Functional Servicing & Stormwater Management Report

Mixed-Use Development

2026 Queen Street East

City of Toronto

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APPENDICES

1.0 INTRODUCTION

fabian papa & partners has been retained by Crombie REIT to prepare this Functional Servicing & Stormwater Management Report in support of the Zoning By-Law Amendment application for the subject property. This report discusses the provision of municipal services for the development proposal, including the stormwater management servicing strategy.

The subject site consists of the properties municipally known as 2026-2040 Queen Street East (M4L 1J4), in the City of Toronto. The site is located on the northwest corner of Queen Street East and Lee Avenue. Currently, the site consists of various 2 storey commercial/residential building units, with vehicular access from Lee Avenue. The total area of the subject site is approximately 1,854 m² (0.185 ha) and is bound by Queen Street East to the south, Lee Avenue to the east, and a commercial/residential building to the west and south. A key map and aerial image of the site can be found in Appendix A for reference.

The development proposal envisions the construction of a 6-storey mixed-use building consisting of 60 residential units, an at-grade retail space, and 1 level of underground parking. Vehicular access to the underground parking will be from a private driveway along the north side of the site, which can be accessed from Lee Avenue at the northeast corner of the site. An excerpt of the architectural plans for the proposed development can be found in Appendix A for reference.

2.0 WATER SUPPLY

2.1 Overview

The existing municipal water infrastructure surrounding the subject property consists of a 300 mm diameter watermain which runs along the north side of Queen Street East, a 1050 mm diameter watermain on the south side of Queen Street East, and a 150 mm diameter watermain along the west side of Lee Avenue. Pertinent information has been included on the Site Servicing & Grading Schematic, and excerpt copies of the plan and profile drawings can be found in Appendix B for reference. In accordance with City policies, all existing water services to the property will be disconnected from the main and abandoned. A new water service will be installed to current City standards.

To confirm that the existing water supply infrastructure has adequate pressure to accommodate the proposed building, a hydrant flow test Queen Street East was commissioned and performed on 24 October 2022 at the existing fire hydrant adjacent to the site at 2036 Queen Street East (residual pressures were measured at the hydrant in front of 2000 Queen Street East). A copy of the test report is provided in Appendix C while the results are summarized as follows:

Hydrant Flow Test Results

Flow (usgpm)	0 (Static)	856	1506	2536
Flow (L/s)	0 (Static)	54	95	160
Pressure (psi)	84	82	78	68
Pressure (kPa)	577	562	535	467

2.2 Supply Demands

The domestic water demand was calculated based on the City of Toronto design criteria using the following parameters.

Design Flow:	190 Lpcd	
Population Density:	1-Bed	1.4 persons per unit
	2-Bed	2.1 persons per unit
	3-Bed	3.1 persons per unit
Peak Hour Factor:	2.5 for residential and 1.2 for commercial	
Max Day Factor:	1.3 for residential and 1.1 for commercial	

The detailed demand calculations can be found in Appendix C. The domestic demand is summarized in the following table.

Domestic Water Supply Demand

Building	Average Day Demand, ADD (L/s)	Peak Hour Demand, PHD (L/s)	Maximum Day Demand, MDD (L/s)
Residential	0.23	0.58	0.3
Retail	0.03	0.03	0.03
Total	0.26	0.61	0.33

The recommended fire flow demand for the building is calculated using the criteria outlined in the Water Supply for Public Fire Protection Manual, 2020, by the Fire Underwriters Survey. Appropriate reductions and increases have been applied to the calculation as follows:

Fire Underwriters Survey Coefficients

Construction Coefficient	0.6 (fire resistive construction)
Building Occupancy	-15% (limited-combustible)
Fire Suppression System	-40% (standard, adequately designed sprinkler protection system)
Exposure / Proximity	+75%

The detailed fire flow calculations, which can be found in Appendix C, are based on the largest floor area, and a calculation is provided below:

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

$A = 1{,}372\ m^2 + [(1{,}112\ m^2 + 1{,}337\ m^2) \times 0.25] = 1{,}984\ m^2$

$F = 220 \times C \times A^{0.5} = [220 \times 0.6 \times (1{,}984\ m^2)^{0.5}] = 5{,}880$ L/min

F = 6,000 L/min (rounded to nearest 1,000)

Fire Flow = 6,000 × (1 - 0.15) – 5,100 × 0.40 + 5,100 × 0.75 = 6,885 L/min

Fire Flow = 7,000 L/min (rounded to nearest 1,000)

Fire Flow = 116.7 L/s

The design flows applied in the design of the service connections to the property are as follows:

- Domestic supply lines (PHD): 0.61 L/s
- Fire supply line (MDD + Fire): 0.33 + 116.7 = 117.0 L/s

2.3 Proposed Connections and Layout

Based on the above demands, a single 150 mm diameter water service is proposed to be connected to the existing 300 mm diameter municipal watermain on Queen Street East. The connection will branch to a 100 mm diameter domestic supply line and a 150 mm fire supply line within the City right-of-way before crossing the property line. The valves and boxes will be installed directly adjacent to the property line. The meters and back-flow preventers will be installed within the mechanical room on the P1 parking level which is less than 30 m from the streetline.

The Ontario Building Code requires that any building above 84 m in height shall be protected by two independent fire service connections separated by an isolation valve. Since the top of the highest occupied floor of the proposed building is 20.25 m±, one fire service connection is sufficient. However, since the building is less than 23 m, it is not considered a high rise per the NFPA 14 and therefore only one siamese connection is required. The Siamese is located adjacent to the residential entrance on the west side of the building along Queen Street East.

The nearest hydrant to the site is located adjacent to the site on the north side of Queen Street East, and within 45 m of the siamese connection. Furthermore, the existing municipal hydrant is within 90 m to all building faces with municipal frontage, thereby satisfying the requirements of the Ontario Building Code. Refer to the Site Servicing Schematic (found in Appendix F) for the location of the proposed service connections and existing hydrant.

2.4 Domestic and Fire Flow Analysis

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

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

Head Loss & Residual Pressure Summary Table

Proposed Service Connection	Flow (L/s)	Head Loss, psi (kPa)	Residual pressure at main, psi (kPa)	Residual pressure at Building, psi (kPa)
100 mm Domestic (PHD)	0.6	0.0 (0.0)	**84** (577)	**84** (577)
150 mm Fire (Fire + MDD)	117.0	0.9 (6.4)	**74** (512)	**73** (506)

The calculations above show that the residual pressures at the building face are above the City's minimum acceptable pressures of 40 psi (275 kPa) for PHD and 20 psi (140 kPa) for MDD + Fire demand conditions.

Therefore, the existing municipal water infrastructure and the proposed service connections can support the development without any upgrades.

3.0 FOUNDATION DRAINAGE

A hydrogeology assessment, prepared by Pinchin Ltd., was completed to assess the existing groundwater levels in relation to the proposed development excavation and underside of footings both for short-term (construction dewatering) and long-term (permanent foundation drainage) conditions.

However, a decision has been made by the developer to install a watertight foundation drainage system. Therefore, the proposed development will not be discharging any permanent foundation or groundwater drainage into the City's sewer network. As such, only a short-term (construction) discharge rate will be considered. The short-term discharge rate expected during construction is estimated to be 92 m^3/day (1.06 L/s). Excerpts of the Hydrogeological Assessment can be found in Appendix G for reference.

