant. All controlled roof drainage will be directed to the proposed stormwater tank in the P1 level of the building (to be discussed in subsequent sections).

4.4.2 Surface Storage (Parking Area)

A single catch basin is proposed within the parking area which is encumbered by easements. The catch basin shall be connected to the stormwater management tank via an orifice control on the catchbasin lead. The orifice will serve to attenuate storm flows within the parking lot and in turn, reduce the size of the stormwater management tank. Through an iterative process, it was determined that the 100-Year storage depth in the parking lot is 220 mm resulting in a driving head for the orifice flow calculation of 1.42 m.

Utilizing a 100 mm diameter orifice tube, the orifice discharge is calculated as follows:

$$Q_{\text{Orifice (Parking Area)}} = (0.80) \times \frac{\pi \times (0.0100)^2}{4} * \sqrt{2 \times 9.81 \times (1.42\text{-}0.0100/2)} \times \left(\frac{1000 \text{ L}}{m^3}\right) = 33.2 \text{ L / s}$$

The orifice discharge rate and corresponding storage requirements are summarized as follows:

100-Year Surface Storage Requirements

Orifice Release Rate	33.2 L/s
Storage Required (see Appendix E)	48.2 m^3
Storage Provided (above orifice to emergency overflow)	49.6 m^3

4.4.3 Underground Storage

To further attenuate flows from the site, an underground stormwater storage tank, complete with an orifice plate, is proposed in the southwest corner of the underground P1 level. All storm run-off generated on the site will be collected and directed to the proposed tank. Utilizing a 98 mm diameter orifice plate and with the 100-Year storage depth in the system set at 1.67 m, the orifice discharge is calculated as follows:

$$Q_{\text{Orifice (SWM Tank)}} = (0.60) \times \frac{\pi \times (0.0099)^2}{4} * \sqrt{2 \times 9.81 \times (1.67\text{-}0.0099/2)} \times \left(\frac{1000\ \text{L}}{\text{m}^3}\right) = 25.9\ \text{L/s}$$

The orifice discharge rate and corresponding storage requirements are summarized as follows:

100-Year SWM Tank Storage Requirements

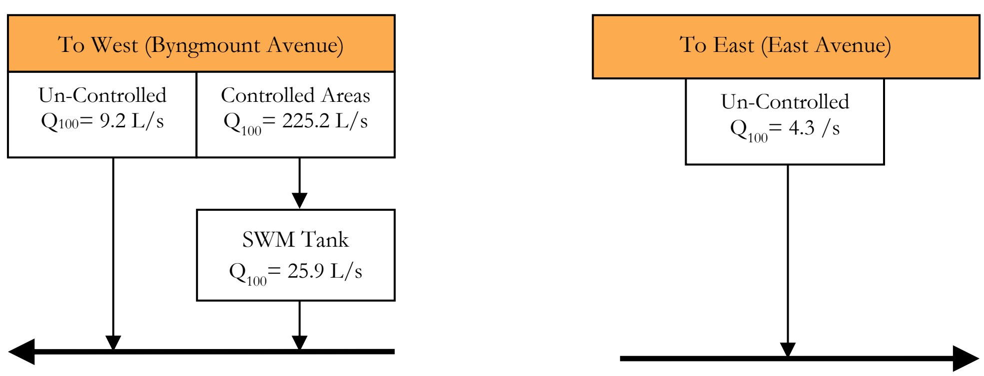
Allowable SWM Tank Release Rate (to Byngmount Avenue)	26.1 L/s
Actual Orifice Release Rate (to Byngmount Avenue)	25.9 L/s
Tank Storage Required (see Appendix E)	127.6 m^3
Tank Storage Provided (above orifice to emergency overflow)	130.9 m^3

It is proposed that a cast-in-place reinforced concrete stormwater storage tank, with a minimum inside area of 77.0 m^2, be installed in the P1 level to contain the storage volume.

4.4.4 Overall Quantity Control

A schematic representation of the stormwater management system is as follows:

Overall Stormwater Management Plan



Q Allowable ≥ Q Actual
35.3 L/s ≥ 35.1 L/s (9.2 L/s + 25.9 L/s)

Q Allowable ≥ Q Actual
12.8 L/s ≥ 4.3 L/s

It is important to note that regular maintenance inspections of the tank system, orifices, and outlet pipe should be conducted to ensure that there are no blockages or other conditions which would prevent the proper functioning of this design element. The recommended minimum frequency of such inspections is annually.

By providing on-site storage via an underground stormwater storage system with a controlled discharge release rate to the receiving sewer, the subject site satisfies the City's stormwater management objectives.

4.5 Water Quality Management

Pursuant to the City's design criteria, stormwater quality controls are required to be implemented on-site to achieve Enhanced Level 1 Protection (i.e. 80% TSS removal). The subject site will be largely comprised of "clean" areas such as roof, green roof, landscaping, and permeable pavers. All "dirty" areas shall first be directed to permeable paves prior to entering the sub surface drainage collection system. The following chart summarizes the subject site's inferred TSS removal rate:

Water Quality Summary

Site Area	Area (m^2)	% of total	TSS Removal Rate	Overall
Bare Roof	1,398	18.4%	95%	17.5%
Green Roof	811	10.7%	100%	10.7%
Landscape	1,797	23.7%	95%	23.7%
Permeable Pavers	1,147	15.1%	80%	12.1%
Hard Surfaces	2,425	32.0%	80%[4]	25.6%
Total	7,578	100%		**89.7%**

Due to the high percentage of roof, green roof, landscaping, and permeable pavers, the requirements for quality control (i.e. minimum 80% TSS removal) have been satisfied.

4.6 Water Balance Management

In order to promote preservation of the site's natural hydrological water balance, the City and CVC require a minimum volume of 5 mm over the total site area be retained. The total water balance volume required to be detained is calculated as follows:

- Volume Required = $A_{SITE} \times 5$ mm
- Volume Required = 7,826 $m^2 \times 5$ mm /(1000 mm/m) = 37.9 m^3

Based on initial abstraction rates for the site surfaces, the total abstraction is calculated as follows:

Initial Abstraction Table

Site Area	Area (m^2)	% of total	Initial Ab. for Each Area	Total Initial Ab. (m^3)
Bare Roof	1,398	18.4%	1 mm	0.2 m^3
Green Roof	811	10.7%	5 mm	0.5 m^3
Landscape	1,797	23.7%	5 mm	1.2 m^3
Permeable Pavers	1,147	15.1%	5 mm	0.8 m^3
Hard Surfaces	2,425	32.0%	1 mm	0.3 m^3
Total	7,578	100%		**3.0 m^3**

After a thorough review of the Low Impact Development Stormwater Management Planning and Design Guide (hereinafter referred to as the "LID Guide"), the most feasible methods of meeting the required water balance objectives are through a combination of rainwater harvesting and groundwater infiltration. It is therefore proposed that stormwater be captured and stored within an infiltration gallery for groundwater infiltration. Furthermore, water will be captured and stored within a sump located in the stormwater management tank and will be used for irrigation purposes.

[4] Areas to first be conveyed over areas containing permeable pavers to provide water quality control.

The following is a summary of the water balance achieved:

Water Balance Summary

Water Balance Uses	Volumes
Initial Abstraction	3.0 m^3
Infiltration Gallery	16.2 m^3
Landscape Irrigation	23.1 m^3
Total	**42.3 m^3**

By retaining the required 5 mm volume on-site, the City's requirements with respect to water balance have been satisfied. The detailed design sheets can be found in Appendix E. Additional details pertaining to the infiltration gallery and irrigation system will be provided at the Site Plan Application stage.

4.7 Future Byngmount Avenue Extension

As previously mentioned, storm flows from a majority of the subject site are conveyed in a southwesterly direction towards an existing drainage swale, which in turn is conveyed to the 375 mm storm sewer within Byngmount Avenue. The City's storm sewer design sheet for the existing residential development to the west confirms that a portion of the subject site was taken into account for the storm sewers within Byngmount Avenue.

In order to service the subject site, it is proposed to extend the 375 mm storm sewer along the future extension of Byngmount Avenue. In addition to conveying controlled flows from the subject site, the 375 mm storm sewer should be sized to convey flows from the external area to the north, the uncontrolled area and the future roadway itself. The anticipated 10-year design flow is calculated as follows:

$$Q_{10\text{-YR}} = \frac{(A \times R) \times I_{10}}{360} = \frac{(0.4862 \text{ ha} \times 0.52) \times 99.2 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{m^3}\right) \times 1.0_{\text{Adjust}} = 87.2 \text{ L/s}$$

The 10-year controlled orifice discharge from the SWM tank is calculates as follows:

$$Q_{\text{Orifice (SWM Tank)}} = (0.60) \times \frac{\pi \times (0.0099)^2}{4} * \sqrt{2 \times 9.81 \times (1.351\text{-}0.0099/2)} \times \left(\frac{1000 \text{ L}}{m^3}\right) = 23.8 \text{ L/s}$$

The anticipated 10-year flow within the proposed 375 mm PVC storm sewer is 111.0 L/s (87.2 L/s + 23.8 L/s).

As derived from the City's plan and profile drawings, the invert of the sewer at the tie-in location is approximately 79.00 m. It is proposed that 74.8 m of new 375 mm PVC storm sewer be installed at 0.65% gradient, and a new manhole be installed within the future right-of-way. As shown below, the proposed service and storm sewer can easily convey the attenuated post-development storm flow operating at 79% of full flow capacity.

Storm Service Performance Table

From	To	Pipe Size (mm)	Pipe Slope	Peak Flow (L/s)	Capacity (L/s)	Percent of Full Flow
Prop.MH1	Ex.MH	375	0.65%	111.0	141.4	79%

Please refer to Appendix E for the detailed design calculations.

4.8 Storm Service Connection

From the new manhole, 14.6 m of new 250 mm PVC storm service shall be installed at a 1.0% gradient resulting in an invert elevation of 79.70 m at the storm control manhole. As shown below, the proposed service can easily convey the attenuated post-development storm flow operating at 44% of full flow capacity.

Storm Service Performance Table

From	To	Pipe Size (mm)	Pipe Slope	Peak Flow (L/s)	Capacity (L/s)	Percent of Full Flow
Control MH	Prop.MH1	250	1.0%	25.9	59.5	44%

Please refer to Appendix E for the detailed design calculations.

4.9 Emergency Overflow

All building roof areas shall be provided with rooftop scuppers which will ensure a safe emergency overflow should the rooftop drains become blocked or clogged. The areas surrounding the perimeter of the buildings have been designed with positive drainage (away from building).

To provide a relief point within the underground system, the stormwater management tank shall be fitted with an emergency overflow catchbasin (open grate) lid, located within the subject site at street-line at the east side of the property. Should the property experience a storm greater than the 100-year event or should the orifice become clogged, surplus water will overflow through the catchbasin grate (elevation is 82.10 m), and spill to the south and eventually to the Byngmount Avenue right-of-way.

It is recommended that all other incoming pipes to the tank be fitted with one-way flap gate valves (i.e. backflow preventers) to prevent surcharging in the building's plumbing system. The location and details pertaining to the safety overflow are shown on the Site Servicing & Grading Exhibits.

4.10 Sediment and Erosion Control

In accordance with the Erosion and Sediment Control (E&SC) Guidelines for Urban Construction, temporary erosion and sediment control measures are required for any development application.

To mitigate potential environmental impacts from sediment pollution, it is proposed that a sediment control fence be installed along the entire perimeter of the site. Any existing and proposed catch basins shall be protected with a Terrafix 360R geotextile fabric (or approved equal), and a mud mat shall be installed in the northwest corner to prevent any mud tracking onto the municipal roads.

4.11 Low-Impact Development (LID) Alternatives

To limit the impact of the development to the natural environment several LID measures are proposed for this development. As noted in section 4.5, water quality targets have been achieved through a combination of green roof, landscaping, and permeable pavers.

Furthermore as noted in section 4.6, water balance targets for this development have been achieved through the implementation of an infiltration gallery to promote groundwater infiltration, and by storing stormwater in the sump of the stormwater management tank which will be used for landscape irrigation.

5.0 CONCLUSIONS

This letter report illustrates that the proposed development is feasible from municipal servicing and stormwater management perspectives.

Proposed fire and domestic water demands are within acceptable ranges and can be accommodated by the existing municipal water supply network as confirmed by the Region of Peel.

The receiving sanitary sewer network within Byngmount Avenue can accommodate the subject site without improvements as confirmed by the Region of Peel.

The proposed internal storm sewer network, on-site storage, and the controlled discharge release rate are at appropriate levels and therefore the subject site satisfies the City's stormwater management objectives.

We trust that this satisfies your current needs. Should you have any questions, or require additional information, please do not hesitate to contact the undersigned.

Respectfully Submitted,

fabian papa & partners
A Division of FP&P HydraTek Inc.

LICENSED PROFESSIONAL ENGINEER
J. M. JENKINS
2020-03-18
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R. D. FILIPUZZI
2020-03-18
PROVINCE OF ONTARIO

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PEO Designated Consulting Engineer
Partner

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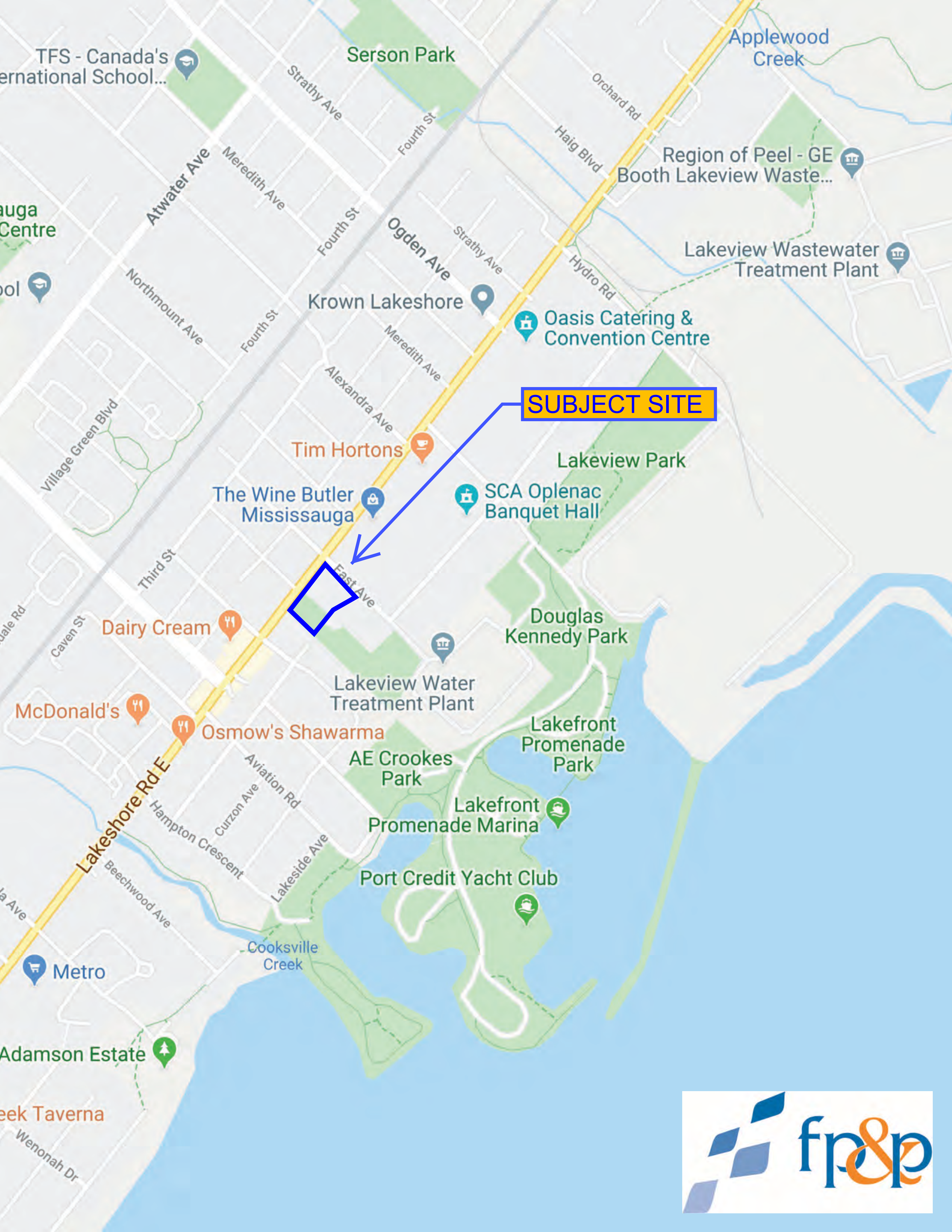
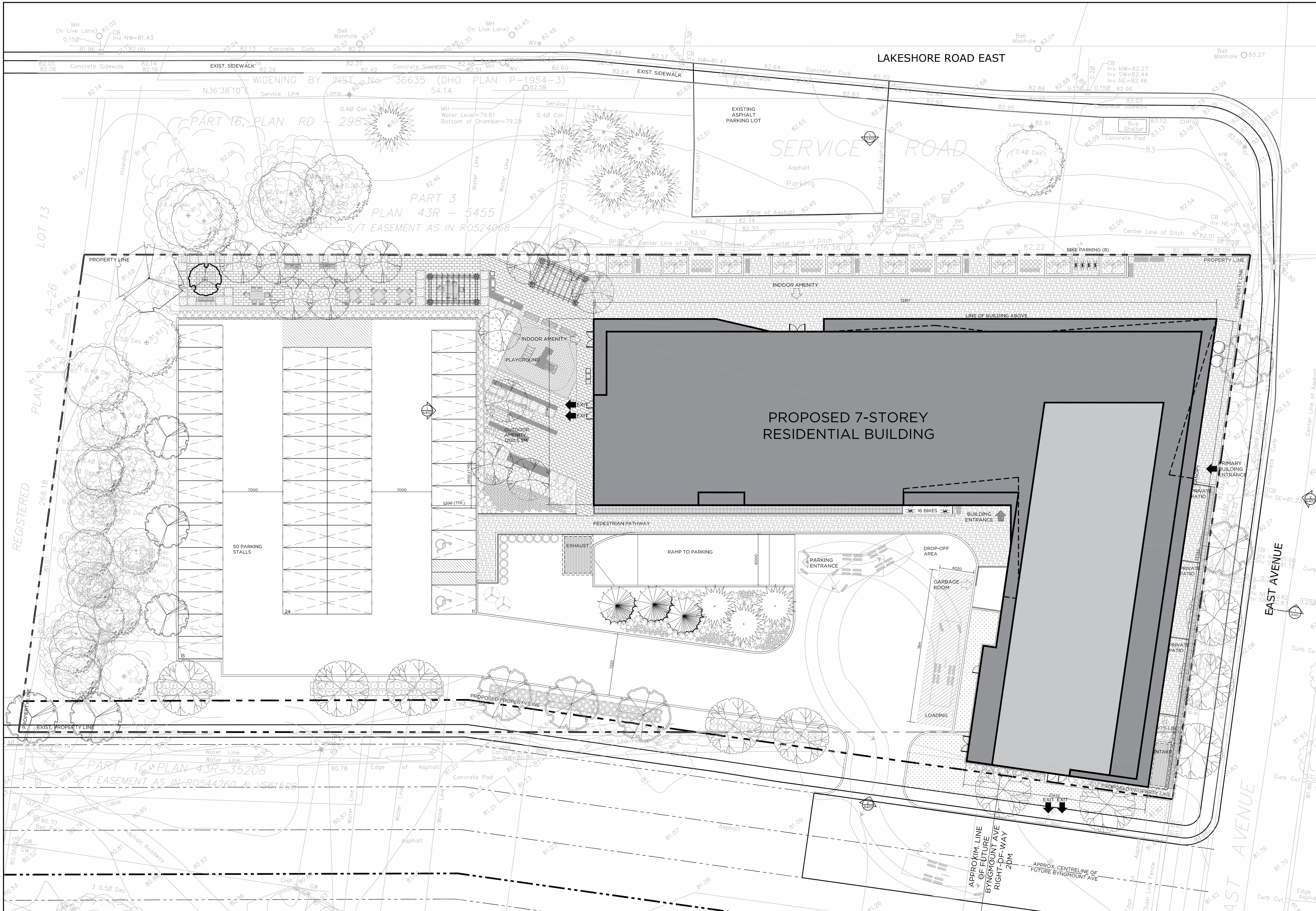
E-Mail: rfilipuzzi@fabianpapa.com

h:\fp&p hydratek\projects\2019\19002 - 958-960 east avenue, mississauga\report\re-zoning\revision 0\19002 - fs and swm report (revision 0).docx