Based on these estimated inflow rate, the short-term pumped rate is estimated to be a maximum 20 usgpm (1.26 L/s) and the actual flow will be determined as part of the shoring design and permit process. Notwithstanding this, the calculated sanitary flow exceeds the short-term groundwater pump rate and therefore the former will be carried as the total site discharge to the sanitary control manhole.

It was determined that the quality of the groundwater meets the sanitary and combined sewer parameters for discharge (per the City's Municipal Code 681) without any exceedances, therefore the construction dewatering will be metered and then discharged to the 350 mm diameter combined sewer on the north side of Queen Street East. Details will be provided by the dewatering contractor prior to the execution of a sewer discharge agreement.

4.0 SANITARY DRAINAGE

4.1 Overview

Sanitary infrastructure adjacent to the site consists of a 350 mm diameter combined sewer along the north boulevard of Queen Street East (±4.0m depth), draining westerly toward Bellefair Avenue where it connects to the 300 mm combined sewer on the south side of Queen Street East and continues to drain westerly. There is a 300mm combined sewer across the frontage of the site on the south side of Queen Street East draining westerly that eventually collects the 300 mm diameter sewers from the north side of the street and Bellefair Avenue and discharges into a 450 mm combined sewer draining south on Kenilworth Avenue (approx. 300m west of the site). In addition, there is a 1350 mm combined sewer which also runs along the north side of Queen Street East that drains westerly past Kenilworth Avenue to a different 2100 mm diameter combined sewer system.

After detailed review of City of Toronto plan and profile data, it notes that the existing drain connections for the buildings on the site are serviced by the 350 mm combined sewer on Queen Street East. In addition, there is a small portion of driveway sheet drainage (approx. 34 m^2) from the site that discharges onto Lee Avenue. This drainage from the site eventually goes into the storm sewer on Lee Avenue. As such, the total area contributing stormwater drainage to the 350 mm combined sewer system is 1,820 m^2 (1,854 m^2 - 34 m^2). Please refer to the Pre-Development Storm Drainage Plan in Appendix E for more details.

In accordance with City policies, all existing sanitary services to the property will be disconnected from the main and abandoned. A new sanitary service will be installed and connected to the 350 mm combined sewer to current City standards.

4.2 Sanitary Design Flow

The pre- and post-development sanitary flow from this site is calculated in accordance with the current City of Toronto Design Criteria.

Design Flow:	240 Lpcd (for existing residential populations) 250 Lpcd (for industrial/commercial populations) 450 Lpcd (for proposed residential developments)
Infiltration Flow:	0.26 L/s/ha (for dry weather flow)
Peaking Factor:	Calculated using the Harmon Formula
Population Densities:	1.4 ppu (1-Bedroom Unit) 2.1 ppu (2-Bedroom Unit) 3.1 ppu (3-Bedroom Unit)

The pre-development flow is calculated as follows:

$$Q_{SAN(PRE)} = \left(\frac{250\text{ Lpcd} \times 31\text{ persons}}{86{,}400\text{ s / day}}\right) + 0.26\ \text{ L/s/ha} \times 0.1854\text{ ha} = 0.1\text{ L/s}$$

Using the City's population densities, and the proposed unit count for the 60 units shown on the architectural plans, the total post-development sanitary flow is calculated as follows:

For Sewer Design

$$Q_{SAN\ (POST)} = \left(\frac{450\text{ Lpcd} \times 118\text{ persons} \times 4.22_{\text{Peaking}}}{86{,}400\text{ s / day}}\right) + 0.26\ \text{ L/s/ha} \times 0.1854\text{ ha} = 2.6\text{ L/s}$$

For Receiving Sewer Capacity Analysis

$$Q_{SAN\ (POST)} = \left(\frac{240\text{ Lpcd} \times 118\text{ persons} \times 4.22_{\text{Peaking}}}{86{,}400\text{ s / day}}\right) + 0.26\ \text{ L/s/ha} \times 0.1854\text{ ha} = 1.5\text{ L/s}$$

Based on these criteria, the total pre-development sanitary sewer flow for the site is calculated to be 0.1 L/s, and the post-development sanitary flow for the site is calculated to be 1.5 L/s. Please refer to Appendix D for the detailed design flow calculations.

4.3 MECP Procedure F-5-5 Compliance

In accordance with Ministry of the Environment, Conservation and Parks (MECP) Procedure F-5-5, any developments serviced by a combined sewer must demonstrate the following: (i) elimination of combined sewer overflows (CSO)'s during dry weather flow (i.e., no surcharge allowed); and (ii) demonstration that there is no net increase in total flow from the site to a combined sewer system under post-development wet-weather flow conditions.

As noted in the previous sections, the proposed building is tributary to the 350 mm diameter combined sewer on Queen Street East from which the existing buildings are currently serviced.

The corresponding 2-year pre-development peak storm flow (to Queen Street East) is calculated as follows:

$$Q_{2\text{-Yr Pre}} = \frac{(A \times R) \times I_2}{360} = \frac{(0.1820 \times 0.90) \times 88.2}{360} \times 1000 = 40.1 \text{ L/s}$$

The following table summarizes the site's total discharge to the combined sewer system under pre- and post-development conditions:

Table 1: Site Discharge to Existing Combined Sewer

	Sanitary Flows	Foundation Discharge	2-year Storm (0.90 Coefficient)	Controlled 100- to 2-Year Storm (0.5 Coefficient + Orifice)	Total
Pre-Development (Actual)	0.1 L/s	---	40.1 L/s[1]	---	40.2 L/s
Post-Development (Proposed)	1.5 L/s	---	---	20.7 L/s	22.2 L/s

Based on the above, the proposed development adheres to the MECP Procedure F-5-5 requirements for wet-weather flow.

4.4 Municipal Service Connection

The subject site will be serviced by a 150 mm sanitary service which will be connected to the existing 350 mm combined sewer within the north boulevard of Queen Street East.

Based on drawing records received from the City of Toronto, the invert of the existing 350 mm combined sewer at the proposed sanitary service connection is approximately 79.84 m. The proposed sanitary service will be connected to the existing sewer at an invert of 80.25 m plus a riser, and be constructed a 2.0% gradient resulting in an invert at the streetline of 80.32 m.

A 1.2m x 1.2m cast-in-place control manhole will be located at the property line, within the building footprint of the P1 level to allow for 24/7 inspection and monitoring thus satisfying the City's requirements. The connection will convey the sanitary flows from the building operating at 12% of full flow capacity (22.5 L/s).

The proposed connection will have adequate depth to service the ground floor (and above) of the development, however the basement/parking level will require a grinder pump to discharge to the connection. To prevent backup of sewage into the basement/parking levels, we recommend that the mechanical consultant adequately design the internal system to operate under and withstand the potential for a surcharged municipal system.

Based on the discussion in the previous sections, the proposed development can be adequately serviced from a sanitary sewerage perspective by the existing municipal infrastructure without the need for any upgrades or system modifications (subject to the completion of the downstream hydraulics grade line analyses). The location of the existing and proposed infrastructure is shown on the Site Servicing and Schematic found in Appendix F.

[1] Roof area and most of the driveway area is connected to the combined sewer with the exception of a small area draining to the Lee Avenue storm sewer. Using an area of 1,820 m² and C=0.90, the 2-year pre-development flow is calculated as 40.1 L/s; refer to Section 3.0 and the Storm Sewer Design Sheet in Appendix E.

5.0 STORM DRAINAGE

5.1 Overview

Local municipal storm infrastructure adjacent to the subject site consists of a 450 mm diameter storm sewer on Lee Avenue that drains in a southerly direction, and ultimately connects into a 525 mm storm sewer that continues south on Lee Avenue.

5.2 Design Criteria

The stormwater management servicing strategy proposed for the development has been prepared in conjunction with City design standards and the Wet Weather Flow Management Guidelines (WWFMG). The relevant criteria are summarized below:

Water Quantity Management

- The allowable release rate from the developed site to the existing municipal storm sewer (minor system) during the occurrence of a 2-year storm event must not exceed the runoff rate equivalent to the peak runoff rate achieved by the site under pre-development flow conditions during the occurrence of a 2-year storm event (*Discharge Criteria to Municipal Infrastructure, Section 2.2.3.7 of the WWFMG*).

- Runoff which exceeds the allowable release rate defined above is allowed to discharge off-site via the overland flow route if a suitable overland flow route, of sufficient hydraulic capacities (up to a 100-year storm) and deemed acceptable to the City, exists. If no approved or adequate overland flow route exists, runoff generated by storms up to and including the 100-year event must be contained on-site and released at the allowable release rate defined above (*General Guidelines for On-Site Storage, Section 2.2.3.8(3) of the WWFMG*).

- An overland flow route (major system) shall be provided within the developed site to direct runoff in excess of the 100-year storm runoff to an approved overland flow outlet (*General Guidelines for Suitable Overland Flow, Section 2.2.3.8(4) of the WWFMG*).

Water Quality Management

- All runoff from the site shall achieve a long-term average removal of 80% of Total Suspended Solids (TSS) on an annual loading basis to meet the water quality targets (*TSS Removal Targets, Section 2.2.2.1 of the WWFMG*).

Water Balance Management

- To achieve the water balance targets, a minimum of the first 5 mm from each rainfall event must be retained on-site for rainwater reuse, infiltration and evapotranspiration (*On-Site Stormwater Retention, Section 2.2.1.1 of the WWFMG*).

5.3 Pre-Development Conditions

The subject site currently consists of various 2 storey commercial/residential building units, with vehicular access from Lee Avenue with a combined site area of 0.185 ha.

Based on our review of the topographic survey of the property and historical plan and profile documents, we assume that the service connections for the buildings were combined (i.e., collected stormwater from internal roof downspouts, catchbasins and surface area drains, sanitary and foundation drainage in a single service). The majority of the site's existing storm discharge is still draining directly to the combined sewer system.

As mentioned previously, all of the storm drainage from the site discharges to the 350 mm combined sewer on Queen Street East with the exception of the small uncontrolled drainage area that discharges onto Lee Avenue. It was determined that a connection to the existing 450 mm storm sewer on Lee Avenue would not be feasible for adequate drainage of the site, therefore, the proposed stormwater connection will be connected to the existing 350 mm combined sewer on Queen Street East. Since the pre-development conditions also discharge stormwater to the 350 mm combined sewer, post-development stormwater drainage will be reduced because it will be released at a controlled rate, thus improving the overall drainage conditions of the site. The existing drainage patterns and runoff coefficients are illustrated on the Pre-Development Drainage Area Schematic (STM-1) provided in Appendix E for reference.

The pre-development weighted runoff coefficient for the entire subject property is estimated to be 0.90 (refer to the Pre-Development Drainage Area Plan and the weighted runoff calculations in Appendix E), however, since the WWFMG limits the pre-development coefficient to a maximum of 0.50, then this value is what governs for estimating target post-development release rates to the storm sewer system. It is assumed that the existing municipal sewer network was designed to convey the 2-year return period design storm. Any storms greater than the 2-year event are assumed to be directed uncontrolled via overland sheet drainage to Queen Street East. On-site storage was not previously provided because it was not a common practice at that time.

The 2-year return period design rainfall intensity is calculated per City standards as follows:

$$I_2 = \frac{21.8}{(T)^{0.78}} = \frac{21.8}{(10/60)^{0.78}} = 88.2 \text{ mm / hr}$$

The corresponding 2-Year pre-development flow to the Queen Street East combined sewer is calculated as follows:

$$Q_{\text{2-Year (Site)}} = \frac{(A \times R) \times I_2}{360} = \frac{(0.1820 \text{ ha} \times 0.50) \times 88.2 \text{ mm/hr}}{360} = 0.0223 \text{ m}^3/\text{s} = 22.3 \text{ L/s}$$

Therefore, the allowable drainage from the subject site to the existing municipal infrastructure shall be limited to **22.3 L/s**. Please refer to the detailed storm sewer design sheet which can be found in Appendix E.

5.4 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 recommended entry time of 10 minutes, and the following storm drainage runoff coefficients:

Storm Drainage Runoff Coefficients

Site Area	Coefficient
Bare Roof	0.90
Green Roof	0.50
Permeable Paver	0.50
Landscaped	0.25
Hard Surfaces	0.90

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

$$I_{100} = \frac{59.7}{(T)^{0.80}} = \frac{59.7}{(10/60)^{0.80}} = 250.3 \text{ mm/hr}$$

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

In order to ensure the water quantity design criteria in Section 4.2 is met, it is proposed that all drainage from the site (up to and including the 100-year storm event) be collected, controlled and discharged to the proposed combined service connection at the allowable release rate.

We note that due to grading constraints and building footprint, a small portion of the sidewalk on the private side that is not covered by roof will sheet drain uncontrolled to the City right-of-way on Queen St. East; the total uncontrolled area measures approximately 0.006 ha (56.8 m^2) with runoff coefficient of 0.90). This results in a 100-year flow rate of 3.6 L/s. We note that the tank will be designed to over-control the discharge from the site to ensure that the total is at or less than the allowable rate for the development. Thus, the total 100-year allowable release rate from the tank must be limited to **18.7 L/s** (i.e., 22.3 – 3.6 = 18.7 L/s). To attenuate flows to the target rate, on-site storage will be required. Typically, a combination of roof top, surface and/or underground storage is used to achieve the required volumes. In this case, there will be an underground storage tank that is proposed to contain flows generated from the subject site up to the 100-year level. Please refer to the detailed storm sewer design sheet which can be found in Appendix E.

5.4.1 Underground Storage Tank

As previously mentioned, the total allowable discharge from the stormwater tank shall be limited to **18.7 L/s**. The uncontrolled 100-year post-development release rate from the site to the combined sewer is calculated as follows:

$$Q_{100\text{-Unattenuated}} = \frac{(A \times R) \times I_{100}}{360} \times R = \frac{(0.1797 \text{ ha} \times 0.87) \times 250.3 \text{ mm / hr}}{360} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 108.3 \text{ L/s}$$

The uncontrolled flow of 108.3 L/s is significantly greater than the allowable release rate of 18.7 L/s. To attenuate flows from the site, an underground stormwater tank (2.85 m high above the orifice invert with a minimum footprint area of 27.8 m^2), complete with an orifice plate (upstream of the control manhole), is proposed to be located at the southeast corner of the property (within the P1 level of underground parking). Storm runoff generated on site will be collected and directed to this tank. It is important to stress that regular maintenance inspections of the tank and orifice should be conducted to ensure that there are no blockages or other conditions which would prevent the proper functioning of these design elements. The recommended minimum frequency of such inspections is annually.