APPENDIX A


SUBJECT SITE
TFS - Canada's
ernational School...
Serson Park
Applewood
Creek
Strathy Ave
Orchard Rd
Fourth St
Haig Blvd
Region of Peel - GE
Booth Lakeview Waste...
Atwater Ave
Meredith Ave
auga
Centre
Fourth St
Ogden Ave
Strathy Ave
Lakeview Wastewater
Treatment Plant
Hydro Rd
ool
Northmount Ave
Fourth St
Krown Lakeshore
Oasis Catering &
Convention Centre
Meredith Ave
Alexandra Ave
Village Green Blvd
Tim Hortons
Lakeview Park
The Wine Butler
Mississauga
SCA Oplenac
Banquet Hall
Third St
East Ave
Douglas
Kennedy Park
Caven St
Dairy Cream
Lakeview Water
Treatment Plant
McDonald's
Osmow's Shawarma
Lakefront
Promenade
Park
Lakeshore Rd E
Aviation Rd
AE Crookes
Park
Curzon Ave
Hampton Crescent
Lakefront
Promenade Marina
Lakeside Ave
Beechwood Ave
Port Credit Yacht Club
Cooksville
Creek
Metro
Adamson Estate
eek Taverna
Wenonah Dr


West Ave
Lakeshore Rd E
SUBJECT SITE
Land Conveyance to Region
Land Conveyance to Peel Housing Corporation
Montbeck Crescent
East Ave
fp&p



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ISSUED RECORD

2019.08.26 - ISSUED FOR DARC

REVISION RECORD

NORTH

RAW

405-317 ADELAIDE ST. W.
TORONTO CANADA M5V 1P9
+1 416 599 9729
WWW.RAWDESIGN.CA

19006

958-960 East Avenue
Mississauga

SITE
PLAN

1:400

A100

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ISSUED RECORD

2019.08.26 - ISSUED FOR DARC

REVISION RECORD

NORTH

RAW

405-317 ADELAIDE ST. W.
TORONTO CANADA M5V 1P9
+1 416 599 9729
WWW.RAWDESIGN.CA

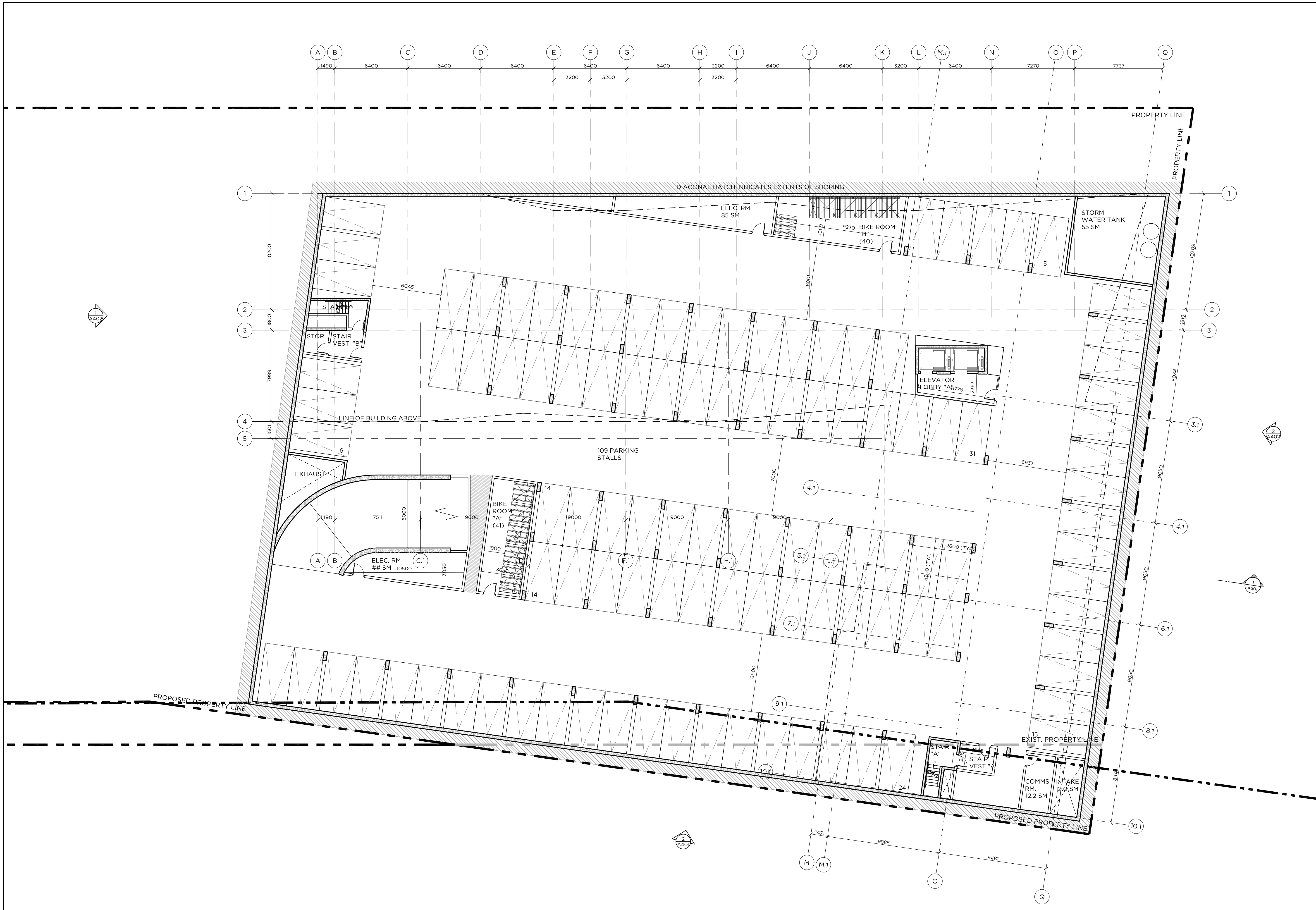
19006

958-960 East Avenue
Mississauga

GROUND FLOOR PLAN

1:300

A201



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ISSUED RECORD

2019.08.26 - ISSUED FOR DARC

REVISION RECORD

NORTH

RAW

405-317 ADELAIDE ST. W.
TORONTO CANADA M5V 1P9
+1 416 599 9729
WWW.RAWDESIGN.CA

19006

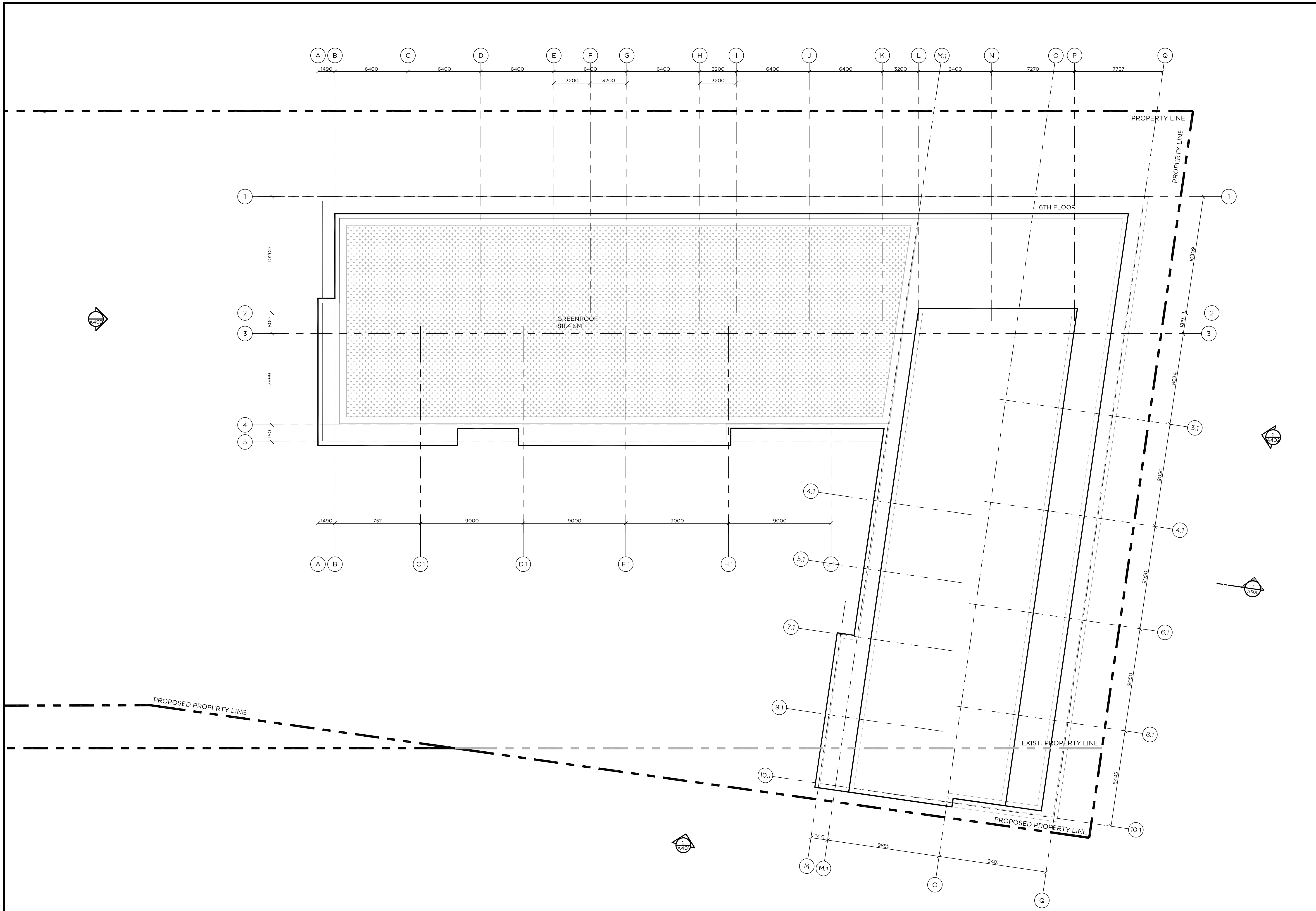
958-960 East Avenue
Mississauga

PARKING
PLAN
P1

1:300

A101

O:\19006 958 East Avenue Mississauga\Drawings\Current\19006_A100-A207_Rev1.dwg / Tuesday, February 11, 2020 11:29:44 AM



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ISSUED RECORD

2019.08.26 - ISSUED FOR DARC

REVISION RECORD

NORTH

405-317 ADELAIDE ST. W.
TORONTO CANADA M5V 1P9
+1 416 599 9729
WWW.RAWDESIGN.CA

19006

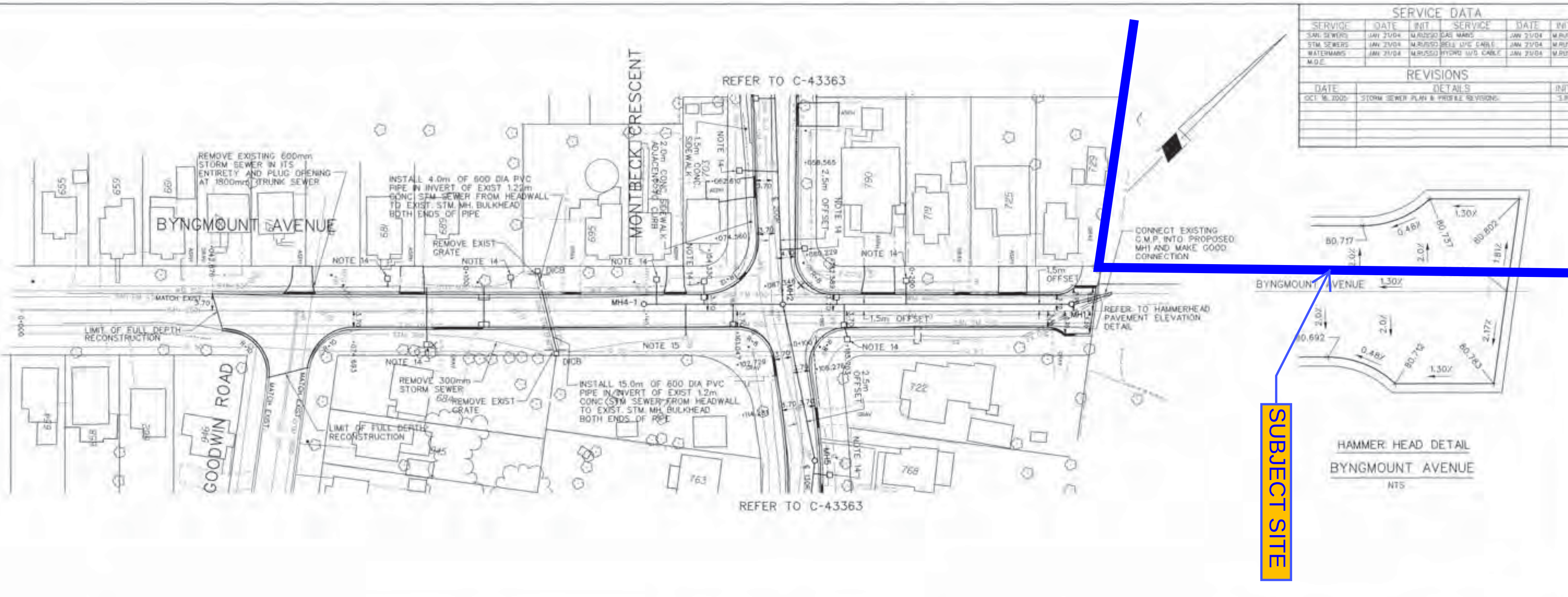
958-960 East Avenue
Mississauga

ROOF
PLAN

1:300

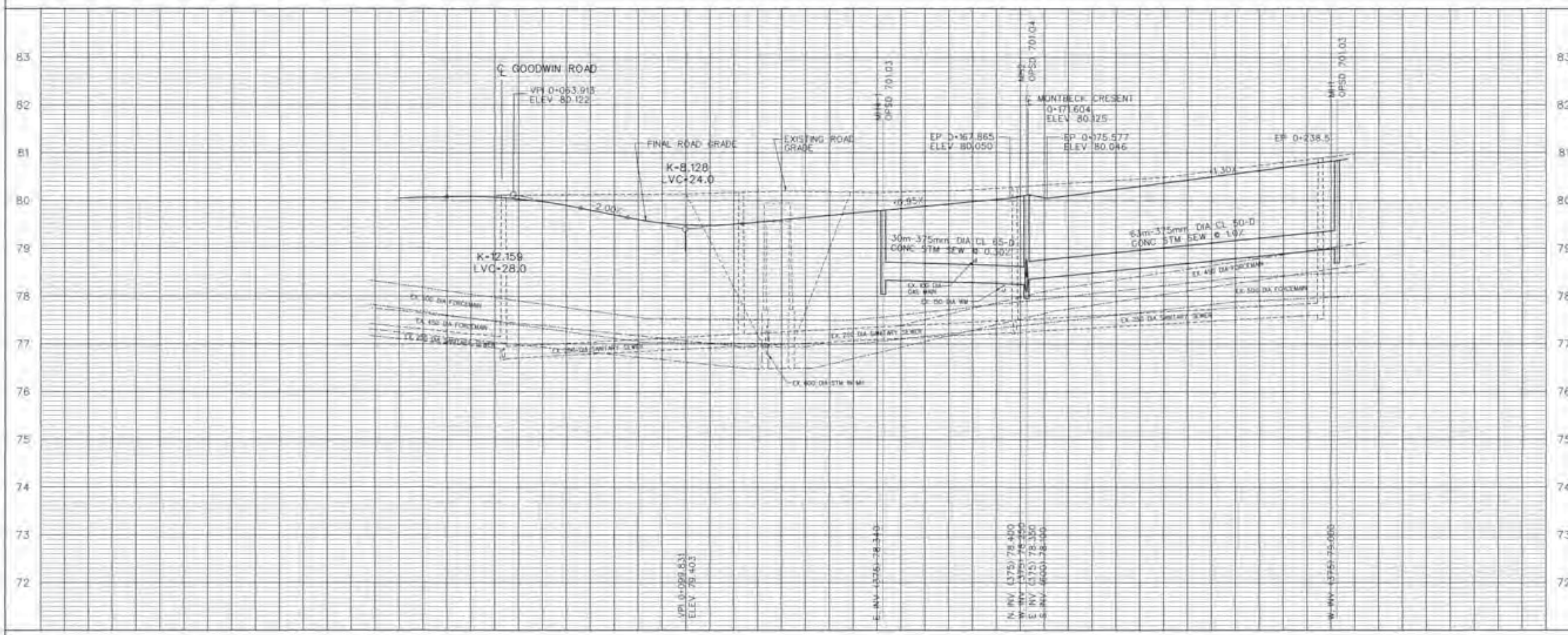
A209

APPENDIX B



SERVICE DATA					
SERVICE	DATE	INIT	SERVICE	DATE	INIT
SAN. SEWERS	JAN 21/04	M.RUSSO	GAS MAINS	JAN 21/04	M.RUSSO
STM. SEWERS	JAN 21/04	M.RUSSO	BELL U/G CABLE	JAN 21/04	M.RUSSO
WATERMAINS	JAN 21/04	M.RUSSO	HYDRO U/G CABLE	JAN 21/04	M.RUSSO
M.O.E.					

REVISIONS		
DATE	DETAILS	INIT.
OCT 18, 2005	STORM SEWER PLAN & PROFILE REVISIONS	S.R.



FINAL RD. GR.	80.050	80.068	80.800	79.491	79.595	79.785	79.975	80.103	80.363	80.623	80.883
EXIST.RD.GR.	80.050	80.111	80.127	80.140	80.198	80.168	80.219	80.363	80.484	80.735	80.978
CHAINAGE	0+040	0+060	0+080	0+100	0+120	0+140	0+160	0+180	0+200	0+220	0+240

GENERAL NOTES

1. ALL DRIVEWAYS ASPHALT UNLESS OTHERWISE NOTED
2. ALL SERVICE LOCATIONS ARE APPROXIMATE AND MUST BE LOCATED ACCURATELY IN FIELD
3. ALL MEASUREMENTS FOR THIS PROJECT ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE INDICATED
4. ALL SINGLE C.B. LATERALS TO BE 250mm Ø CL 65-D ALL OTHERS TO BE 300mm Ø CL 65-D
5. ALL TREES AND SHRUBS NOT MARKED IN THE FIELD FOR REMOVAL, TO BE PROTECTED
6. ALL SEWERS AND LATERALS TO HAVE CLASS 'B' BEDDING AND SAND COVER BACKFILL UNLESS OTHERWISE INDICATED
7. ALL UTILITY RELOCATION BY OTHERS
8. "X" - DENOTES REMOVALS
9. DURING SEWER CONSTRUCTION, WATERMAINS TO BE PROTECTED TO THE SATISFACTION OF THE PUBLIC WORKS DEPARTMENT OF THE REGIONAL MUNICIPALITY OF PEEL
10. COLD GRINDS TO BE MAX. 50mm IN DEPTH UNLESS OTHERWISE NOTED
11. INFORMATION SHOWN HEREON IS FOR USE BY THE CITY OF MISSISSAUGA, TRANSPORTATION AND WORKS DEPARTMENT, AND IS NOT INTENDED FOR USE BY ANY OTHER PARTIES UNLESS EXPRESSED WRITTEN CONSENT IS OBTAINED. MEASUREMENTS SHOWN MUST BE CONFIRMED BY FIELD SURVEY BEFORE USE.
12. PROTECTED BY COPYRIGHT, MAY NOT BE REPRODUCED WITHOUT PERMISSION
13. THIS IS NOT A PLAN OF SURVEY
14. PUF CB COM STD 2114.01 WITH FISH SYMBOL GRATE
15. LEAVE EXISTING CATCHBASIN IN PLACE/CONNECT TO NEW SEWER AND REPLACE FRAME AND GRATE WITH FISH SYMBOL GRATE

DESIGN BY
BORIS LENCE C.E.T.