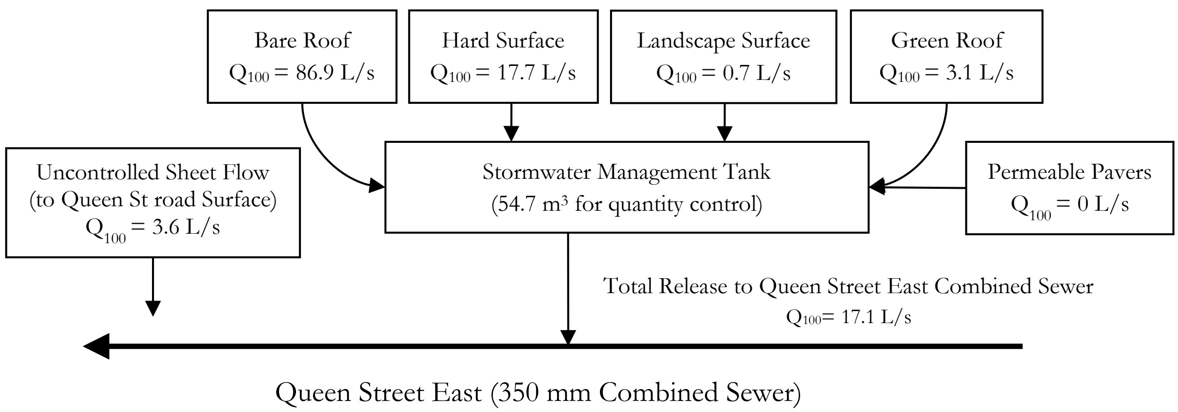
Utilizing a 75 mm diameter orifice plate and with the 100-Year storage depth in the system set at 1.97 m, the orifice discharge is calculated as follows:

$$Q_{\text{Orifice (SWM Tank)}} = (0.63) \times \frac{\pi \times (0.075)^2}{4} \times \sqrt{2 \times 9.81 \times (1.97\text{-}0.075/2)} \times \left(\frac{1000 \text{ L}}{\text{m}^3}\right) = 17.1 \text{ L/s}$$

Based on the storage provided, the total discharge from the site under 100-year conditions is summarized as follows:

Total Allowable Release Rate (see above):	22.3 L/s
Proposed Release Rate:	
Uncontrolled:	3.6 L/s
SWM Tank:	17.1 L/s
	20.7 L/s
Storage Required (quantity control only):	54.7 m^3
Storage Provided (to top of tank):	79.2 m^3

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



$$Q_{\text{SITE RELEASE}} \leq Q_{\text{ALLOWABLE DISCHARGE}}$$

$$20.7\ \text{L/s} \leq 22.3\ \text{L/s}$$

Based on the above, the total release rate from the subject site is less than the target post-development 2-year release rate and thus is deemed acceptable.

5.5 Water Quality Management

Pursuant to the City guidelines, stormwater quality controls are required to be implemented on-site to achieve a minimum of 80 % long-term total suspended solid (TSS) removal. For the purposes of determining the quality control achieved on the site, the following TSS removal rates will be applied for the various site areas:

TSS Removal Rates

Site Area	TSS Removal Rate
Bare Roof	80%
Green Roof	80%
Landscape	80%
Permeable	80%
Hard Surface	0%

Based on the above considerations, the following chart summarizes the subject site's inferred TSS removal rate:

Estimated TSS Removals on Subject Site

Site Area	Area (m^2)	% of Total	TSS Removal Rate	Overall
Bare Roof	1,389	74.9%	80%	59.9%
Green Roof	88	4.4%	80%	3.8%
Landscape	37	2.0%	80%	1.6%
Permeable Pavers	0	0.0%	80%	0%
Hard Surfaces	339	18.3%	0%	0%
Total	1,854	100 %		**65.4%**

In order to achieve the required TSS removal targets, the subject site requires supplemental treatment for storm runoff. It is therefore proposed to install a Contech Stormfilter unit (model SFPD0608 CIP), or approved equal, to treatment of runoff from the hard and landscaped surfaces collected by the area drains at-grade (i.e., upstream of the stormwater storage facility) and achieve the targeted 80% TSS removal. All roof areas are deemed clean and will drain directly to the stormwater tank. Please refer to Sizing Report for this unit which can be found in Appendix E.

5.6 Water Balance Management

In order to promote preservation of the site's natural hydrological water balance, the City's WWFMG recommends that a minimum volume of 5 mm over the total site area be retained.

The total water balance volume required to be retained for the subject site is calculated as follows:

- Volume Required = $A_{SITE} \times 5$ mm
- Volume Required = 1,854 $m^2 \times 5$ mm /(1000 mm/m) = **9.3 m^3**

Per Section 2.4 of the City's WWFMG, the acceptable methods for water balance reuse include:

- irrigation of landscaped and green roof areas (including evapo-transpiration),
- groundwater infiltration, or
- greywater systems (i.e., flushing toilets)

Since the underground parking level footprint spans the entire property, the opportunity to implement an infiltration system is not feasible, and therefore it is proposed that a sump be installed within the stormwater management tank in order to retain the required water balance volume and store it for active irrigation of landscaped and green roof areas on the property or greywater systems.

The following initial abstraction rates will be applied for the various site areas:

Initial Abstraction Rates

Site Area	Initial Abstraction
Bare Roof	1 mm
Green Roof	5 mm
Landscaped Areas	5 mm
Hard Surface	1 mm

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

Estimated Initial Abstraction Equivalent Depths

Site Area	Area (m^2)	% of Total	Initial Abstraction for Site Area	Total Initial Abstraction for Site Area (m^3)
Bare Roof	1,389	74.9%	1.0 mm	1.4
Green Roof	88	4.4%	5.0 mm	0.4
Landscape	37	2.0%	5.0 mm	0.2
Permeable Pavers	0	0.0%	5.0 mm	0
Hard Surfaces	339	18.3%	1.0 mm	0.3
Total	1,854	100 %		**2.4**

As such, it is recommended that a portion of the storm drainage collected in the stormwater storage tank be designated for on-site irrigation.

Water Balance Summary

Water Balance Uses	Volumes
Initial Abstraction	2.4 m^3
Water Re-Use (Active Irrigation)	6.9 m^3
Total	**9.3 m^3**

The volume available in the sump of the stormwater management tank for rainwater harvesting is calculated as follows:

$$V_{Provided} = A_{Tank} \times \text{Depth} \quad = 27.8\ m^2 \times 0.3\ m \quad = \mathbf{8.3\ m^3} > 6.9\ m^3$$

The majority of the storm drainage collected is from the roof (which is deemed clean) and is suitable for irrigation. The balance will be treated prior to entering the stormwater management tank.

Therefore, as shown above, the total volume of **9.3 m^3** retained on-site is equal to the required volume of **9.3 m^3**, and the available sump in the stormwater management tank is larger than the minimum required, thus satisfying the City's requirement for water balance.

5.7 Municipal Service Connection

The storm runoff from development will be captured and directed to a new 200 mm diameter storm service which will be connected to the existing 350 mm diameter combined sewer on Queen Street East. It is noted that the existing storm sewer is too shallow to feasibly service the proposed building, therefore, the storm service connection will be made to the combined sewer. Since the pre-development conditions also discharge stormwater to the 350 mm combined sewer, the total discharge to the combined sewer will be reduced because it will be released at a controlled rate, thus improving the overall conditions of the combined sewer system.

Based on information gathered from the City's plan and profile drawings, the invert of the existing sewer on Queen Street East is approximately 79.84 m±. The proposed storm service will be connected to the existing sewer at an invert of 80.25 m and be constructed at a 2.0% slope. The resultant invert of the storm service at the control manhole will be 80.32 m.

A 1.2m x 1.2m cast-in-place concrete storm control manhole shall be placed within the building footprint at the northeast corner of the site (within the P1 level), to allow for 24/7 inspection and monitoring thus satisfying the City's requirements. The service has adequate capacity to convey the post-development storm flow from the site and will operate at 35% of full flow capacity (22.5 L/s) under 100-year controlled flow conditions.

The location of the existing storm sewer infrastructure and proposed storm service connection is shown on the Site Servicing Schematic found in Appendix F.

5.8 Emergency Overflow

The stormwater management tank is proposed to be located at the southeast corner, along the frontage of the site in Parking Level 1 (P1). The proposed underground storage tank will be provided with an access frame and cover with an "open grate" which will act as an emergency overflow. If a storm greater than the 100-year return period is experienced, or the orifice becomes clogged, the stormwater tank is expected to surcharge via the open grate and spill onto the boulevard on the corner of Lee Avenue and Queen Street East (Elevation = 83.95 m).

We recommend backwater valves be installed between the stormwater management tank and the incoming connections to prevent water from entering the underground parking garage should a blockage occur within the individual leads or the stormwater management tank.

5.9 Erosion and Sediment Control

In accordance with the Erosion and Sediment Control Guidelines for Urban Construction, temporary erosion and sediment control measures are required for any development application. Due to the small size of the subject site, it is proposed that a sediment control fence be installed along the entire perimeter of the site per the City of Toronto standard drawing T-219.130-1.

Any existing/adjacent catchbasins shall be protected with a Terrafix 360R geotextile fabric (or approved equivalent). In addition, a mud mat shall be installed at the construction access to prevent any mud tracking onto the municipal roads.

6.0 CONCLUSIONS

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

It is anticipated that proposed domestic water and fire demands can be accommodated by the existing municipal water supply infrastructure on Queen Street East.

The receiving combined sewer network on Queen Street East can accommodate the proposed development without improvements.

The proposed internal storm sewer network, on-site underground storage tank, and the controlled discharge release rate to the receiving sewer satisfy the City's stormwater management, and combined sewer 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
A. MOKIN
100205528
2023 12 12
PROVINCE OF ONTARIO

Alessandro Stefenatti, B.Eng
Engineering Assistant

Angela Mokin, P.Eng.
Partner

Tel: +1.905.264.2420 x450
E-Mail: amokin@fabianpapa.com

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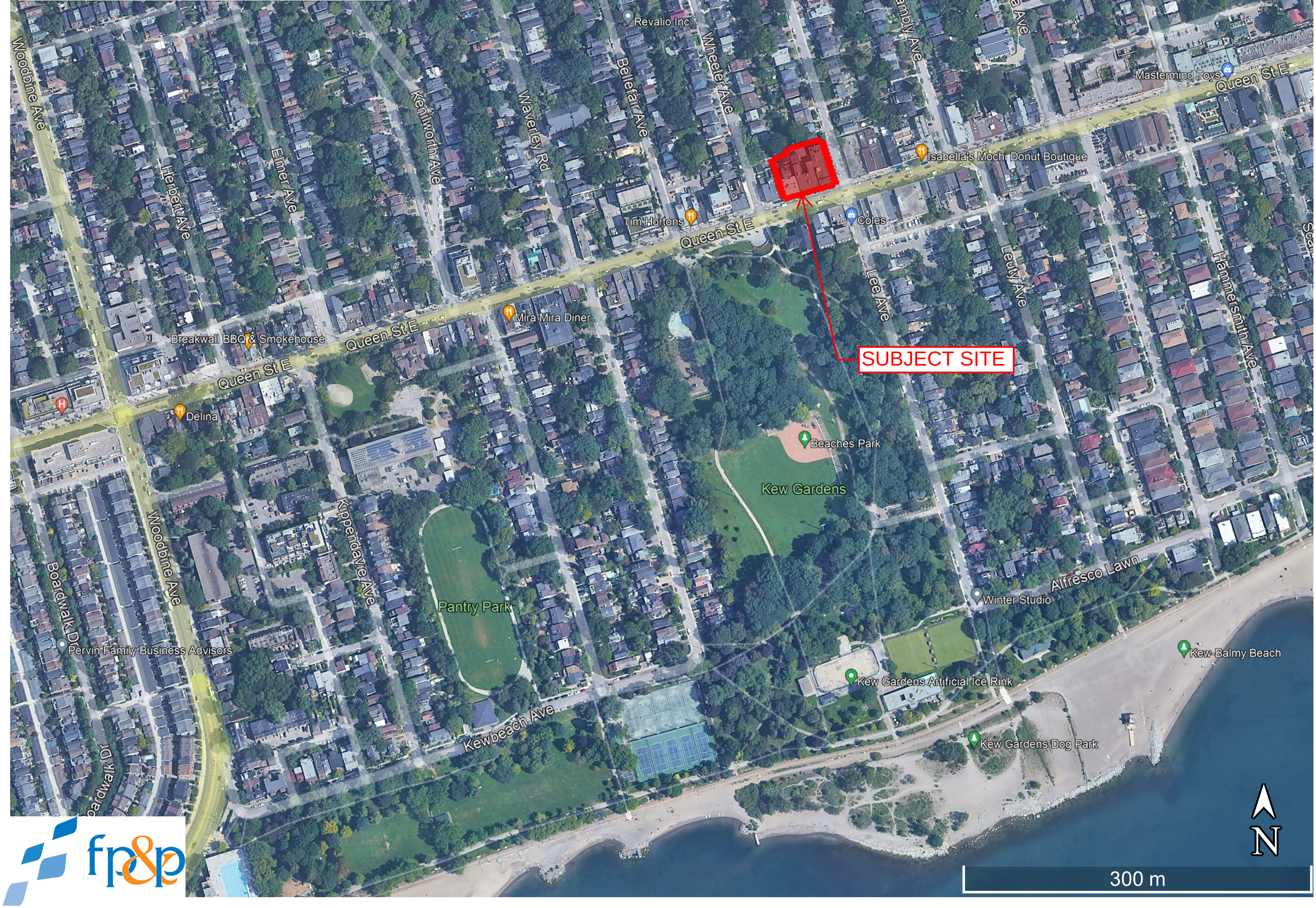
APPENDIX A