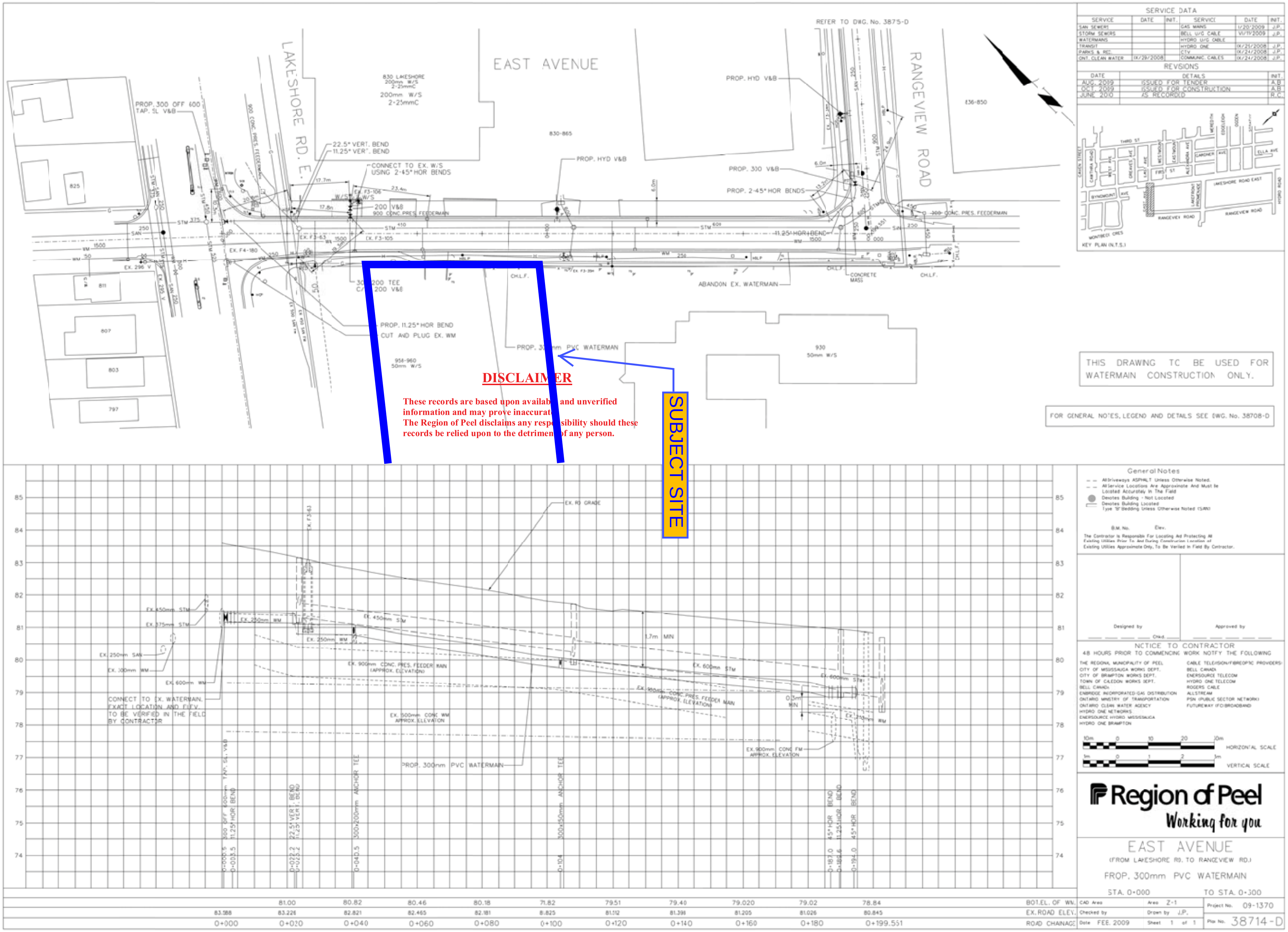
DEPARTMENTAL APPROVAL
W. SCOTT ANDERSON P.ENG.

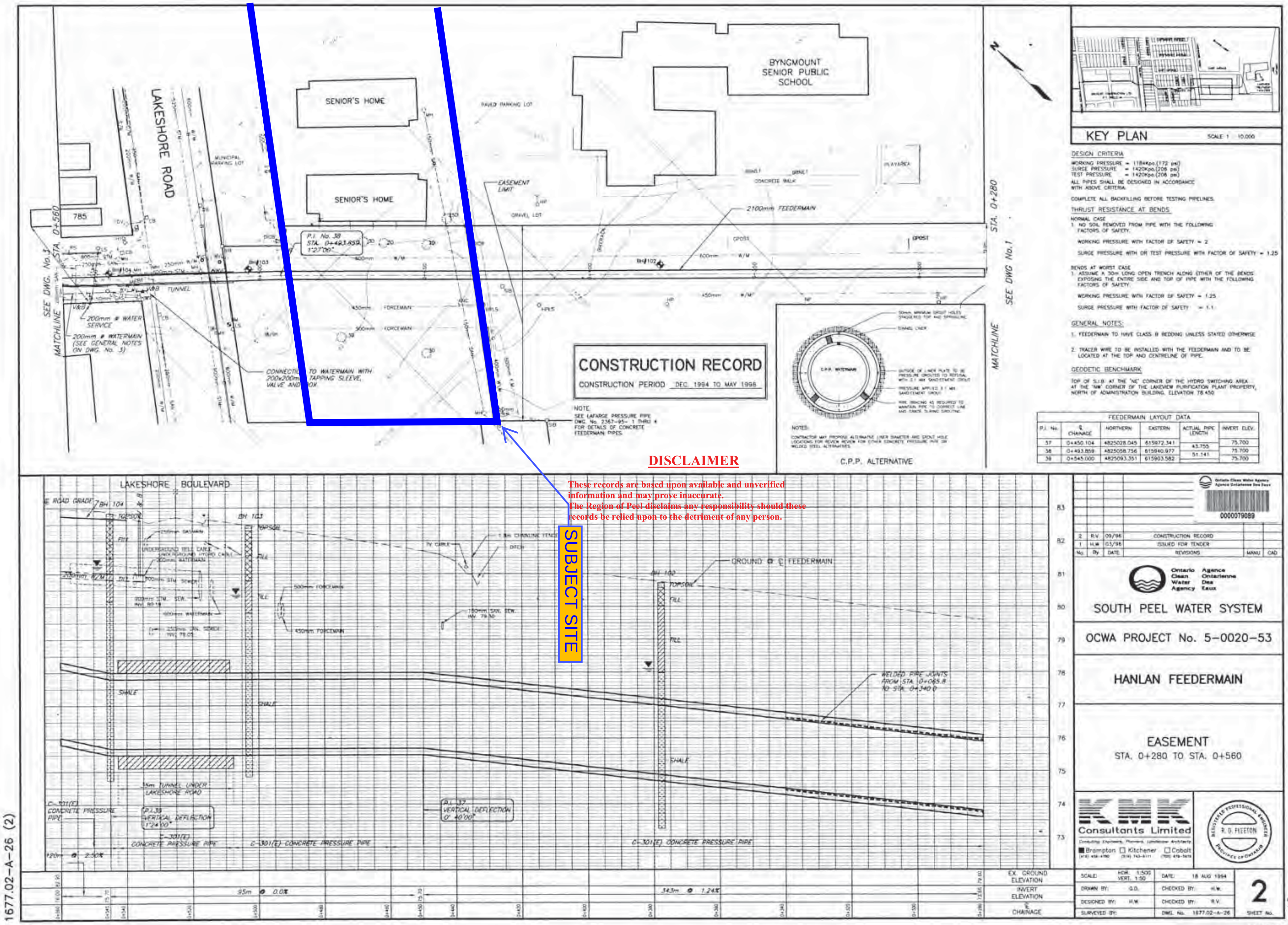
APPROVED BY
LICENSED PROFESSIONAL ENGINEER
K.A. BEDMAN
12/07/05
PROVINCE OF ONTARIO

MISSISSAUGA
Leading today for tomorrow
PRODUCED FOR - T&W, ENGINEERING AND WORKS

BYNGMOUNT AVENUE
EASTERLY FROM GOODWIN RD
STA 0+040 TO STA 0+245

SCALE HOR. 1:500 VERT. 1:50	AREA 2-1	PROJECT No. 04-112
C.A.D.D. BY S.R.	CHECKED BY B.L.	PLAN No. C-43365
DATE JULY 2005	SHEET 3 OF 4	


SUBJECT SITE
DISCLAIMER
These records are based upon available and unverified information and may prove inaccurate.
The Region of Peel disclaims any responsibility should these records be relied upon to the detriment of any person.
EAST AVENUE
LAKESHORE RD. E.
RANGEVIEW ROAD
REFER TO DWG. No. 3875-D
PROP. 300mm PVC WATERMAN
ABANDON EX. WATERMAIN
THIS DRAWING TO BE USED FOR WATERMAIN CONSTRUCTION ONLY.
FOR GENERAL NOTES, LEGEND AND DETAILS SEE DWG. No. 38708-D
SERVICE DATA
REVISIONS
KEY PLAN (N.T.S.)
General Notes
NOTICE TO CONTRACTOR
48 HOURS PRIOR TO COMMENCING WORK NOTIFY THE FOLLOWING
CONNECT TO EX. WATERMAIN. EXACT LOCATION AND ELEV. TO BE VERIFIED IN THE FIELD BY CONTRACTOR
PROP. 300mm PVC WATERMAIN
HORIZONTAL SCALE
VERTICAL SCALE
Region of Peel
Working for you
EAST AVENUE
(FROM LAKESHORE RD. TO RANGEVIEW RD.)
PROP. 300mm PVC WATERMAIN
STA. 0+000 TO STA. 0+300
Project No. 09-1370
Plan No. 38714-D
Date FEB. 2009
Drawn by J.P.
Sheet 1 of 1



LAKESHORE ROAD
SENIOR'S HOME
SENIOR'S HOME
BYNGMOUNT SENIOR PUBLIC SCHOOL
2100mm FEEDERMAIN
EASEMENT LIMIT
PAVED PARKING LOT
GRAVEL LOT
MUNICIPAL PARKING LOT
PLAYAREA
CONCRETE WALK
TUNNEL
785
P.I. No. 38 STA. 0+493.859 1°27'09"
200mm ø WATER SERVICE
200mm ø WATERMAIN (SEE GENERAL NOTES ON DWG. No. 3)
CONNECTION TO WATERMAIN WITH 200x200mm TAPPING SLEEVE, VALVE AND BOX
SEE DWG. No. 3 STA. 0+560 MATCHLINE
STA. 0+280 SEE DWG No.1 MATCHLINE
CONSTRUCTION RECORD
CONSTRUCTION PERIOD DEC. 1994 TO MAY 1996
NOTE: SEE LAFARGE PRESSURE PIPE DWG. No. 2367-95- 1 THRU 4 FOR DETAILS OF CONCRETE FEEDERMAIN PIPES.
C.P.P. ALTERNATIVE
KEY PLAN SCALE 1 : 10,000
DESIGN CRITERIA
WORKING PRESSURE = 1184Kpa (172 psi)
SURGE PRESSURE = 1420Kpa (206 psi)
TEST PRESSURE = 1420Kpa (206 psi)
ALL PIPES SHALL BE DESIGNED IN ACCORDANCE WITH ABOVE CRITERIA.
COMPLETE ALL BACKFILLING BEFORE TESTING PIPELINES.
THRUST RESISTANCE AT BENDS
NORMAL CASE
1. NO SOIL REMOVED FROM PIPE WITH THE FOLLOWING FACTORS OF SAFETY.
WORKING PRESSURE WITH FACTOR OF SAFETY = 2
SURGE PRESSURE WITH OR TEST PRESSURE WITH FACTOR OF SAFETY = 1.25
BENDS AT WORST CASE
1. ASSUME A 30m LONG OPEN TRENCH ALONG EITHER OF THE BENDS EXPOSING THE ENTIRE SIDE AND TOP OF PIPE WITH THE FOLLOWING FACTORS OF SAFETY.
WORKING PRESSURE WITH FACTOR OF SAFETY = 1.25
SURGE PRESSURE WITH FACTOR OF SAFETY = 1.1
GENERAL NOTES:
1. FEEDERMAIN TO HAVE CLASS B BEDDING UNLESS STATED OTHERWISE
2. TRACER WIRE TO BE INSTALLED WITH THE FEEDERMAIN AND TO BE LOCATED AT THE TOP AND CENTRELINE OF PIPE.
GEODETIC BENCHMARK
TOP OF S.I.B. AT THE 'NE' CORNER OF THE HYDRO SWITCHING AREA AT THE 'NW' CORNER OF THE LAKEVIEW PURIFICATION PLANT PROPERTY, NORTH OF ADMINISTRATION BUILDING. ELEVATION 78.450
FEEDERMAIN LAYOUT DATA
P.I. No. | ℄ CHAINAGE | NORTHERN | EASTERN | ACTUAL PIPE LENGTH | INVERT ELEV.
37 | 0+450.104 | 4825028.045 | 615972.341 | | 75.700
38 | 0+493.859 | 4825058.756 | 615940.977 | 43.755 | 75.700
39 | 0+545.000 | 4825093.351 | 615903.582 | 51.141 | 75.700
DISCLAIMER
These records are based upon available and unverified information and may prove inaccurate.
The Region of Peel disclaims any responsibility should these records be relied upon to the detriment of any person.
SUBJECT SITE
LAKESHORE BOULEVARD
GROUND @ ℄ FEEDERMAIN
BH 104
BH 103
BH 102
TOPSOIL
FILL
SHALE
35m TUNNEL UNDER LAKESHORE ROAD
WELDED PIPE JOINTS FROM STA. 0+065.8 TO STA. 0+340.0
P.I. 37 VERTICAL DEFLECTION 0°40'00"
P.I.38 VERTICAL DEFLECTION 1°24'00"
C-301(E) CONCRETE PRESSURE PIPE
120m @ 2.50%
95m @ 0.0%
343m @ 1.24%
EX. GROUND ELEVATION
INVERT ELEVATION
CHAINAGE
2 | R.V. | 09/96 | CONSTRUCTION RECORD
1 | H.W. | 03/96 | ISSUED FOR TENDER
No. | By | DATE | REVISIONS
Ontario Clean Water Agency
Agence Ontarienne Des Eaux
SOUTH PEEL WATER SYSTEM
OCWA PROJECT No. 5-0020-53
HANLAN FEEDERMAIN
EASEMENT
STA. 0+280 TO STA. 0+560
KMK Consultants Limited
Brampton Kitchener Cobalt
R. D. FEETON
SCALE: HOR. 1:500 VERT. 1:50
DATE: 18 AUG 1994
DRAWN BY: G.G.
CHECKED BY: H.W.
DESIGNED BY: H.W.
CHECKED BY: R.V.
DWG. No. 1677.02-A-26
SHEET No. 2
1677.02-A-26 (2)
38277-D

Region of Peel
working with you

Public Works

Water and Wastewater Servicing Analysis - Redevelopment of Peel Housing

958 & 960 East Avenue - City of Mississauga

GROWTH AND WATER RESOURCES

958 & 960 East Avenue -Existing Conditions

The subject property located at 959 & 960 East Avenue in the city of Mississauga and it is bounded by:

- **North:** Lakeshore Road East, fronted by small commercial uses and a five-storey independent living apartment building (Lakeside Court). Beyond, there are low-rise residential and the Gospel Assembly Church.

- **West**: Stable low-rise residential along Byngmount Avenue and Montbeck Crescent, known as the Lakeside Residential Neighbourhood.

- **East:** The site of the previous Byngmount Beach Public School and Light industrial uses fronting Lakeshore Road East and Rangeview Road

- **South:** The former Bygnmont Beach Site and Future EMS Satelite Station, Waterworks Park, Lakeview Water Treatment Plant and A.E. Crookes Park, which forms part of Mississauga's Waterfront Trail. Further south is the public marina.

Figure 1- Site Location

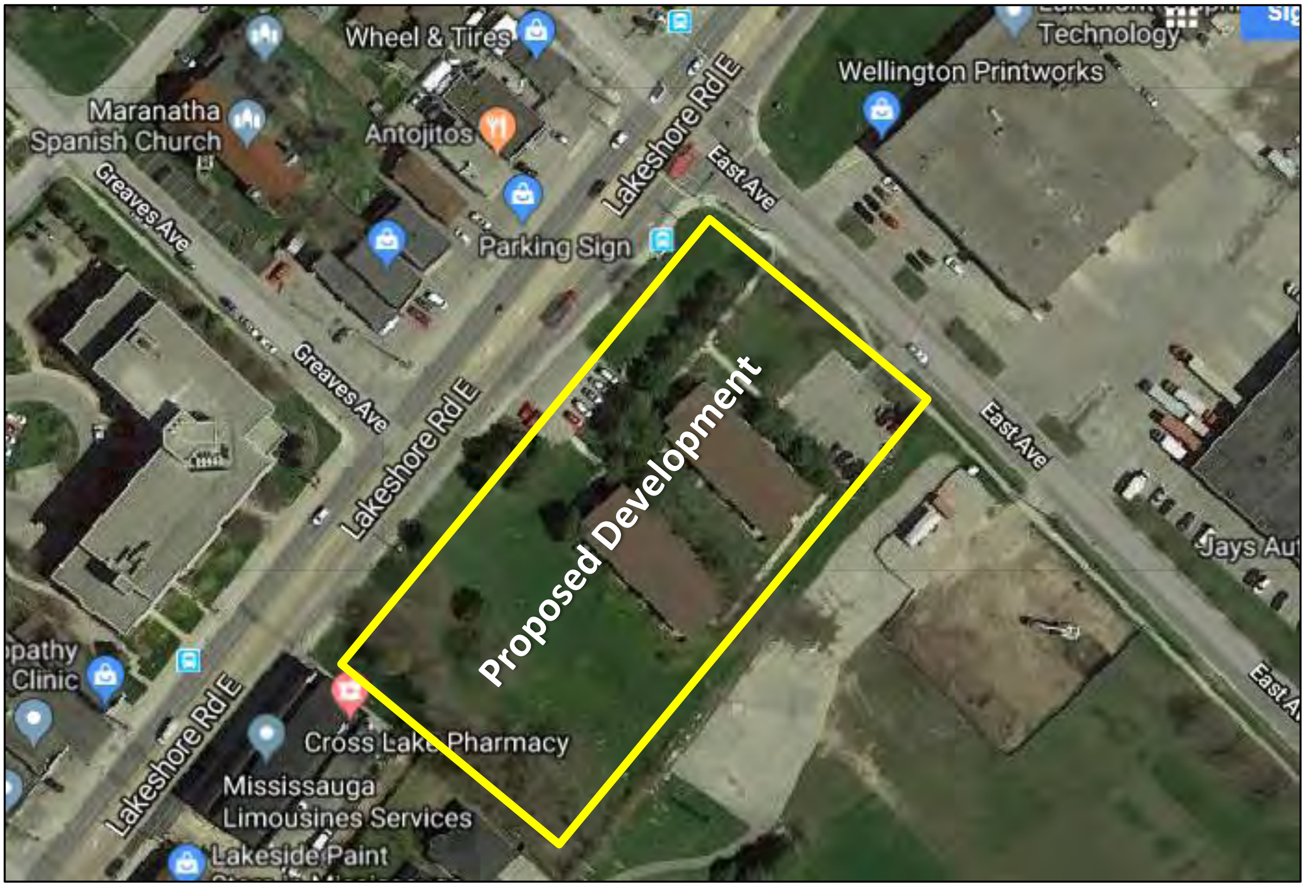


Figure 2- Site Location

The site currently consists of 30 units townhouses. The proposal is to demolish the existing units and redevelop by a new 7-storey building with a maximum of 180 units. The unit mix is expected to be 81 units at under 700 sq. feet and 99 units at over 700 square feet.