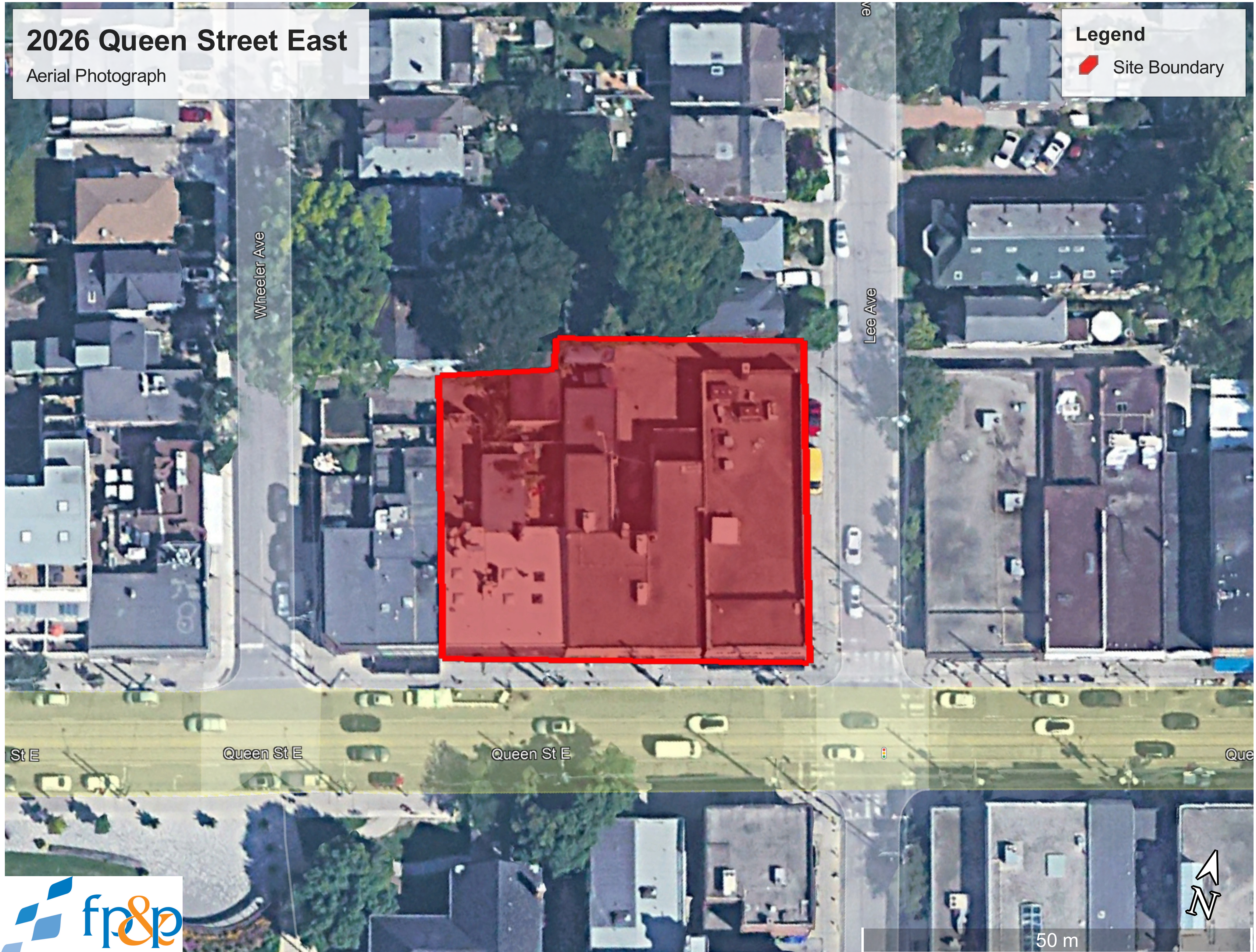
2026 Queen Street East

Key Plan



2026 Queen Street East

Aerial Photograph



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SITE STATISTICS

21018
2026-2042 QUEEN STREET EAST
Toronto, ON

7 December 2023

OFFICIAL PLAN	Residential	FSI:	RETAIL	0.61	AREA:	GROSS SITE	1854 sq.m.	19,956 sq.ft.
AVENUE WIDTH	20 m		RESIDENTIAL	2.47		LANE WIDENING	0 sq.m.	0 sq.ft.
CURRENT ZONING	By-Law 438-86		TOTAL	3.09		NET SITE	1854 sq.m.	19,956 sq.ft.
	MCR T2.0 C1.0 R2.0							

AREA CALCULATIONS

FLOOR	UNITS					GCA						GFA EXCLUSIONS*		GFA		NSA-RES	
						RETAIL		INDOOR AMENITY		RESIDENTIAL							
	STUDIO	1B/1B+	2B/2B+	3B/3B+	TOTAL	sq.m.	sq.ft.	sq.m.	sq.ft.	sq.m.	sq.ft.	sq.m.	sq.ft.	sq.m.	sq.ft.		
P1										1703.0	18330.9	1703.0	18330.9				
1					0	1140.0	12270.9			211.0	2271.2	73.0	785.8	1278.0	13756.3		
2		14	3		17			70.0	753.5	1267.0	13637.9	109.0	1173.3	1158.0	12464.6	1026.0	11043.8
3		19	2		21					1372.0	14768.1	28.0	301.4	1344.0	14466.7	1258.0	13541.0
4		6	1	6	13					1112.0	11969.5	28.0	301.4	1084.0	11668.1	1004.0	10807.0
5		1	8		9			51.0	549.0	619.0	6662.9	28.0	301.4	591.0	6361.5	453.0	4876.1
6										267.0	2874.0	18.0	193.8	249.0	2680.2	235.0	2529.5
7 (MPH)										154.0	1657.6	129.0	1388.5	25.0	269.1		0.0
SUB-TOTAL	0	40	14	6	60	1140.0	12271.3	121.0	1302.5	6705.0	72174.4	2116.0	22776.4	5728.0	61655.7	3976.0	42797.3
TOTAL RES + RETAIL										7845.0	84445.6			5728.0	61655.7		
TOTAL RES														4588.0	49384.8		
TOTAL RETAIL														1140.0	12270.9		
UNIT MIX	0%	67%	23%	10%				1									
BF UNITS REQ (15%)	0	6	2	1	9												
BF UNITS PROVIDED		6	2	1	9												

< Limited Indoor Amenity Exlcusion: subtract this number from total GFA if postiive

* Res. GCA totals include indoor amenity and parking

* GFA Exclusions: elevator and garbage shafts, exit stairs, loading area above-grade, bike parking (indoor), indoor amenity (limited to 2sm/unit), the MPH. and storage rooms, washrooms, electrical, utility, mechanical and ventilation rooms below grade.

AMENITY

INDOOR AMENITY	Required		Provided		
	sq.m.	sq.ft.	sq.m.	sq.ft.	
	120	1292	121.0	1,302	
OUTDOOR AMENITY					
	120	1292	136	1,464	
Total Amenity	240	2583	257	2766	4.28 m2/unit

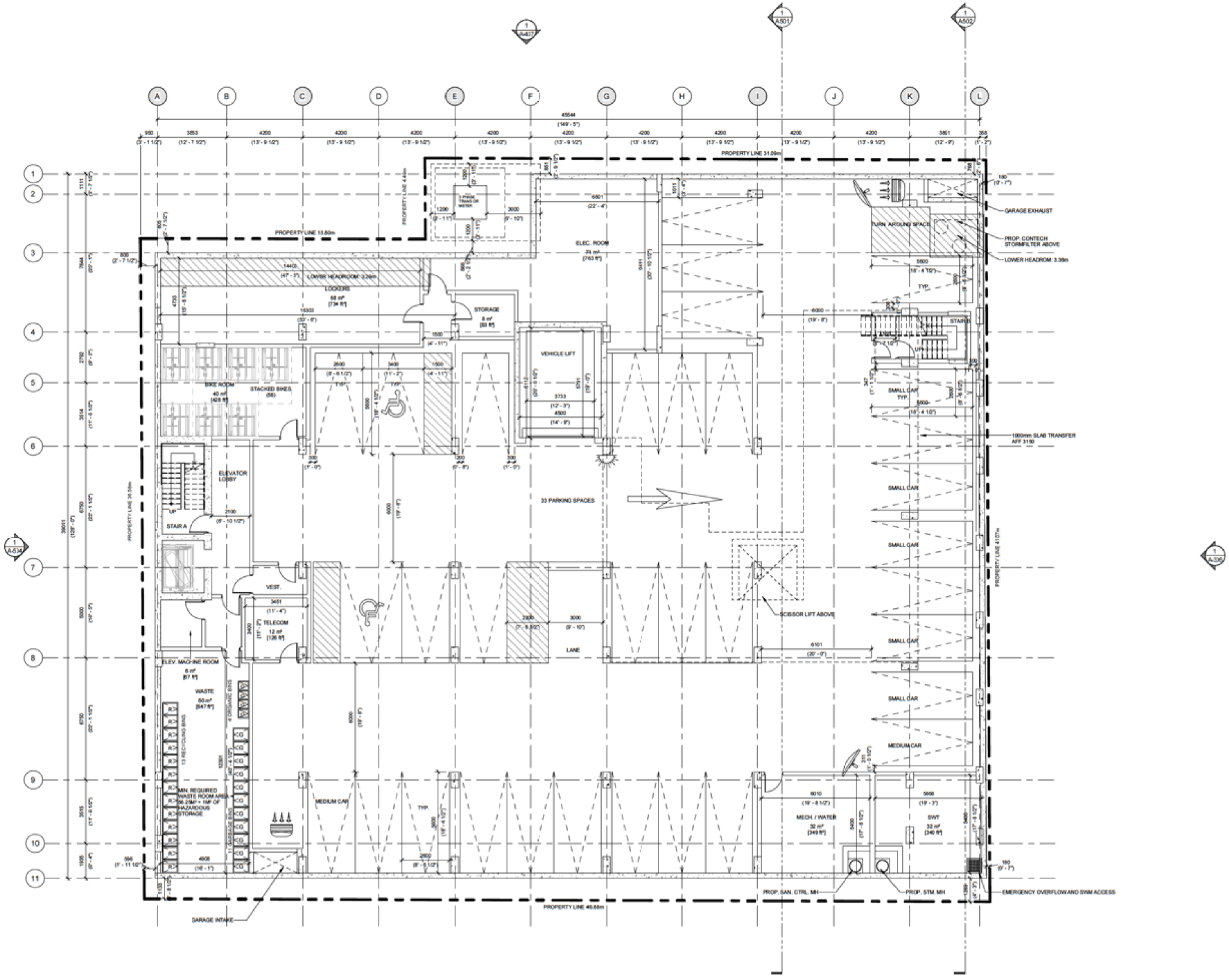
PARKING

VEHICLE	Provided
Residents	33
Visitor	
Commercial	
Total	33

NOTE:
All residential parking provided on p1 are provided with energized outlets capable of providing level 2 charging

BIKE	Required	Provided
RESIDENTIAL		
(Short-Term)		
Required Ratio (0.2)	12	12
(Long-Term)		
Required Ratio (0.9)	54	56
Total	66	68

NOTE:
All long-term bike parking provided on p1 - net area 2.35%
15% of bicycle parking spaces will contain an energized outlet (120V) adjacent to the parking space.



NOTES:

-ALL RESIDENTIAL PARKING PROVIDED ON P1 ARE PROVIDED WITH ENERGIZED OUTLETS CAPABLE OF PROVIDING LEVEL 2 CHARGING.

-15% OF BICYCLE PARKING SPACES WILL CONTAIN AN ENERGIZED OUTLET (120V) ADJACENT TO THE PARKING SPACE.

ISSUE RECORD

REVISION RECORD

405-317 ADELAIDE STREET WEST
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21018