Based on the 2.7 people per unit the total growth forecast for this development is 405 people.

The Region of Peel provided the 2041 population and employment projections by small geographic unit (SGU) across the Region, referred to as SGU Scenario 16. The subject site is located within the SGU number M0169 as shown in Figure 3.
The total growth forecast for SGU M0169 is 300, and calculated growth forecast for this property is very lower than the proposed growth, as shown in Figure 3 & Table 1.

Table 1

Scenario / Year	Total
Growth to 2041 based on Weighted SGUs [1]	3
Proposed Development Growth	405

[1] *Weighted growth takes a proportion of SGUs based on area, 2041 growth numbers reference SGU Scenario 16.*

Table 2

Building Name	Address Existing Units	Existing Units	Proposed Growth Units	Proposed Growth Population	Growth Peak Wet Weather Flow (L/s)	Growth Water Demands (L/s)		
						Avg. Day	Max. Day	Peak Hour
East Ave.	958 & 960 East Ave.	130	150	405	5.0	1.3	2.6	3.9

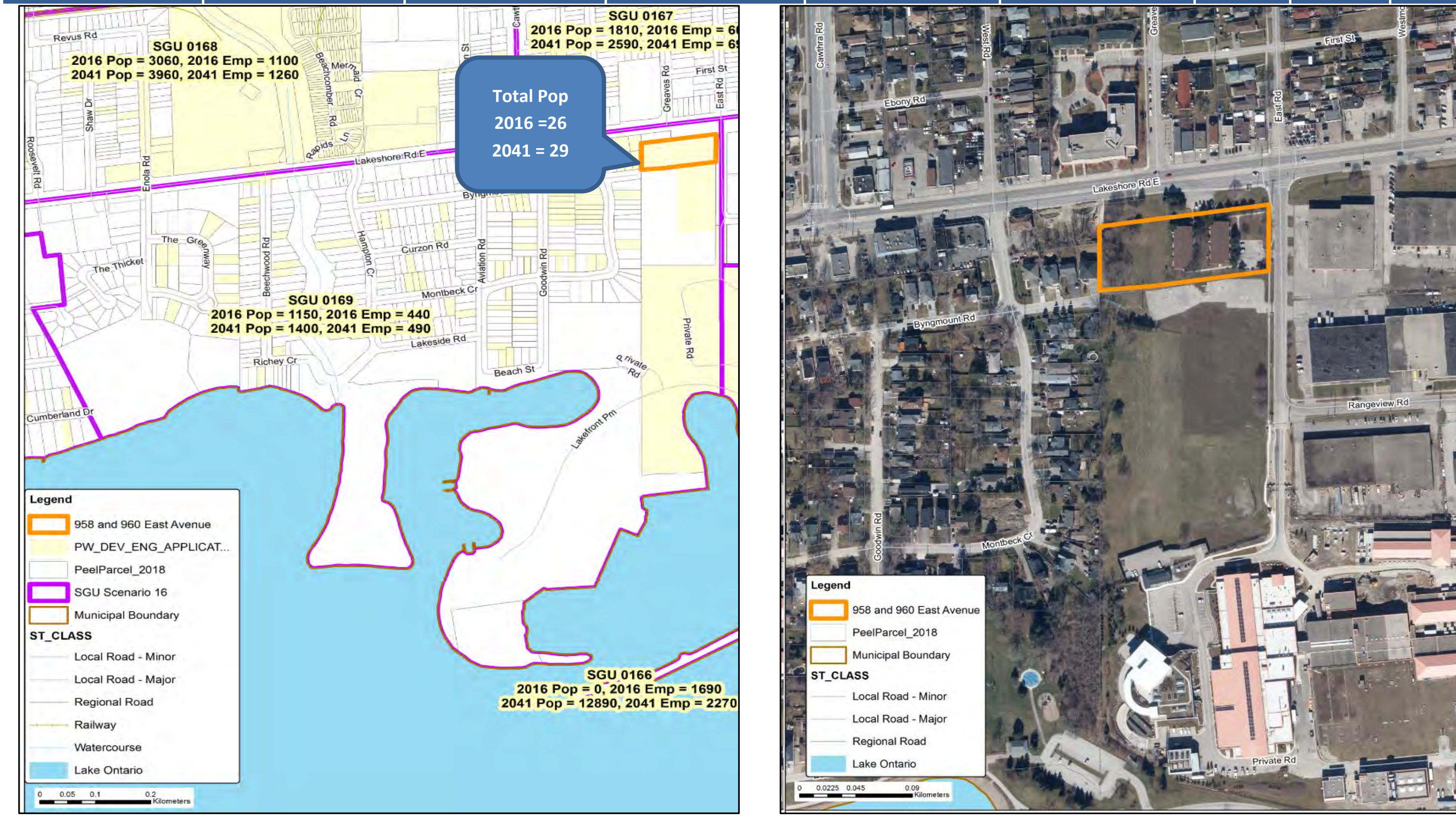


Figure 3 – Existing Condition

Water Servicing

This development located in Pressure Zone 1, close to the Lakeview Water Treatment Plant (WTP). Existing population in these parcels are serviced by the existing 300mm local watermain on East Avenue. The infrastructure in the vicinity of this development are as follows:

- A 300mm local distribution watermain on East Avenue (Zone 1)
- A 1500mm transmission main on East Road (Zone 1)
- A 900m transmission main on East Road (Zone 1)
- A 600mm major distribution feedermain on Lakeshore Road East (Zone 1)
- A 300mm local watermain on Lakeshore Road East (Zone 1)
- A 2100mm transmission main crossing the parcel (Zone 2)
- A 600mm major distribution feedermain crossing the parcel (Zone 1)

Proposed Water Servicing:

A 400 mm watermain is currently planned in the vicinity of the development, with expected installation year of 2026.

Based on the hydraulic modeling analysis, the existing 300mm local watermain on East Avenue has sufficient capacity in servicing the extra 150 units (equivalent population of 405) using the same service connection. The design criteria used in calculating the demand is:

- Unit Consumption Rate (Mix-used) = 280 L/cap/day
- MDD peak factor (Res)=2
- MDD peak factor (Emp)=1.4
- PHD peak factor (Res and Non-Res) =3

This servicing strategy demonstrates there is sufficient capacity under emergency conditions (i.e. fire) to service the additional proposed population for this development.

Wastewater Servicing

Sanitary flow from the existing units is currently being discharged to an existing 250 mm gravity sewer on Byngmount Avenue, west of the subject site, which conveys flow west via a network of sewers to the Beach Street Wastewater Pumping Station (WWPS). Flows from the Beach Street WWPS are then pumped to the 1650 mm Lakeshore Road East trunk sewer via two forcemains, 450 mm and 500 mm in diameter (Figure 5).

Proposed Wastewater Servicing

The existing 250 mm sewer on Byngmount Avenue is the only sanitary sewer can service the proposed development by gravity right now, however, the existing 300 mm & 375 mm downstream sewers already show some capacity issues.

The sanitary modeling analysis also revealed that the additional flow from this development will trigger downstream constraints and a potential surcharge along Montbeck Cr., Goodwin Road & Beach Street. Therfore the actual flow is being dischared from Lakeview Plant Backwash to the Montbeck Creek sewer was compared with the design flow, applied in sanitary model. The actual flows discharging from lakeview Backwash are little lower than the design flow right now.

Wastewater servicing option 1

As part of Inspiration Lakeview Development Servicing Study, a potential servicing option for Rangeview Road drainage area is to convey flow west by gravity to Beechwood SPS via a new proposed sewer along East Ave. and Lakeshore Road (Figure 4).

There is a possibility to service the proposed development area through the future proposed sanitary sewer along East Avenue and Lakeshore Road which it is proposed as part of Inspiration Lakeview Development Servicing Study, as shown in Figure 6. This will be a development driven project(Figure 6).

Figure 4 – Inspiration Lakeview Development Wastewater Servicing Strategy

Wastewater servicing option 2

Region of Peel has a plan to divert sanitary flow from Beach Street Sewage Pumping Station via a new gravity sewer along Aviation and Lakeshore Road East to Beechwood Sewage Pumping station. This trunk sewer will provide an opportunity to divert flow from Montbeck Cr. Sewer drainage area, north of Lakeshore Road to the new planned trunk sewer. Diverting flow from Montbeck Cr. Sewer drainage area, will provide extra capacity downstream of Montbeck Cr. Sewer to accommodate sanitary flow from Peel housing development (Figure 7).

The sanitary modeling analysis revealed capacity constraint in Montbeck Creek sewer, downstream of the development buildings. Therfore the actual flow is being dischared from Lakeview Plant Backwash to the Montbeck Creek sewer was compared with the design flow, applied in sanitary model. The actual flows, discharging from lakeview Backwash right now are little lower than the design flow. Therefore there is a opportunity to service this development buildings priror the completion of the Avition Road trunk sewer without sewer capacity restrictions.

Conclusions

Growth expected for this SGU based on the Growth Plan is lower than the development proposal. There is sufficient planned and future water infrastructure in the area to support the proposal. On the other hand, the sanitary flow from the development buildings can be dischared to the existing 250 mm sewer on Byngmount Avenue.

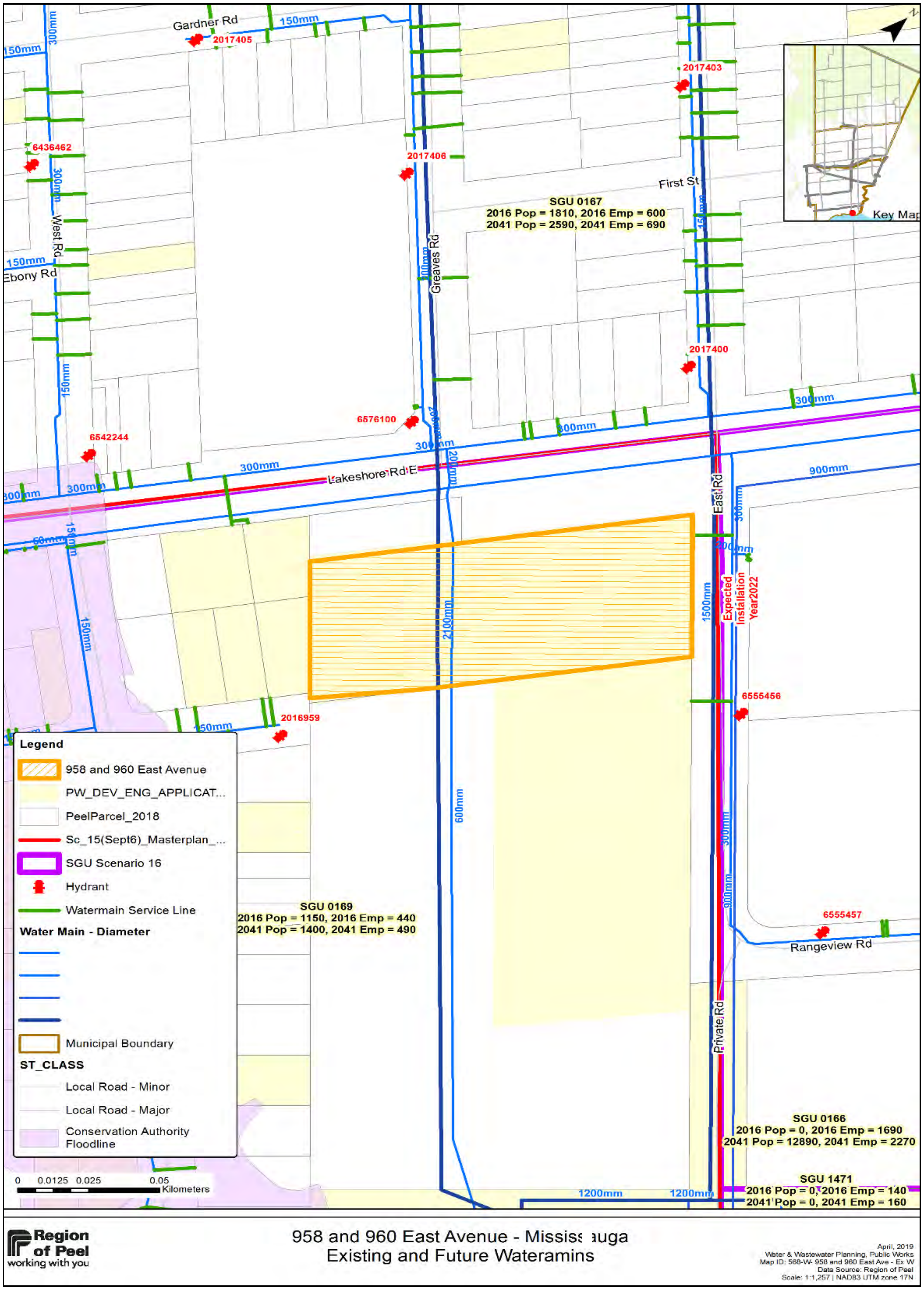


Figure 5– Water Servicing Strategy

Figure 6 – Wastewater Servicing Strategy- Option 1

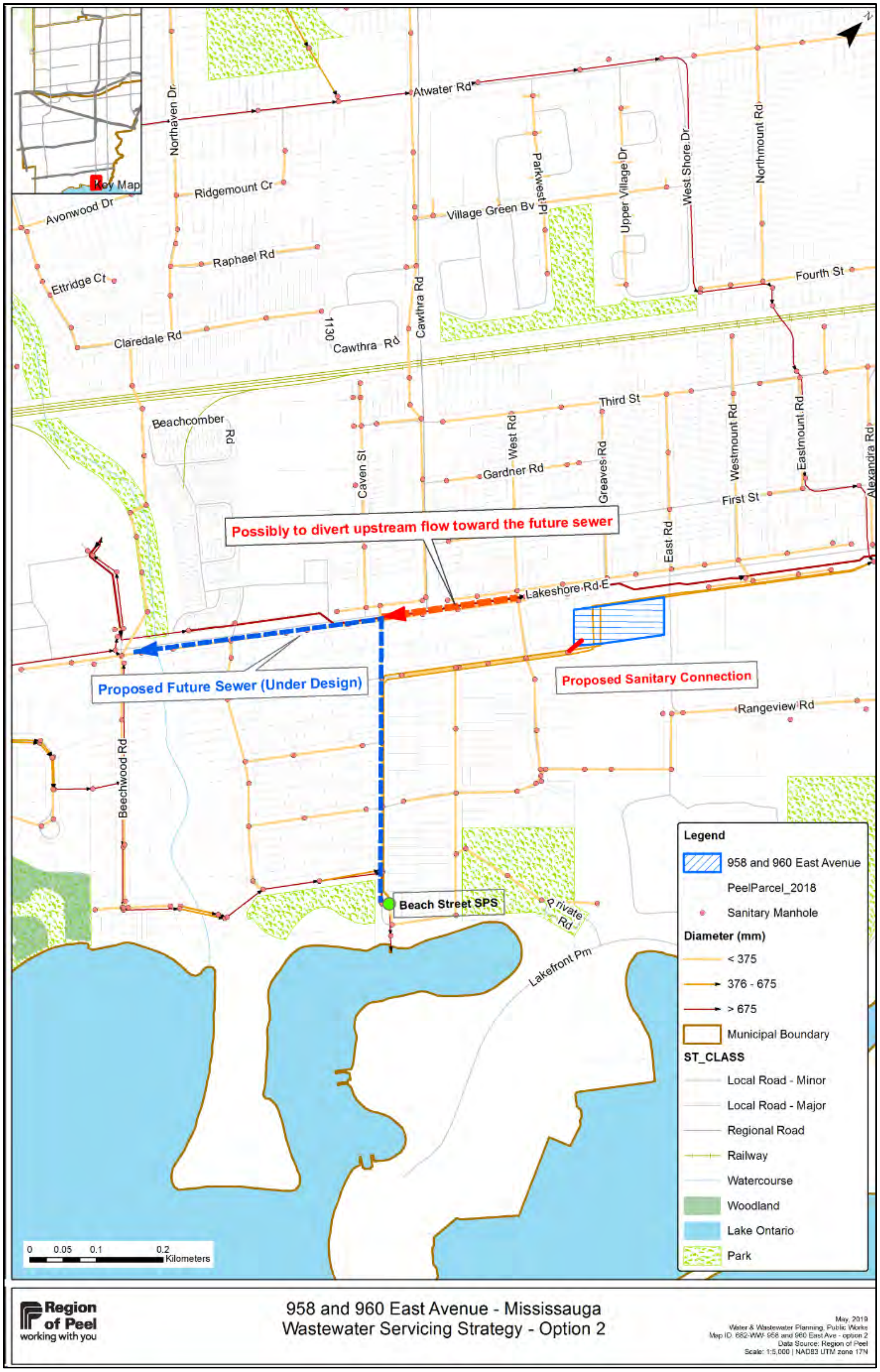


Figure 7– Wastewater Servicing Strategy- Option 2

APPENDIX C

958-960 East Avenue - Mixed Use Development
Water Demand Calculations

Designed By: **Jason M. Jenkins, P.Eng.**
Checked By: **Robert Filipuzzi, P.Eng.**
File No. 19002
Date: 10 March 2020

Domestic Water Supply Demands:

Per Region of Peel Watermain Design Criteria for Water Distribution Systems
- assume Average Day demand is 280 L/capita/day for residential uses
- assume Average Day demand is 300 L/capita/day for ICI uses
- assume Population Density (see chart)

Unit Type	Population Density
1 Bedroom Unit	1.68 Persons / Unit
2 Bedroom Unit	2.54 Persons / Unit
3 Bedroom Unit	3.1 Persons / Unit
Townhouse Unit	2.7 Persons / Unit
Retail Space	50 Persons / ha floor area
Office Space	3.3 Persons / 100 sq.m

Building	Building Data		Population	ADD	PHD = ADD x PF_{PHD}[1]	MDD = ADD x PF_{MDD}[2]
	Units	(sq.m)	pers	(L/s)	(L/s)	(L/s)
1 Bd Unit	65	--	109	0.35	1.06	0.71
2 Bd Unit	74	--	188	0.61	1.83	1.22
3 Bd Unit	12	--	37	0.12	0.36	0.24
Total	151	--	334	1.1	3.3	2.2

[1] Peak Hour Demand, PHD, is 3.0 for residential and 3.0 for ICI
[2] Max Day Demand, MDD, is 2.0 for residential and 1.4 for ICI

Fire Protection Supply Demands:
Per Water Supply for Public Fire Protection Manual, 1999, by the Fire Underwriters Survey

STEP 1: Calculate Fire Flow

$$F = 220 \cdot C \cdot \sqrt{A} \cdot \left(\text{various adjustments}\right)\ \text{L/min}$$

C = Coefficient related to type of construction:
= 1.5 for wood frame construction (Structure essentially all combustable)
= 1.0 for ordinary construction (brick or other masonry walls, combustable floor and interior)
= 0.8 for non combustable construction (unprotected metal structure components, masonry or metal walls)
= 0.6 for fire resistive construction (fully protected frame, floors, roof)

C =	0.6		
Largest Floor Area =	2,156	m2	
Floor Area Above =	2,156	m2	
Floor Area Below =	2,156	m2	Largest Floor + 25% x (Floor Above + Floor Below)
A =	3,234	m2	
F =	7,507	L/min	Round to the nearest 1000
F =	8,000	L/min	

STEP 2: Adjust for building occupancy (Note: Number shall not be less then 2000 L/min)
= -25% (Non-Combustible)
= -15% (Limited Combustible)
= 0 (Combustible)
= +15% (Free Burning)
= +25% (Rapid Burning)

Factor =	-15%	
F1 = F x Factor =	6,800	L/min

958-960 East Avenue - Mixed Use Development
Water Demand Calculations

STEP 3: Decrease F1 if building contains fire supression system

= -50% (Automatic Sprinklers)
= -30% (Adequately Designed System)
= Additional -10% if the water supply is standard for the system and the fire department hose lines required
= Additional -10% if the system is fully supervised

Factor =	-30%	
F2 = F1 x Factor =	-2,040	L/min

STEP 4: Increase F1 due to exposure / close proximity to other buildings (Note: Total shall not exceed 75%)

= 25% (0m to 3m)
= 20% (3.1m to 10m)
= 15% (10.1m to 20m)
= 10% (20.1m to 30.1m)
= 5% (30.1m to 45m)
= 0% (Greater then 45m)

Distances =	>45m / 35.7m / 35.1m / >45m	
Factors =	0% + 5% + 5% + 0%	
Factor =	10%	(max 75%)
F3 = F1 x Factor =	680	L/min

STEP 5: Calculate Fire Flow (Note: Fireflow shall not be less then 2000 L/min or greater then 45,000 L/min)

Fire Flow =	F1-F2+F3		
F1 =	6,800	L/min	
+ F2 =	-2,040	L/min	
+ F3 =	680	L/min	
Fire Flow =	5,440	L/min	Round to the nearest 1000
Fire Flow =	5,000	L/min	
Fire Flow =	83.3	L/s	

STEP 6: Calculate Total Water Demand (Max Day Demand + Fire Flow)

Recall Max Day Demand (from chart above) =	2.17	L/s
TOTAL Fire Demand =	85.5	L/s

958-960 East Avenue - Mixed Use Development

Water Demand Calculations

Designed By: **Jason M. Jenkins, P.Eng.**
Checked By: **Robert Filipuzzi, P.Eng.**
File No. 19002
Date: 10 March 2020

Recall Total Fire Demand = 85.5 L/s (Taken From Fire Flow Spreadsheet)

Hydrant Flow Test Results

Flow (gpm)	Flow (L/s)	Pressure (psi)	Pressure (kPa)	
0	0.0	86	593	- Assumed
314	19.8	85.5	590	
738	46.6	84	579	
1297	81.8	80	552	
1,355	85.5	79.49	548	- Interpolated using:
5,039	317.9	17.13	118	- Projected using:

$$Q_R = Q_F \times \frac{h_r^{0.54}}{h_f^{0.54}}$$

Hazen-Williams formula for watermain head loss:

$h_L = (10.675 * L * Q^{1.85}) / (C^{1.85} * D^{4.8655})$

where
- h_L = pressure drop (m)
- *L = length of pipe (m)*
- *Q = flow rate (m^3/s)*
- *C = roughness coefficient*
- *D = inside hydraulic diameter (m)*

New 100 mm domestic watermain

L= 2.5 m
D= 100 mm
C= 100

L= 13.0 m
D= 150 mm
C= 100

Peak Hour Flow		Head Loss, h_L				Residual Pressure [1]		Residual Pressure	
Q (L/s)	Q (m3/s)	(m)	(in)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)
3.3	0.00	0.02	0.7	0.02	0.16	86.0	593	**86.0**	**593**

[1] Residual pressure taken from above

New 150 mm fire service

L= 14.9 m
D= 150 mm
C= 100

Total Fire Flow (Max Day + Fire Flow)		Head Loss, h_L				Residual Pressure [1]		Residual Pressure	
Q (L/s)	Q (m3/s)	(m)	(in)	(psi)	(kPa)	(psi)	(kPa)	(psi)	(kPa)
85.5	0.09	3.43	134.9	4.87	33.61	79.5	548	**74.6**	**514**

[1] Residual pressure taken from above

APPENDIX D

958-960 East Avenue - Mixed Use Development

Region of Peel - Public Works Department

SANITARY SEWER DESIGN SHEET

Relevant Region of Peel Design Criteria:

Domestic sewage flow based upon a unit flow of **302.8 Lpcd**
Infiltration flow based upon a unit flow of **0.20 L/s/ha**
Residential density per Table 3-3 from Region DC Update dated May 2015
Commercial Population based on 50 people/ha floor area

Minimum flow velocity for partial flow = **0.75 m/s.**
Maximum flow velocity for pipe full flow = **3.5 m/s**
Peaking Factor per Harmon Equation
Mannings = 0.013

Designed By: **Jason M. Jenkins, P.Eng.**
Checked By: **Robert Filipuzzi, P.Eng.**
File No. 19002
Date: 10 March 2020

Location	from M.H.	to M.H.	DESIGN FLOW CALCULATIONS: # of Units	Cumulative Area (ha)	Density (p/unit) or (p/ha)	Cumulative Population (p)	Peaking Factor M	Sewage Flow (1) (L/s)	Infiltration Flow (2) (L/s)	Foundation Drain (3) (L/s)	Total Flow, Qd (1)+(2)+(3) (L/s)	SEWER DESIGN & ANALYSIS: Nominal Diameter (mm)	Pipe Slope (%)	Pipe Length (m)	Nominal Full Flow Capacity, Qf (L/s)	Nominal Full Flow Velocity (m/s)	Percent of Full Flow (Qd/Qf)	Actual Flow Velocity V (m/s)	Remarks
									0.20 L/s/ha										
PRE-DEVELOPMENT																			
958-960 East Ave	Subject Site	Byngmount Ave	30	0.7588	175	133	4.21	1.96	0.15		2.1								
REGION OF PEEL SERVICING ASSESSMENT																			
958-960 East Ave	Subject Site	Byngmount Ave	180	0.7588	2.7	486	3.98	6.78	0.15		6.9								
											4.8	← Increase in Sanitary Flow (Pre to Post) Analyzed by Region							
POST-DEVELOPMENT																			
958-960 East Ave	Subject Site	Prop. MH1	149	0.7588		334	4.06	4.76	0.15	1.6	6.5	200	2.00%	14.6	48.4	1.49	13.4%	1.04	
	Prop. MH1	Byngmount Ave									6.5	250	1.00%	74.8	62.0	1.22	10.5%	0.79	
											4.4	← Increase in Sanitary Flow (Pre to Post)							

Per the Hydrogeological report, long-term foundation subdrainage is expected to be 0.27 L/s. It is assummed that the sump pump will operate for 4 hours per day, with a corresponding maximum pumping rate of 1.6 L/s.

PRE-DEVELOPMENT	Units	Density	Population
Townhouses	40	2.7 / unit	108

POST-DEVELOPMENT	Units / GFA	Density	Population
1 Bd Unit	65	1.7 / unit	109
2 Bd Unit	74	2.5 / unit	188
3 Bd Unit	12	3.1 / unit	37
	151		334

226 Increase in Population

Unit Type	Population Density
1 Bedroom Unit	1.68 Persons / Unit
2 Bedroom Unit	2.54 Persons / Unit
3 Bedroom Unit	3.1 Persons / Unit
Townhouse Unit	2.7 Persons / Unit
Retail Space	50 Persons / ha floor area
Office Space	3.3 Persons / 100 sq.m

HYDROGEOLOGICAL INVESTIGATION

958-960 EAST AVENUE, MISSISSAUGA, ON

REGION OF PEEL

PROJECT NO.: 181-11306
DATE: DECEMBER 2018

DRAFT REPORT

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Table 4.5: Summary of Analytical Results

PARAMETER	LIMITS FOR SANITARY AND COMBINED SEWER DISCHARGE (TABLE 1)	LIMITS FOR STORM SEWER DISCHARGE (TABLE 2)	CONCENTRATION FOR BH18-2D (21/NOV/2018)	CONCENTRATION FOR BH18-4 (21/NOV/2018)
Total Suspended Solids (TSS, mg/L)	350	15	**<u>3000</u>**	**<u>1000</u>**
Total Manganese (Mn, mg/L)	5	0.05	**1.7**	**4.8**
Total Zinc (Zn, mg/L)	3	0.04	**0.15**	**0.081**
Total Kjeldahl Nitrogen (TKN, mg/L)	100	1	**7**	0.95

BOLD: Exceeds the Table 2 limit for discharge to the storm sewer
<u>BOLD AND UNDERLINED</u>: Exceeds both Table 1 and Table 2

4.6.1 BH18-2D WATER QUALITY (REGION OF PEEL STORM AND SANITARY DISCHARGE ASSESSMENT)

The BH18-2d sample had four (4) exceedances against the Table 2 – Limits for Storm Sewer Discharge: total suspended solids (TSS), Total Kjeldahl Nitrogen (TKN), total manganese (Mn), and total zinc (Zn). When compared to the less stringent Table 1 – Limits for Sanitary Sewer Discharge, the BH18-2d sample had one (1) exceedance: total suspended solids (TSS). If discharge to the storm sewer is considered, the groundwater will need pre-treatment for the exceedances listed above. If the option is to discharge to the sanitary sewer, the groundwater will need pre-treatment for TSS.

4.6.2 BH18-4 WATER QUALITY (REGION OF PEEL STORM AND SANITARY DISCHARGE ASSESSMENT)

The BH18-4 sample had three (3) exceedances against the Table 2 – Limits for Storm Sewer Discharge: total suspended solids (TSS), total manganese (Mn), and total zinc (Zn). When compared to the less stringent Table 1 – Limits for Sanitary Sewer Discharge, the BH18-4 sample had one (1) exceedance: total suspended solids (TSS). If discharge to the storm sewer is considered, the groundwater will need pre-treatment for the exceedances listed above. If the option is to discharge to the sanitary sewer, the groundwater will need pre-treatment for TSS.

CONSTRUCTION DEWATERING ASSUMPTIONS

Excavation, P2, No Caisson

PROJECT TITLE: 958-960 Lakeshore Road - Hydrogeological Investigation

Dewatering Calculation

Dupuit-Thiem Equation

FORMATION BEING DEWATERED (upper silty clay till/shale)

Table E-1: Construction Dewatering Flow Rate

Description	Symbol	Value	Unit	Explanation
Input Data				
Ground surface		82.12	m asl	approximate, based on BH Logs
Groundwater Elevation		80.20	m asl	BH18-4 Nov 2018 *High Water Level
Lowest excavation depth		72.12	m asl	Estimated
Base of Aquifer		70.00	m asl	Base of bedrock aquifer
Hydraulic Conductivity	K	6.74E-07	m/s	Geomean - SWRT
	K	5.82E-02	m/day	Converted to m/day
Dimensions of excavation	a	57.0	m	Plan of Survey (Young & Young Surveying Inc.)
	b	136.0	m	Plan of Survey (Young & Young Surveying Inc.)
Output				
Static Water Level		80.2	m asl	Shallow water level, maximum
Target Pumping Water Level		71.1	masl	1m below the underside of excavation
Water Level above aquifer bottom before dewatering	H	10.2	m	
Water level at excavation wall	h	1.1	m	
Effective Radius	r_e	49.7	m	Effective radius of rectangular excavation
Sichardt Estimate for Radius of Influence	R_{sich}	22.4	m	where c = 3000 for well approximation
Radius of Influence	R_0	72.0	m	Manipulated value, when $R_{sich} < r_{eff,}$ otherwise $R_0 = R_{sich}$
Construction Dewatering Flow Rate	Q	50.6	m^3/day	Construction flow rate - Duputit Equation
Safety Factor	S.F.	200.00	%	Enter desired safety factor
Maximum Construction Flow Rate (with applied factor of safety)	Q_{max}	101.2	m^3/day	during the initial period
Estimated Construction Dewatering Flow Rate		**50,600**	L/day	
Estimated Maximum Construction Flow Rate with Safety Factor		**101,000**	L/day	

Location	Volume of Excavation (m^3)	Porosity	Total (m^3)
Excavation, initial storage	70388.16	0.3	21116

Short Term (no caisson) = 1.17 L/s

CONSTRUCTION DEWATERING ASSUMPTIONS

Excavation, P2, Interlocking caisson

PROJECT TITLE: 958-960 Lakeshore Road - Hydrogeological Investigation

Dewatering Calculation

Dupuit-Thiem Equation

FORMATION BEING DEWATERED (upper silty clay till/shale)

Table E-2: Construction Dewatering Flow Rate

Description	Symbol	Value	Unit	Explanation
Input Data				
Ground surface		82.12	m asl	approximate, based on BH Logs
Groundwater Elevation		80.20	m asl	BH18-4 Nov 2018 *High Water Level
Lowest excavation depth		72.12	m asl	Estimated
Base of Aquifer		70.00	m asl	Base of bedrock aquifer
Hydraulic Conductivity	K	5.00E-08	m/s	Concrete caisson, conservative, assumed leaky
	K	4.32E-03	m/day	Converted to m/day
Dimensions of excavation	a	57.0	m	Plan of Survey (Young & Young Surveying Inc.)
	b	136.0	m	Plan of Survey (Young & Young Surveying Inc.)
Output				
Static Water Level		80.2	m asl	Shallow water level, maximum
Target Pumping Water Level		71.1	masl	1m below the underside of excavation
Water Level above aquifer bottom before dewatering	H	10.2	m	
Water level at excavation wall	h	1.1	m	
Effective Radius	r_e	49.7	m	Effective radius of rectangular excavation
Sichardt Estimate for Radius of Influence	R_{sich}	6.1	m	where c = 3000 for well approximation
Radius of Influence	R_0	55.8	m	Manipulated value, when $R_{sich} < r_{eff,}$ otherwise $R_0 = R_{sich}$
Construction Dewatering Flow Rate	Q	12.1	m^3/day	Construction flow rate - Dupuit Equation
Safety Factor	S.F.	200.00	%	Enter desired safety factor
Maximum Construction Flow Rate (with applied factor of safety)	Q_{max}	24.1	m^3/day	during the initial period
Estimated Construction Dewatering Flow Rate		**12,100**	L/day	
Estimated Maximum Construction Flow Rate with Safety Factor		**24,000**	L/day	

Location	Volume of Excavation (m^3)	Porosity	Total (m^3)
Excavation, initial storage	70388.16	0.3	21116

Short Term (if caisson) = 0.27 L/s

Long Term Drainage Calculation
PROJECT TITLE: 958-960 Lakeshore Road - Hydrogeological Investigation

Foundation Drains, P2
Permanent drainage calculation

Dupuit-Thiem Equation

Table F-1: Permanent Drainage Flow Rate Estimate

Description	Symbol	Value	Unit	Explanation
Input Data				
Ground surface		82.12	m asl	approximate, based on BH Logs
Groundwater Elevation		80.20	m asl	BH18-4 Nov 2018 *High Water Level
Approximate P2 Slab		75.12	m asl	Estimated
Base of Aquifer		59.0	m asl	Base of bedrock aquifer
Hydraulic Conductivity	K	5.00E-09	m/s	Caisson effect
	K	4.32E-04	m/day	Converted to m/day
Dimensions of excavation	a	57.0	m	Plan of Survey (Young & Young Surveying Inc.)
	b	136.0	m	Plan of Survey (Young & Young Surveying Inc.)
Output				
Static Water Level		80.2	m asl	Shallow water level, maximum
Target Pumping Water Level		74.6	masl	0.5m below the P2 slab
Water Level above aquifer bottom before dewatering	H	21.2	m	
Water level at excavation wall	h	15.6	m	
Effective Radius	r_e	49.7	m	Effective radius of rectangular excavation
Sichardt Estimate for Radius of Influence	R_{sich}	1.2	m	where c = 3000 for well approximation
Radius of Influence	R_0	50.9	m	
Long Term Drainage Calculation	Q	11.8	m^3/day	Dupuit Thiem Steady State Flow to subdrain
Safety Factor	S.F.	200.00	%	Enter desired safety factor
Maximum Long-Term Flow Rate (with applied factor of safety)	Q_{max}	23.7	m^3/day	Seepage from infiltration, and through caisson
Estimated Long-Term Drainage Flow Rate		**11,800**	L/day	
Estimated Long-Term Drainage Flow Rate with Safety Factor		**23,700**	L/day	

Long Term = 0.27 L/s

APPENDIX E

WATERSHED BOUNDARY: CAWTHRA CREEK

A-1 - Watershed Boundaries

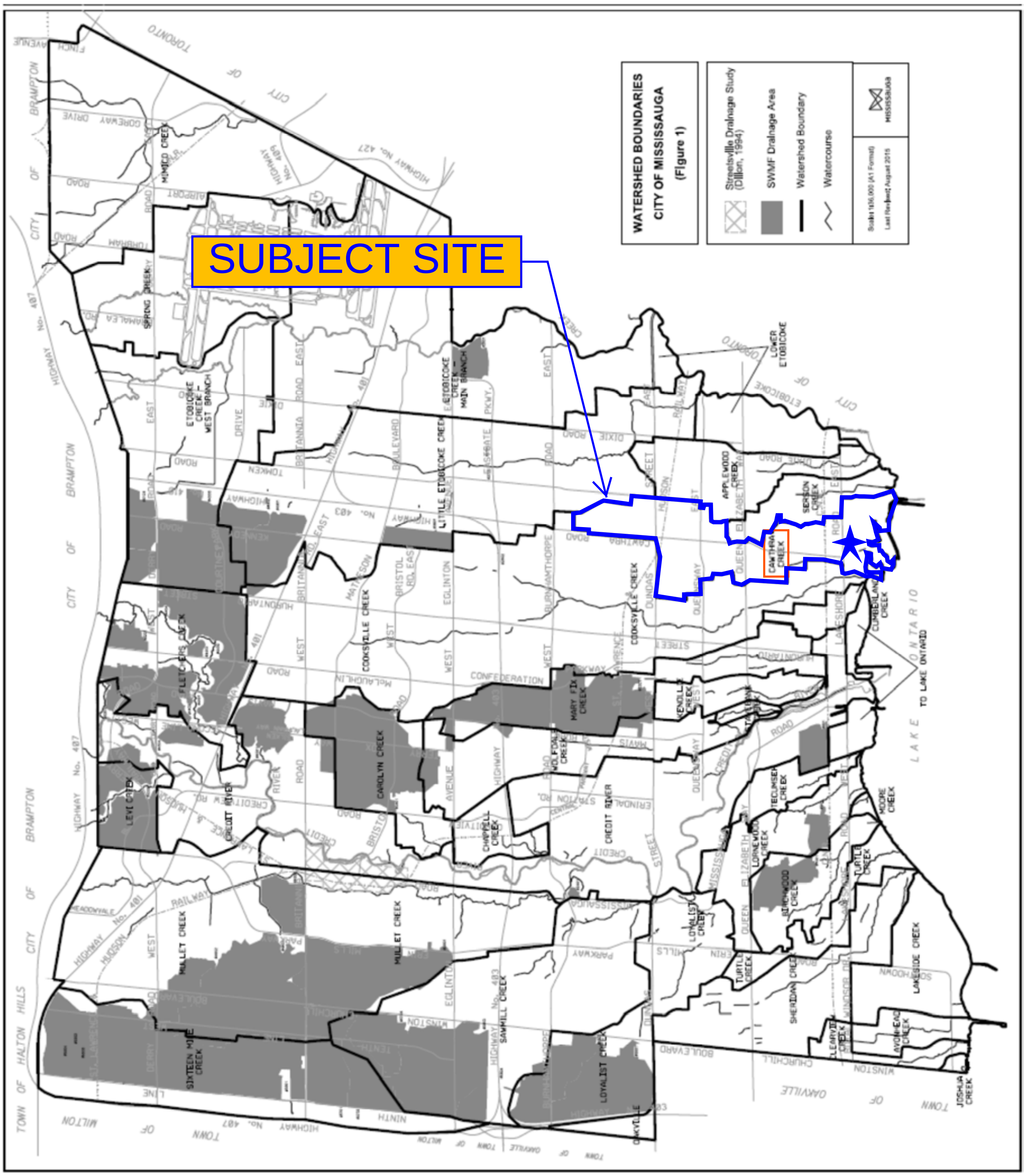


TABLE 2.01.03.03a: STORMWATER QUANTITY CONTROL REQUIREMENTS

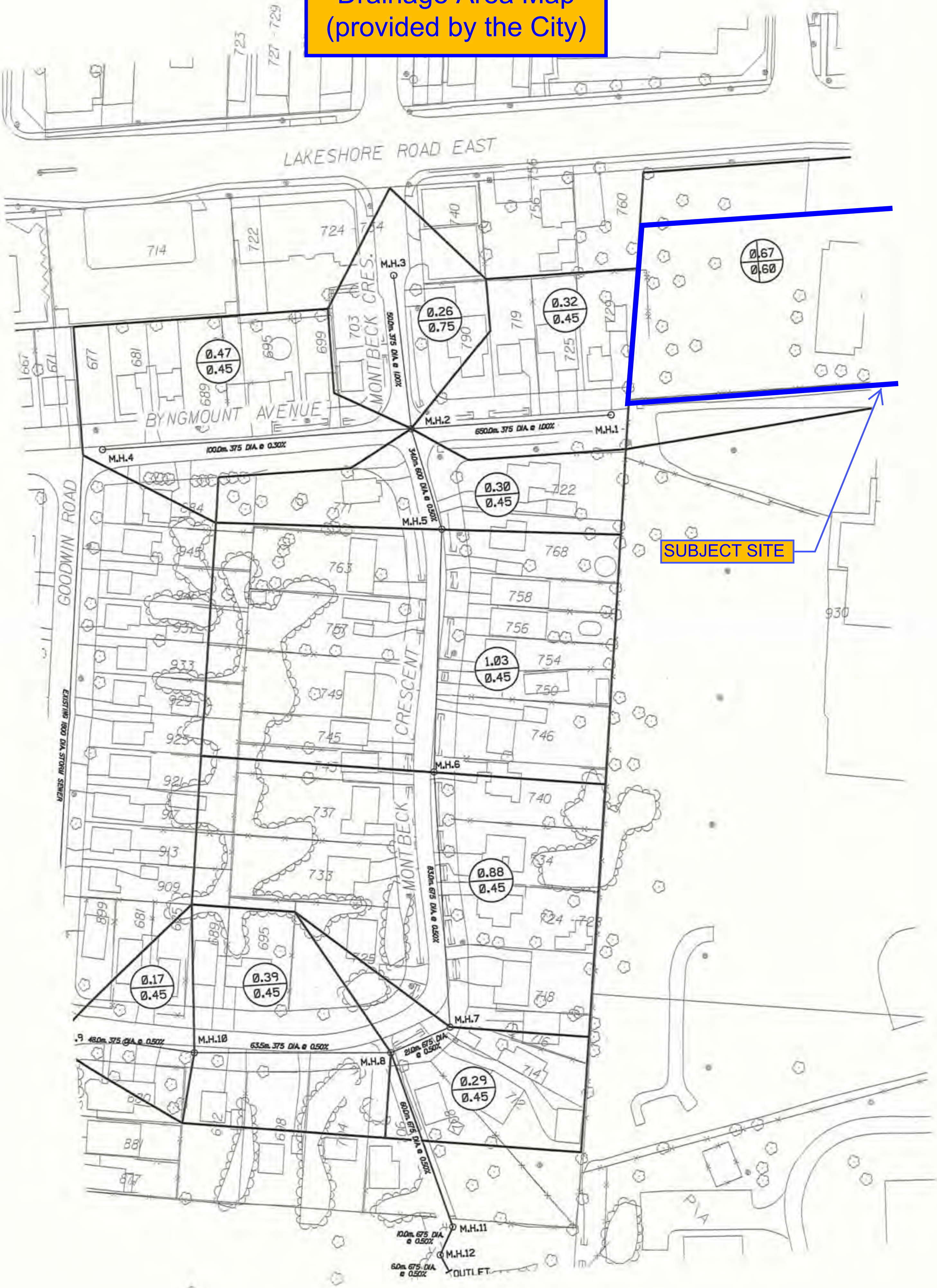
Note 1: In all cases, storm sewer capacity constraints or downstream concerns may govern

Note 2: Where “pre-development” is listed as part of the requirement, it is implied as raw land for which the run-off co-efficient=0.25 but will not exceed 0.50 for a site that may already be developed

Note 3: CVC-Credit Valley Conservation, TRCA-Toronto Region Conservation Authority, CH-Conservation Halton

Subwatershed Name (Conservation Authority)	Quantity Control Criteria	References & Notes
Applewood Creek (CVC)	100 Year Post to 2 Year Pre-development Control	-
Avonhead Creek (CVC)	100 Year Post to 2 Year Pre-development Control	Southdown District Master Drainage Plan (Totten Sims Hubicki, 2000)
Birchwood Creek (CVC)	100 Year Post to 2 Year Pre-development Control	-
Carolyn Creek (CVC)	Provide post to pre control for all storms (i.e. 2,5,10,25,50 & 100 year)	Master Drainage Study (Winter Associates, 1987)
Cawthra Creek (CVC)	100 Year Post to 2 Year Pre-development Control	Drainage diversion to Cooksville Creek and a very small area draining to creek.
Chappell Creek (CVC)	10 Year Post to 2 Year Pre-development Control	-
Clearview Creek (CVC)	100 Year Post to 2 Year Pre-development Control	Southdown District Master Drainage Plan (Totten Sims Hubicki, 2000)
Cooksville Creek (CVC)	100 Year Post to 2 Year Pre-development Control	Revised development standards via Mississauga Staff report to City Council
Credit River - Norval to Port Credit (CVC)	No control required	Subwatershed Study in progress (partially complete)
Cumberland Creek (CVC)	No control required	-
Etobicoke Creek - Main Branch & Lower Etobicoke (TRCA)	No control required in the City of Mississauga	Hydrologic Model: VISUAL OTTHYMO-Return period peak flows based on the AES - 12 hour design storm Hydrology Study:Etobicoke Creek Hydrology Update (MMM Group, 2013)

Drainage Area Map
(provided by the City)



SUBDIVISION__MOUNTBECK CRESCENT / BYNGMOUNT ROAD

CONSULTANT

MAJOR DRAINAGE AREA____Z-10

CITY OF MISSISSAUGA

STORM DRAINAGE DESIGN CHART FOR CIRCULAR DRAINS FLOWING FULL

SHEET No.__1__OF__1 DATE__FEB 2004

PROJECT No.___17111 04112 / 04113

DESIGNED BY__BORIS LENCE C.E.T.

LINE No.	LOCATION	MH	MH	DRAINAGE AREA A (ha)	RUN OFF COEFF C	IMP AREA A*C	ACCUMM DRAINAGE AREA (ha)	ACCUMM IMP AREA	TIME OF FLOW (min)	TIME OF ENTRY (min)	TIME OF CONC (min)	R'FALL INTENS I (mm/hr)	EXP FLOW (m3/sec)	TYPE OF PIPE	MANNING COEFF n	SLOPE S (%)	DIA D (mm)	LENGTH L (m)	VEL V (m/sec)	CAP Q (m3/sec)	UPSTREAM INVERT (m)	DOWNSTREAM INVERT (m)	TIME IN SECTION (min)
1																							
2	EXTERNAL (School East Of Byngmount)	-	1	0.67	0.60	0.40	0.67	0.40															
3	Byngmount Rd.- East Of Montbeck Cres.	1	2	0.32	0.45	0.14	0.99	0.55	-	15.00	15.00	99	0.15	conc.	0.013	1.00	375	65.00	1.60	0.18	79.000	78.350	0.68
4																							
5																							
6	Montbeck Cres. - North Of Byngmount Rd.	3	2	0.26	0.75	0.20	0.26	0.20	-	15.00	15.00	99	0.05	conc.	0.013	1.00	375	50.00	1.60	0.18	78.900	78.400	0.52
7																							
8																							
9	Byngmount Rd.- West Of Montbeck Cres.	4	2	0.47	0.45	0.21	0.47	0.21	-	15.00	15.00	99	0.06	conc.	0.013	0.30	375	100.00	0.88	0.10	77.960	77.660	1.90
10																							
11																							
12	Montbeck Cres. - South Of Byngmount Rd.	2	5	0.30	0.45	0.14	2.02	1.09	1.90	15.00	16.90	92	0.28	conc	0.013	0.25	600	34.00	1.10	0.32	77.600	77.515	0.52
13	Montbeck Cres. - South Of Byngmount Rd.	5	6	1.03	0.45	0.46	3.05	1.55	0.52	16.90	17.41	91	0.39	conc.	0.013	0.50	600	79.00	1.55	0.45	77.490	77.095	0.85
14	Montbeck Cres. - South Of Byngmount Rd.	6	7	0.88	0.45	0.40	3.93	1.95	0.85	17.41	18.26	88	0.48	conc.	0.013	0.50	675	83.00	1.68	0.62	77.020	76.605	0.82
15	Montbeck Cres. - South Of Byngmount Rd.	7	8	0.29	0.45	0.13	4.22	2.08	0.82	18.26	19.09	86	0.49	conc.	0.013	0.50	675	21.00	1.68	0.62	76.605	76.500	0.21
16																							
17																							
18	Montbeck Cres. - East Of Goodwin Rd.	9	10	0.17	0.45	0.08	0.17	0.08	0.00	15.00	15.00	99	0.02	conc	0.013	1.00	375	24.00	1.60	0.18	77.670	77.430	0.25
19	Montbeck Cres. - East Of Goodwin Rd.	10	8	0.39	0.45	0.18	0.56	0.25	0.25	15.00	15.25	98	0.07	conc.	0.013	1.00	375	63.00	1.60	0.18	77.430	76.800	0.65
20																							
21	Montbeck Cres. Through Easement	8	11	0.00	0.45	0.00	4.78	2.33	0.21	19.09	19.29	85	0.55	conc.	0.013	0.50	675	60.00	1.68	0.62	76.200	75.900	0.60
22	Montbeck Cres. Through Park	11	12	0.00	0.45	0.00	4.78	2.33	0.60	19.29	19.89	83	0.54	conc.	0.013	0.50	675	10.00	1.68	0.62	75.900	75.850	0.10
23	Montbeck Cres. Through Park	12	Outlet	0.00	0.45	0.00	4.78	2.33	0.10	19.89	19.99	83	0.54	conc.	0.013	0.50	675	6.00	1.68	0.62	75.850	75.820	0.06
24																							
25																							
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34																							
35																							

DWN. BY:	JMJ
DESIGNED BY:	JMJ
CHECKED BY:	RDF
SCALE:	NTS
DATE:	MARCH 2020
SHEET NO:	1 OF 1

fabian papa & partners
A Division of FP&P HydraTek Inc.
216 Chrislea Road, Suite 204
Vaughan, Ontario, L4L 8S5
t: 905-264-2420
www.fabianpapa.com

PROJECT NO:	DWG NO:	REV NO:
19002	PRE-SWM1	0

PROJECT NAME:

958-960 EAST AVE.
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

PRE-DEVELOPMENT SWM PLAN
(DRAINING TO BYNGMOUNT AVE.)

DWN. BY:	JMJ
DESIGNED BY:	JMJ
CHECKED BY:	RDF
SCALE:	NTS
DATE:	MARCH 2020
SHEET NO:	1 OF 1

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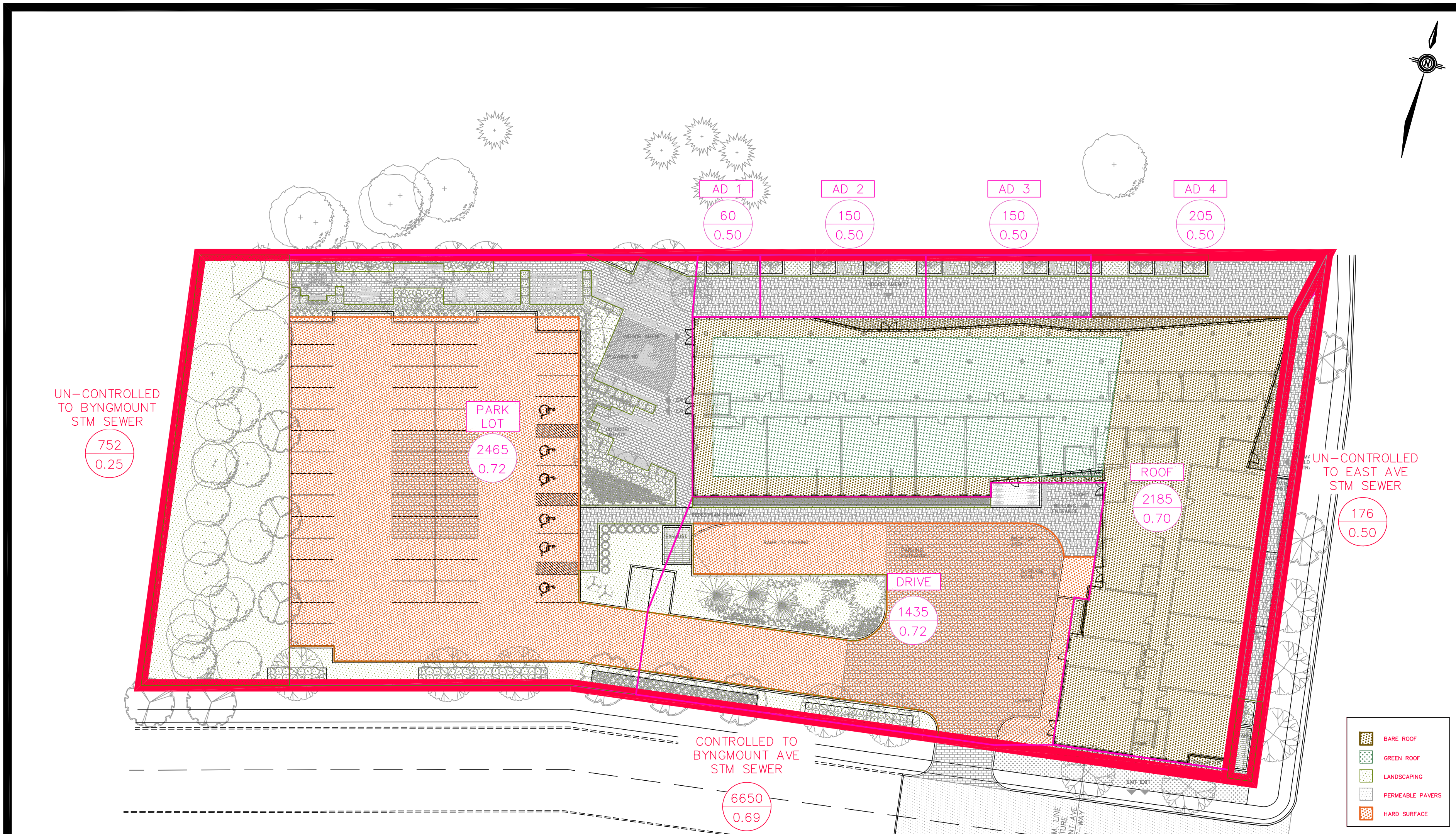
PROJECT NO:	DWG NO:	REV NO:
19002	PRE-SWM2	0

PROJECT NAME:

958-960 EAST AVE.
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

PRE-DEVELOPMENT SWM PLAN
(DRAINING TO EAST AVE.)



DWN. BY:	JMJ
DESIGNED BY:	JMJ
CHECKED BY:	RDF
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DATE:	MARCH 2020
SHEET NO:	1 OF 1

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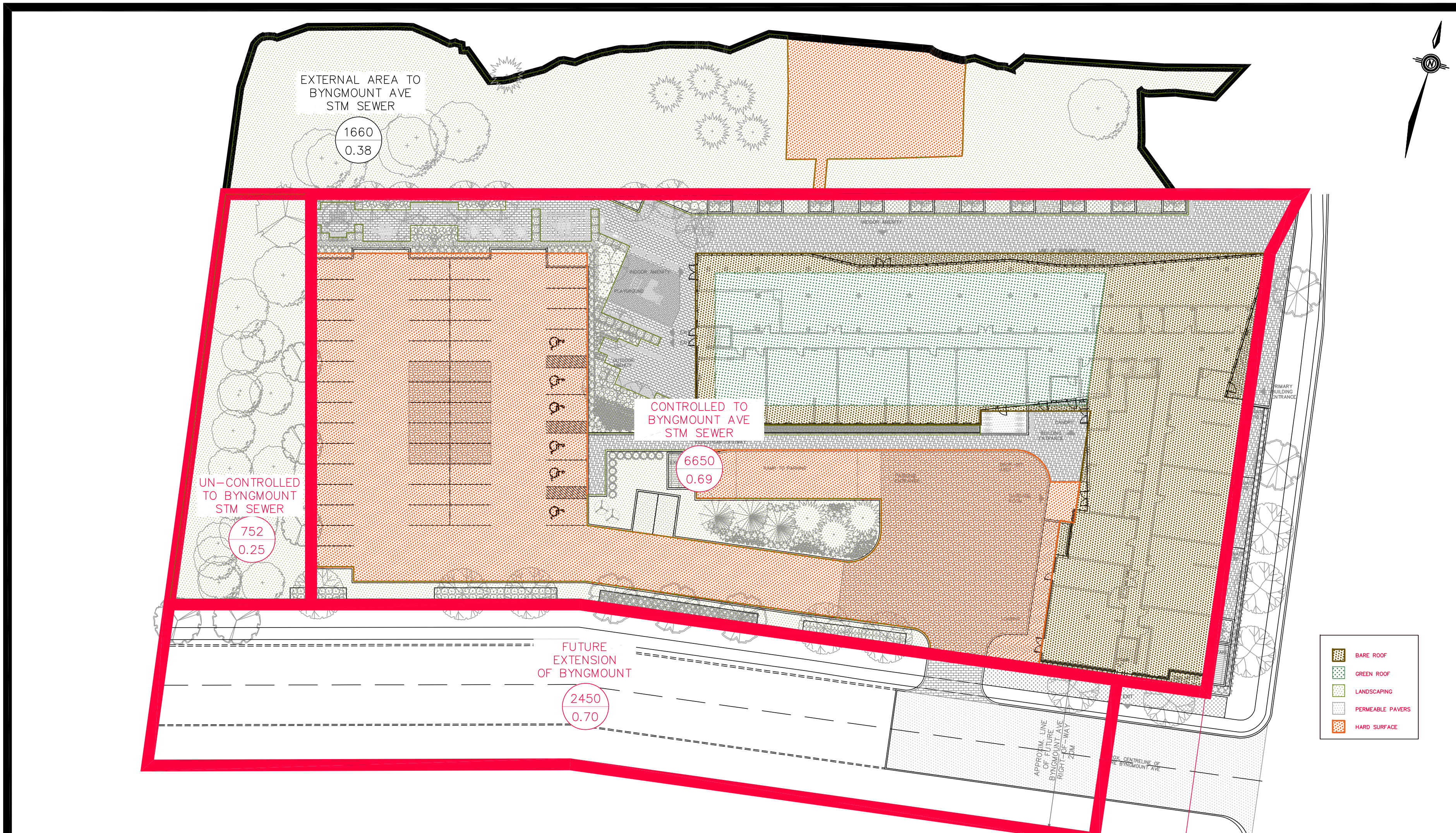
PROJECT NO:	DWG NO:	REV NO:
19002	POST-SWM	0

PROJECT NAME:

958-960 EAST AVE.
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

POST-DEVELOPMENT SWM PLAN



DWN. BY:	JMJ
DESIGNED BY:	JMJ
CHECKED BY:	RDF
SCALE:	NTS
DATE:	MARCH 2020
SHEET NO:	1 OF 1

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PROJECT NO:	DWG NO:	REV NO:
19002	FUTURE-SWM	0

PROJECT NAME:

958-960 EAST AVE.
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

FUTURE SWM PLAN

958-960 East Avenue
Run-Off Coefficients
City of Mississauga

fp&p

Designed By: **Jason Jenkins**, P.Eng., P.E.
Checked By: Robert Filipuzzi, P.Eng.
File No. 19002
Date: 10 March 2020

Runoff Coefficients (Subject Site)

Pre-Development (to Byngmount)

Roof Bare	728	12.1%	0.90	0.11
Green Roof		0.0%	0.45	0.00
Landscape	5,098	84.4%	0.25	0.21
Permeable		0.0%	0.55	0.00
Hard Surface	212	3.5%	0.90	0.03
	6,038	100%		0.35

Pre-Development (to East Avenue)

Roof Bare	150	9.7%	0.90	0.09
Green Roof		0.0%	0.45	0.00
Landscape	826	53.6%	0.25	0.13
Permeable		0.0%	0.55	0.00
Hard Surface	564	36.6%	0.90	0.33
	1,540	100%		0.55

Pre-Development

Roof Bare	878	11.6%	0.90	0.10
Green Roof	0	0.0%	0.45	0.00
Landscape	5,924	78.2%	0.25	0.20
Permeable	0	0.0%	0.55	0.00
Hard Surface	776	10.2%	0.90	0.09
	7,578	100%		0.39

Post-Development (Un-Controlled to Byngmount)

Roof Bare		0.0%	0.90	0.00
Green Roof		0.0%	0.45	0.00
Landscape	752	100.0%	0.25	0.25
Permeable		0.0%	0.55	0.00
Hard Surface		0.0%	0.90	0.00
	752	100%		0.25

Post-Development (Controlled to Byngmount)

Roof Bare	1,398	21.0%	0.90	0.19
Green Roof	811	12.2%	0.45	0.05
Landscape	1,016	15.3%	0.25	0.04
Permeable	1,000	15.0%	0.55	0.08
Hard Surface	2,425	36.5%	0.90	0.33
	6,650	100%		0.69

Post-Development (Un-Controlled to East Avenue)

Roof Bare		0.0%	0.90	0.00
Green Roof		0.0%	0.45	0.00
Landscape	29	16.5%	0.25	0.04
Permeable	147	83.5%	0.55	0.46
Hard Surface		0.0%	0.90	0.00
	176	100%		0.50

Post-Development

Roof Bare	1,398	18.4%	0.90	0.17
Green Roof	811	10.7%	0.45	0.05
Landscape	1,797	23.7%	0.25	0.06
Permeable	1,147	15.1%	0.55	0.08
Hard Surface	2,425	32.0%	0.90	0.29
	7,578	100%		0.64

Runoff Coefficients (External Areas)

Adjacent External Areas

External Area North	1,660	40.4%	0.38	0.15
Future Byngmount	2,450	59.6%	0.70	0.42
	4,110	100%		0.57

958-960 East Avenue

City of Mississauga - Engineering Services Department

STORM SEWER DESIGN SHEET

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$I_{2\text{-}year} = \frac{610}{(T+4.6)^{0.78}}$ = 59.89 mm/hr	$I_{10\text{-}year} = \frac{1010}{(T+4.6)^{0.78}}$ = 99.17 mm/hr	$I_{50\text{-}year} = \frac{1300}{(T+4.7)^{0.78}}$ = 127.13 mm/hr
$I_{5\text{-}year} = \frac{820}{(T+4.6)^{0.78}}$ = 80.51 mm/hr	$I_{25\text{-}year} = \frac{1160}{(T+4.6)^{0.78}}$ = 113.89 mm/hr	$I_{100\text{-}year} = \frac{1450}{(T+4.9)^{0.78}}$ = 140.69 mm/hr

Designed By: **Jason M. Jenkins, P.Eng.**
Checked By: **Robert Filipuzzi, P.Eng.**
File No.: 19002
Date: 10 March 2020

Street	From MH	To MH	A (ha)	R	A x R	Accum. A x R	T_c (min)	I (mm/hr)	Q_{act} (L/s)	Size of Pipe (mm)	Slope (%)	Nominal Capacity Q_{cap} (L/s)	Full Flow Velocity (m/s)	Actual Flow Velocity (m/s)	Length (m)	Time in Sect. (min)	Total Time (min)	Q_{act}/Q_{cap}	Remarks
ALLOWABLE RELEASE RATES (BASED ON PRE-DEVELOPMENT CONDITIONS)																			
2-YEAR (TO BYNGMOUNT)	External + Site	Byngmount	0.6038	0.35	0.212	0.212	15.0	59.9	**35.3**	←	Allowable Discharge to Byngmount Storm Sewer								
2-YEAR (TO EAST AVE.)	Site	East Avenue	0.1540	0.50	0.077	0.077	15.0	59.9	**12.8**	←	Allowable Discharge to East Ave. Storm Sewer (Max. Coefficient = 0.50)								
			0.7578	0.38	0.289														
POST DEVELOPMENT CONDITIONS (NO ATTENUATION)																			
100-YEAR (TO BYNGMOUNT)	External + Site	Byngmount	0.7402	0.65	0.480	0.480	15.0	140.7	**234.4**	←	Attenuation Required								
100-YEAR (TO EAST AVE.)	Site	East Avenue	0.0176	0.50	0.009	0.009	15.0	140.7	**4.3**	←	OKAY. No Attenuation Required				Includes Coefficient Adjustment				
			0.7578	0.64	0.489														
POST DEVELOPMENT CONDITIONS (NO ATTENUATION) - BY CATCHMENT AREA																			
Un-Controlled Area to Byngmount			0.0752	0.25	0.019	0.019	15.0	140.7	9.2										
AD 1			0.0060	0.50	0.003	0.003	15.0	140.7	1.5										
AD 2			0.0150	0.50	0.008	0.008	15.0	140.7	3.7										
AD 3			0.0150	0.50	0.008	0.008	15.0	140.7	3.7										
AD 4			0.0205	0.50	0.010	0.010	15.0	140.7	5.0		Includes Coefficient Adjustment								
Roof			0.2185	0.70	0.153	0.153	15.0	140.7	74.7										
Drive			0.1435	0.72	0.103	0.103	15.0	140.7	50.3										
Parking Lot			0.2465	0.72	0.177	0.177	15.0	140.7	86.4										
			0.7402	0.65	0.480				**234.4**										
NET ALLOWABLE TANK DISCHARGE (2-Year Pre-Development Allowable Discharge to Byngmount - 100-Year Un-Controlled Discharge to Byngmount)																			
									35.3	←	Allowable Discharge to Byngmount Storm Sewer								
									9.2	←	100-Year Un-Controlled Discharge from Subject Site								
									26.1	←	Net Allowable SWM Tank Discharge								
POST DEVELOPMENT CONDITIONS (WITH ATTENUATION)					K	Orif.	Area	head (m)	Q (L/s)										
100-YEAR	Parking Lot CB	Building			k=0.8	100	0.00785	1.42	**33.2**	250	0.60%	46.1	0.94	1.02	27.0	0.5	15.5	72%	On-site Sewer
100-YEAR	Control MH	Prop. MH1			k=0.6	99	0.0077	1.61	**25.9**	250	1.00%	59.5	1.21	1.17	14.6	0.2	15.2	44%	Service
											Okay. Site Discharge to Byngmount is less than allowable.								
FUTURE BYNGMOUNT EXTENSION																			
External Area North of Site			0.1660	0.38	0.063	0.063	15.0	99.2	21.7										
Un-Controlled Area West of Site			0.0752	0.25	0.019	0.019	15.0	99.2	6.5										
Future Byngmount Avenue			0.2450	0.70	0.172	0.172	15.0	99.2	59.1										
			0.4862	0.52	0.253				**87.2**										
10- Yr Controlled Subject Site									23.8										
Ext. Area + Site + Future Extension	Prop. MH1	Exist. MH1							**111.0**	375	0.65%	141.4	1.28	1.41	74.8	1.0	16.0	79%	Public Sewer

958-960 East Avenue

Stormwater Management Storage Calculations using Rational Method

City of Mississauga

Roof Storage

$$I_{100\text{-year}} = \frac{1450}{(T+4.9)^{0.78}} = 140.69 \text{ mm/hr}$$

Project No.	19002	Building Roof Area (ha) =	0.22090
Analysis By:	Jason Jenkins	Weighted Runoff Coefficient =	0.90
Last Revised:	10 March 2020	Roof Discharge (L/s) =	2.7

Time (min)	Intensity (mm/hr)	Q-100 (L/s)	Q-stored (L/s)	Storage Vol. (cu.m)
0	0.0	0.000	0.000	0.000
15	140.7	77.696	75.013	67.512
20	118.1	65.233	62.551	75.061
30	90.8	50.130	47.448	85.406
40	74.6	41.186	38.504	92.409
50	63.8	35.208	32.525	97.575
60	56.0	30.900	28.217	101.581
70	50.0	27.632	24.949	104.786
80	45.4	25.059	22.376	107.405
90	41.6	22.974	20.291	109.573
100	38.5	21.247	18.564	111.387
110	35.8	19.790	17.108	112.911
120	33.6	18.543	15.861	114.196
130	31.6	17.462	14.779	115.279
140	29.9	16.515	13.832	116.190
150	28.4	15.677	12.994	116.950
160	27.0	14.930	12.248	117.579
170	25.8	14.260	11.578	118.092
180	24.7	13.655	10.972	118.502
190	23.7	13.105	10.423	118.820
200	22.8	12.604	9.921	119.054
210	22.0	12.144	9.461	119.213
220	21.2	11.721	9.038	119.302
230	20.5	11.330	8.647	119.329
240	19.9	10.967	8.285	119.297
250	19.2	10.630	7.947	119.212
260	18.7	10.316	7.633	119.077
270	18.1	10.022	7.339	118.896
280	17.6	9.746	7.064	118.672
290	17.2	9.488	6.805	118.407
300	16.7	9.244	6.561	118.106
310	16.3	9.014	6.332	117.769
320	15.9	8.797	6.114	117.398
330	15.6	8.592	5.909	116.997
340	15.2	8.397	5.714	116.566
350	14.9	8.211	5.529	116.106
360	14.6	8.035	5.353	115.621

Storage Volume Required (cu.m)	=	**119.3**
Storage Volume Provided (cu.m)	=	**119.3**
Depth (mm)	=	54
Number of Drains	=	4

958-960 East Avenue

Stormwater Management Storage Calculations using Rational Method

City of Mississauga

Parking Lot

$$I_{100\text{-}year} = \frac{1450}{(T+4.9)^{0.78}} = 140.69 \text{ mm/hr}$$

Project No.	19002	Building Roof Area (ha) =	0.24650
Analysis By:	Jason Jenkins	Weighted Runoff Coefficient =	0.90
Last Revised:	10 March 2020	CB Orifice Discharge (L/s) =	33.2

Time (min)	Intensity (mm/hr)	Q-100 (L/s)	Q-stored (L/s)	Storage Vol. (cu.m)
0	0.0	0.000	0.000	0.000
15	140.7	86.700	53.536	48.182
20	118.1	72.793	39.628	47.554
30	90.8	55.940	22.775	40.996
40	74.6	45.959	12.795	30.707
50	63.8	39.288	6.123	18.370
60	56.0	34.481	1.316	4.738
70	50.0	30.834	0.000	0.000
80	45.4	27.963	0.000	0.000
90	41.6	25.636	0.000	0.000
100	38.5	23.709	0.000	0.000
110	35.8	22.084	0.000	0.000
120	33.6	20.692	0.000	0.000
130	31.6	19.486	0.000	0.000
140	29.9	18.429	0.000	0.000
150	28.4	17.494	0.000	0.000
160	27.0	16.661	0.000	0.000
170	25.8	15.913	0.000	0.000
180	24.7	15.238	0.000	0.000
190	23.7	14.624	0.000	0.000
200	22.8	14.064	0.000	0.000
210	22.0	13.551	0.000	0.000
220	21.2	13.079	0.000	0.000
230	20.5	12.643	0.000	0.000
240	19.9	12.238	0.000	0.000
250	19.2	11.862	0.000	0.000
260	18.7	11.511	0.000	0.000
270	18.1	11.183	0.000	0.000
280	17.6	10.876	0.000	0.000
290	17.2	10.587	0.000	0.000
300	16.7	10.315	0.000	0.000
310	16.3	10.059	0.000	0.000
320	15.9	9.817	0.000	0.000
330	15.6	9.587	0.000	0.000
340	15.2	9.370	0.000	0.000
350	14.9	9.163	0.000	0.000
360	14.6	8.967	0.000	0.000

Max. HGL = 81.67

Top CB = 81.45

CB Outlet = 80.20

Orifice (mm) = 100

Driving Head (m) = 1.42

Storage Volume Required (cu.m) = **48.2**

Storage Volume Provided (cu.m) = **49.6**

Depth of Surface Ponding (m) = 0.22

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958-960 East Avenue

Stormwater Management Storage Calculations using Rational Method
City of Mississauga

SWM Tank

$$I_{100\text{-year}} = \frac{1450}{(T+4.9)^{0.78}} = 140.69 \text{ mm/hr}$$

Project No.	19002	Site Area (Less Controlled Areas), (ha) = 0.2000
Analysis By:	Jason Jenkins	Weighed Runoff Coefficient (Adjusted) = 0.820
Last Revised:	10 March 2020	Peak Discharge (L/s) = 25.9

Time (min)	Intensity (mm/hr)	Q-100 (L/s)	Q-Roof (L/s)	Q-Parking (L/s)	Q-stored (L/s)	Storage Vol. (m^3)
0	0.0	0.000	0.000	0.000	0.000	0.000
15	140.7	64.089	2.683	33.165	74.000	66.600
20	118.1	53.809	2.683	33.165	63.720	76.464
30	90.8	41.351	2.683	33.165	51.262	92.272
40	74.6	33.973	2.683	33.165	43.885	105.323
50	63.8	29.042	2.683	33.165	38.953	116.859
60	56.0	25.488	2.683	33.165	35.399	127.438
70	50.0	22.793	2.683	30.834	30.373	127.568
80	45.4	20.670	2.683	27.963	25.379	121.821
90	41.6	18.951	2.683	25.636	21.334	115.202
100	38.5	17.526	2.683	23.709	17.982	107.893
110	35.8	16.324	2.683	22.084	15.155	100.024
120	33.6	15.296	2.683	20.692	12.735	91.690
130	31.6	14.404	2.683	19.486	10.636	82.964
140	29.9	13.622	2.683	18.429	8.798	73.902
150	28.4	12.932	2.683	17.494	7.172	64.549
160	27.0	12.316	2.683	16.661	5.723	54.942
170	25.8	11.763	2.683	15.913	4.423	45.110
180	24.7	11.264	2.683	15.238	3.248	35.078
190	23.7	10.810	2.683	14.624	2.181	24.866
200	22.8	10.396	2.683	14.064	1.208	14.493
210	22.0	10.017	2.683	13.551	0.315	3.972
220	21.2	9.668	2.683	13.079	0.000	0.000
230	20.5	9.345	2.683	12.643	0.000	0.000
240	19.9	9.046	2.683	12.238	0.000	0.000
250	19.2	8.768	2.683	11.862	0.000	0.000
260	18.7	8.509	2.683	11.511	0.000	0.000
270	18.1	8.267	2.683	11.183	0.000	0.000
280	17.6	8.040	2.683	10.876	0.000	0.000
290	17.2	7.826	2.683	10.587	0.000	0.000
300	16.7	7.625	2.683	10.315	0.000	0.000
310	16.3	7.436	2.683	10.059	0.000	0.000
320	15.9	7.256	2.683	9.817	0.000	0.000
330	15.6	7.087	2.683	9.587	0.000	0.000
340	15.2	6.926	2.683	9.370	0.000	0.000
350	14.9	6.773	2.683	9.163	0.000	0.000
360	14.6	6.628	2.683	8.967	0.000	0.000

Volume Required (cu.m) =	**127.6**
Volume Provided (cu.m) =	**130.9**
HGL Depth in Tank (m) =	1.7
Orifice Size (mm) =	99

958-960 East Avenue
Water Quality, and Water Balance
City of Mississauga

Designed By: **Jason Jenkins**, P.Eng., P.E.
Checked By: Robert Filipuzzi, P.Eng.
File No. 19002
Date: 10 March 2020

Water Quality (TSS Removal)

TSS Removal				
Roof Bare	1,398	18.4%	95	17.5
Green Roof	811	10.7%	100	10.7
Landscape	1,797	23.7%	100	23.7
Permeable	1,147	15.1%	80	12.1
Hard Surface*	2,425	32.0%	80	25.6
	7,578	100%		89.7

**All areas to be directed toward permeable pavers for water quality*

Water Balance

Volume Required	
Required Water Balance (mm):	5.0
Site Area (m^2):	7578.0
Required Water Balance Volume (m^3):	37.9

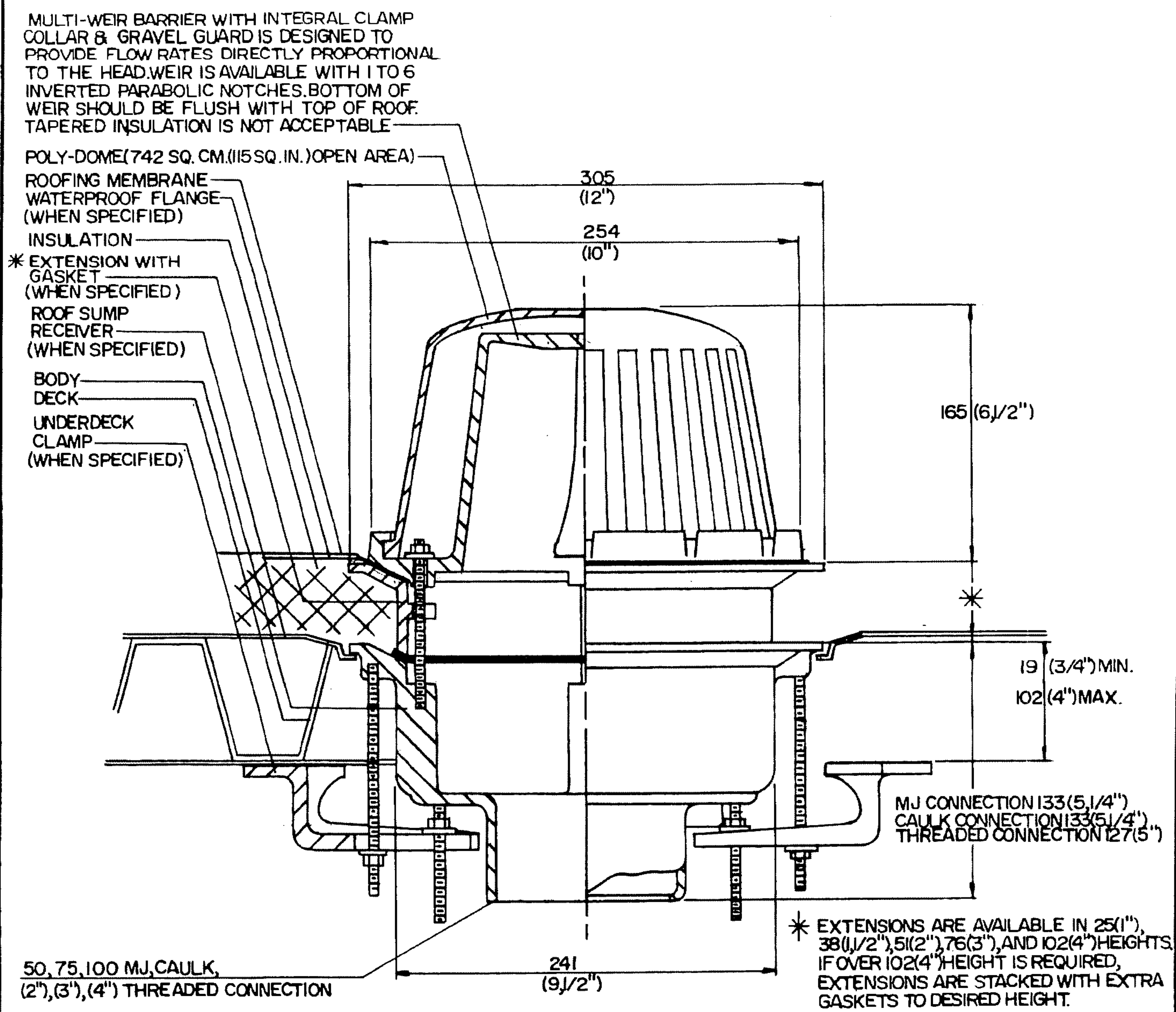
Initial Abstraction				
Roof Bare	1,398	18.4%	1	0.2
Green Roof	811	10.7%	5	0.5
Landscape	1,797	23.7%	5	1.2
Permeable	1,147	15.1%	5	0.8
Hard Surface*	2,425	32.0%	1	0.3
	7,578	100%		3.0

Infiltration Gallery	
Area of Infiltration Gallery (m^2):	135.0
Depth of Stone (m):	0.30
Void Ratio:	0.40
Volume Retained in Stone (m^3):	16.2

Water Re-Use		
Area of SWM Tank (m^2):	77.0	
Depth of Sump (m):	0.30	
Volume Retained in Tank (m^3):	23.1	Water Re-Use Volume to be used for irrigation

Total Volume Retained		
Initial Abstraction + Infiltration + Water Re-Use (m^3):	42.3	Water Balance Volume is Satisfied.

Volume Retained > Volume Required
42.3 > 37.9



MAY 21 '81 LM REV2 OCT.25'85 LM 5/95 BC

FURNISHED WHEN SPECIFIED

- ☐ -ZG GALVANIZED
- ☐ -C UNDERDECK CLAMP
- ☐ -E EXTENSION (SPECIFY HEIGHT REQUIRED)
- ☐ -R ROOF SUMP RECEIVER
- ☐ -A WATERPROOF FLANGE
- ☐ -EB ELEVATING BODY PLATE
- ☐ -SSM STAINLESS STEEL MESH OVER DOME
- ☐ -SS STAINLESS STEEL DRAINAGE GRID
- ☐ -ALUMINUM DOME
- ☐ -LONGER UNDERDECK CLAMP STUDS (SPECIFY LENGTH REQUIRED)
- ☐ -SPECIFY NUMBER OF NOTCHES REQUIRED IN WEIR IF SINGLE NOTCH NOT SUITABLE

REGULARLY FURNISHED

* DURA-COATED CAST IRON BODY, 152(6")HIGH SINGLE NOTCH ALUMINUM CONTROL FLO WEIR WITH INTEGRAL CLAMP COLLAR AND GRAVEL GUARD, HARDWARE, AND POLY-DOME.

FUNCTION - RECOMMENDED FOR INSTALLATION ON FLAT OR SLOPED ROOFS, WHERE CONTROL OF RAINWATER RUN-OFF IS REQUIRED.

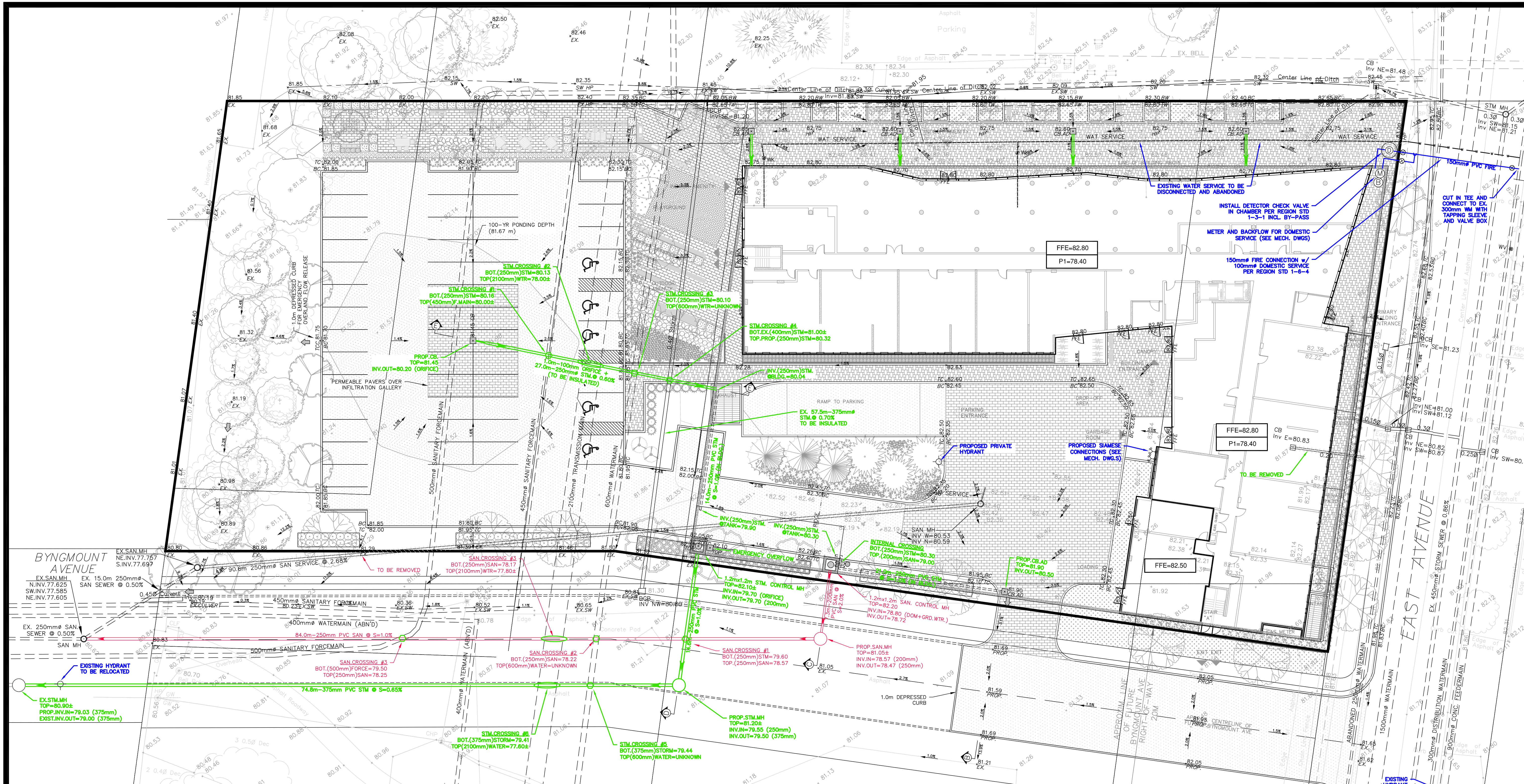
NOTE
- BODY DECK OPENING 292 (11,1/2") DIAMETER
- IF ROOF SUMP RECEIVER IS USED, DECK OPENING 330 (13") DIAMETER.

*D.C. (DURA-COATED) OVEN CURED, FUSION BONDED EPOXY POWDER COATING

DIMENSIONS ARE IN INCHES (BRACKETS) AND TO THE NEAREST MILLIMETRE.

'CONTROL FLO' ROOF DRAIN FOR LEVEL OR SLOPED ROOF	PRODUCT NUMBER ZCF-121
ZURN — ZURN INDUSTRIES LIMITED	DRAWING NUMBER 0013

APPENDIX F



LEGEND

AD.1	PROP AREA DRAIN	FFE=93.74	PROP FINISHED FIRST FLOOR ELEV.
CB.1	PROP CATCHBASIN	P1=93.37	PROP FINISHED P1 ELEV.
MH.1	PROP STORM MANHOLE		PROP VALVE AND BOX
MH.1A	PROP SANITARY MANHOLE		PROP WATER SERVICE CONNECTION
	PROP HYDRANT		PROP SANITARY SEWER / SERVICE
			PROP STORM SEWER / SERVICE

DWN. BY:	J.M.J.
DESIGNED BY:	J.M.J.
CHECKED BY:	R.D.F.
SCALE:	NTS
DATE:	MARCH 2020
SHEET NO:	1 OF 3

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A Division of FP&P HydraTek Inc.
216 Chrislea Road, Suite 204
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t: 905-264-2420
www.fabianpapa.com

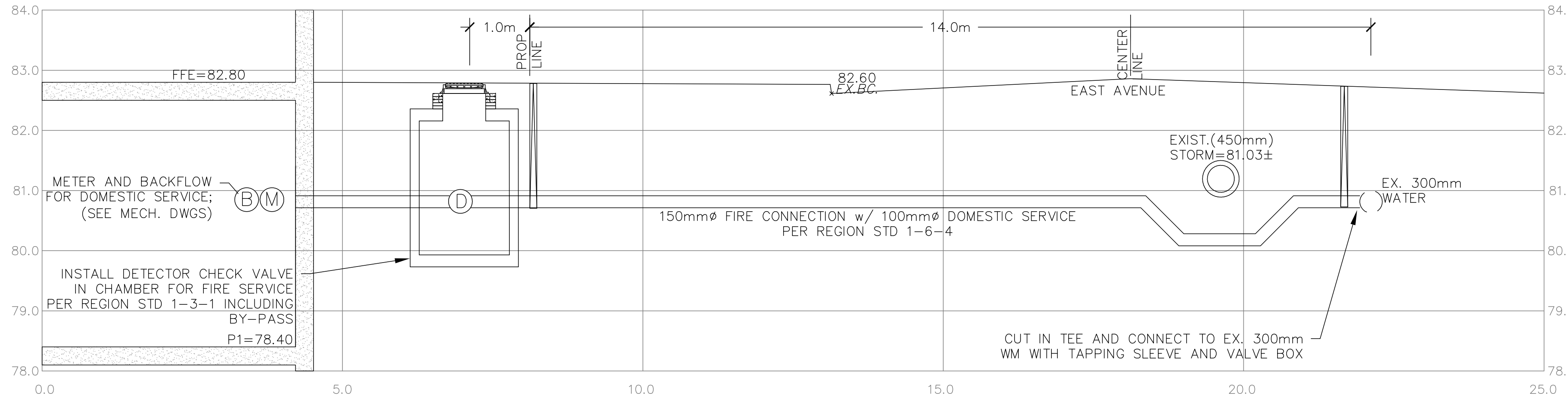
PROJECT NAME:

958-960 EAST AVENUE RESIDENTIAL DEVELOPMENT

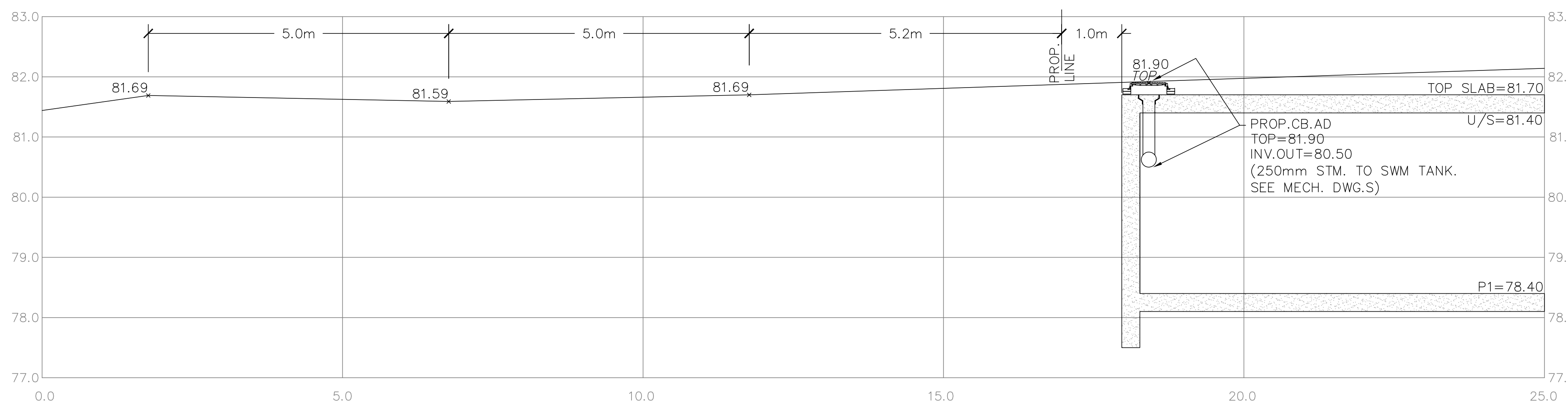
DRAWING TITLE:

SITE SERVICING AND GRADING EXHIBIT

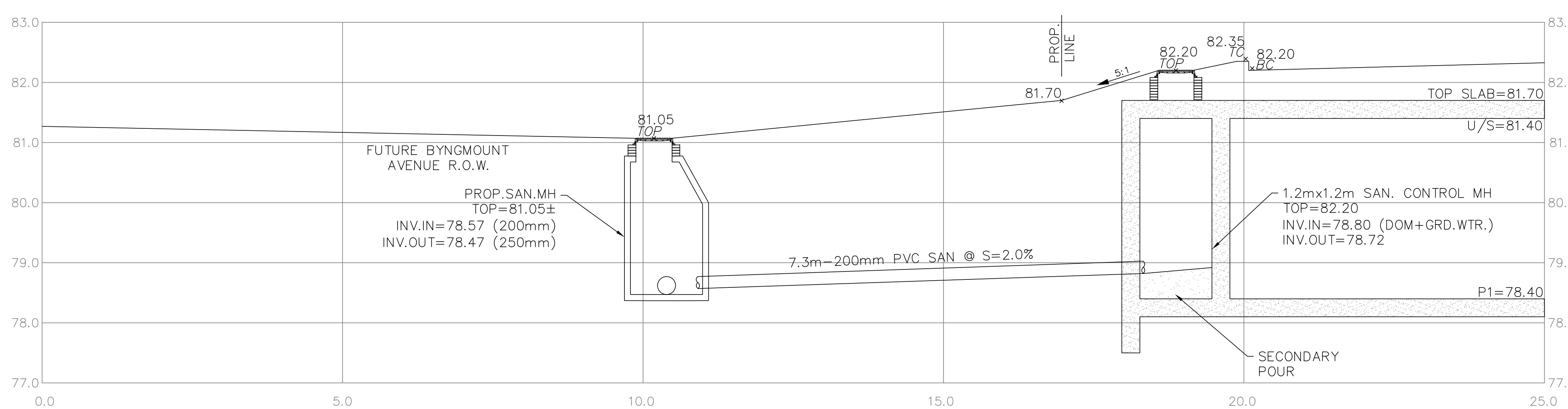
PROJECT NO:	DWG NO:	REV NO:
19002	SSG-1	0



SECTION A: WATER CONNECTION



SECTION B: DRIVEWAY / HEADROOM



SECTION C: SANITARY CONNECTION

PROJECT NAME:
958-960 EAST AVENUE
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:
SITE SERVICING SECTIONS

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REV NO: 0

DWG NO: DET-1

PROJECT NO: 19002

DWN. BY: J.M.J.

DESIGNED BY: J.M.J.

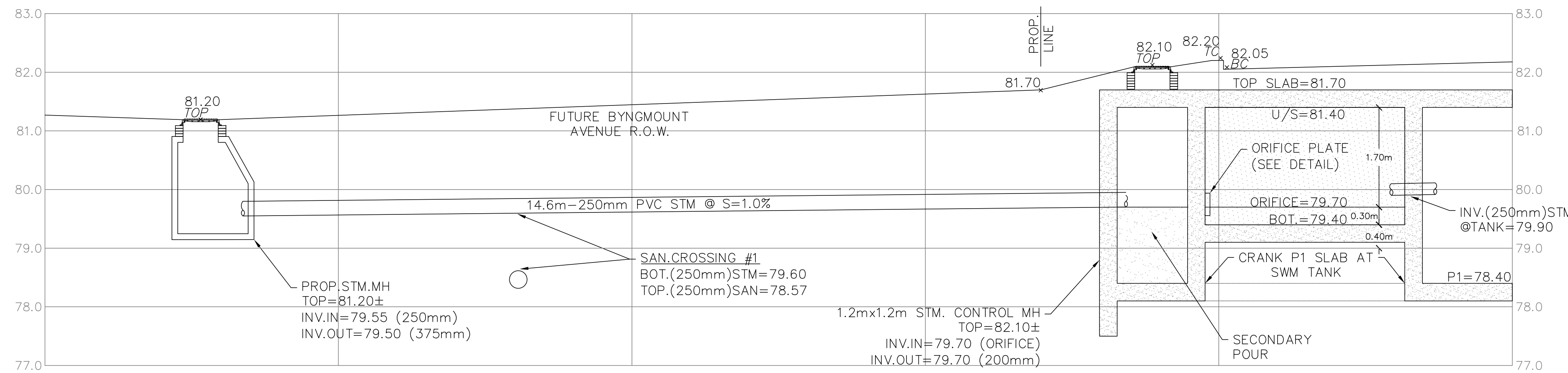
CHECKED BY: R.D.F.

SCALE: NTS

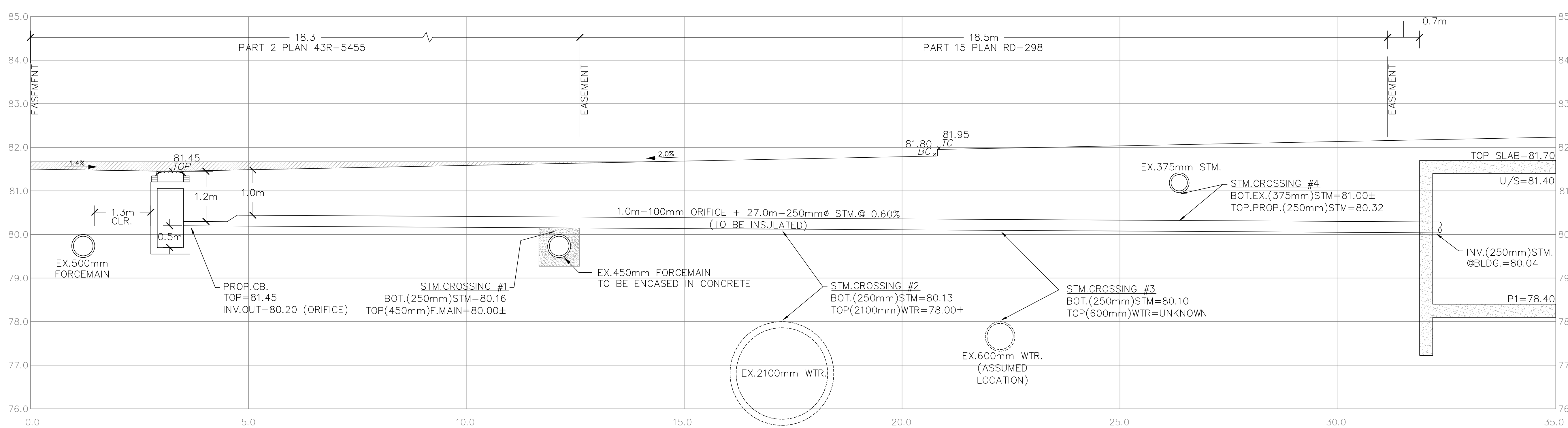
DATE: MARCH 2020

SHEET NO: 2 OF 3

H:\FP&P HydraTek\Projects\2019\19002 - 958-960 East Avenue, Mississauga\Re-Zoning\Revision 0\19002 - Civil Plans (Revision 0, Draft)v2.dwg (Mar 10, 2020 - 3:47pm)



SECTION D: STORM CONNECTION AND SWM TANK



SECTION E: EASEMENT CROSSINGS

LEGEND

Symbol	Description
AD.1	PROP AREA DRAIN
CB.1	PROP CATCHBASIN
MH.1	PROP STORM MANHOLE
MH.1A	PROP SANITARY MANHOLE
	PROP HYDRANT
FFE=93.74	PROP FINISHED FIRST FLOOR ELEV.
P1=93.37	PROP FINISHED P1 ELEV.
	PROP VALVE AND BOX
	PROP WATER SERVICE CONNECTION
	PROP SANITARY SEWER / SERVICE
	PROP STORM SEWER / SERVICE

DWN. BY:	J.M.J.
DESIGNED BY:	J.M.J.
CHECKED BY:	R.D.F.
SCALE:	NTS
DATE:	MARCH 2020
SHEET NO:	3 OF 3

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PROJECT NO:	DWG NO:	REV NO:
19002	DET–2	0

PROJECT NAME:

958–960 EAST AVENUE
RESIDENTIAL DEVELOPMENT

DRAWING TITLE:

SITE SERVICING SECTIONS