2026 QUEEN ST E
TORONTO

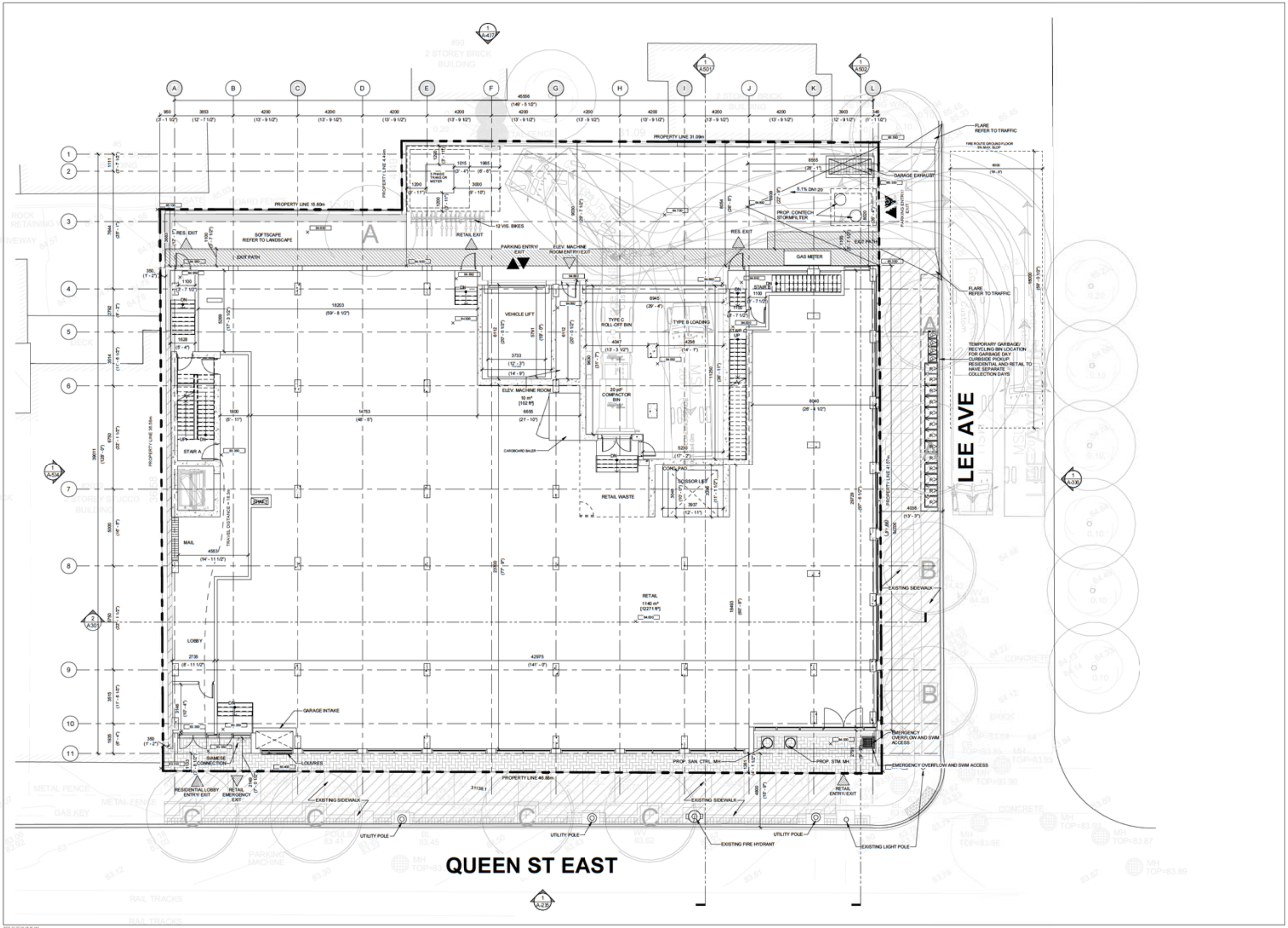
THE BEACHES

CROMBIE REIT

P1 PARKING PLAN

SCALE: 1 : 100

A101



THIS DRAWING IS THE PROPERTY OF THE ARCHITECT AND MAY NOT BE REPRODUCED OR USED WITHOUT THE EXPRESS CONSENT OF THE ARCHITECT. THE CONTRACTOR IS RESPONSIBLE FOR CHECKING AND VERIFYING ALL LEVELS AND DIMENSIONS AND SHALL REPORT ALL DISCREPANCIES TO THE ARCHITECT AND OBTAIN CLARIFICATION PRIOR TO COMMENCING WORK.

ISSUE RECORD

—

REVISION RECORD

—

RAW

—

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—

2026 QUEEN ST E
TORONTO

—

THE BEACHES

—

CROMBIE REIT

—

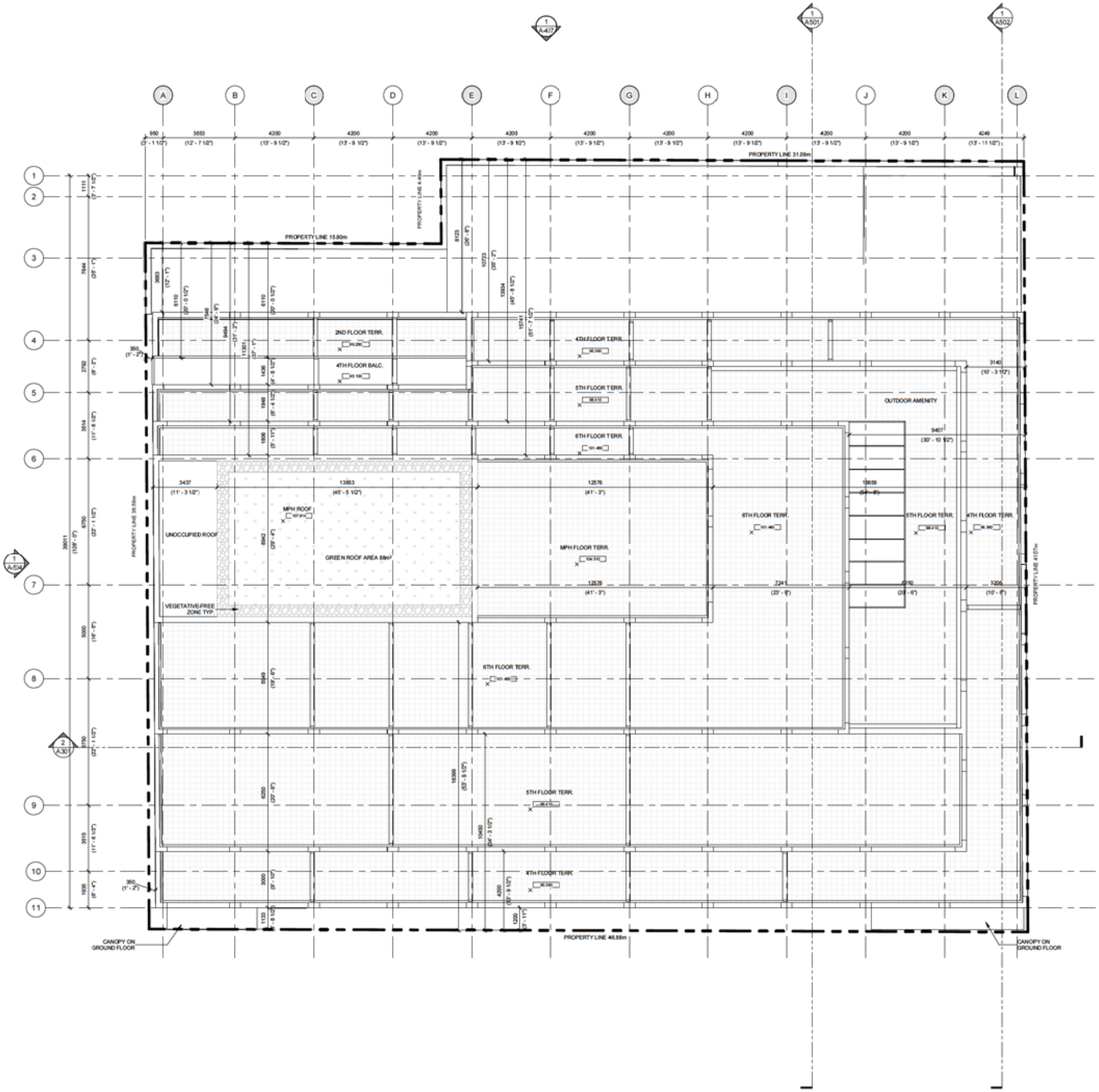
GROUND FLOOR PLAN

—

SCALE: 1 : 100

—

A201



TORONTO City Planning Division

Green Roof Statistics

The Green Roof Statistics Template is required to be submitted for Site Plan Control Applications where a green roof is required under the Toronto Municipal Code Chapter 492, Green Roofs. Complete the table below and **copy it directly onto the Roof Plan** submitted as part of any Site Plan Control Application requiring a green roof in accordance with the Bylaw. Refer to Section § 492-1 of the Municipal Code for a complete list of defined terms, and greater clarity and certainty regarding the intent and application of the terms included in the template. The Toronto Municipal Code Chapter 492, Green Roofs can be found online at: http://www.toronto.ca/legdocs/municode/1184_492.pdf

Green Roof Statistics

		Proposed
Gross Floor Area, as defined in Green Roof Bylaw (m^2)		6,627
Total Roof Area (m^2)		1,477
Area of Residential Private Terraces (m^2)		1,065
Rooftop Outdoor Amenity Space, *if in a Residential Building* (m^2)		120
Area of Renewable Energy Devices (m^2)		N/A
Tower (s)Roof Area with floor plate less than 750 m^2		N/A
Total Available Roof Space (m^2)		292
Green Roof Coverage	Required	Proposed
Coverage of Available Roof Space (m^2)	88	88
Coverage of Available Roof Space (%)	30%	30%

ISSUE RECORD

REVISION RECORD

NORTH

RAW

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2026 QUEEN ST E
TORONTO

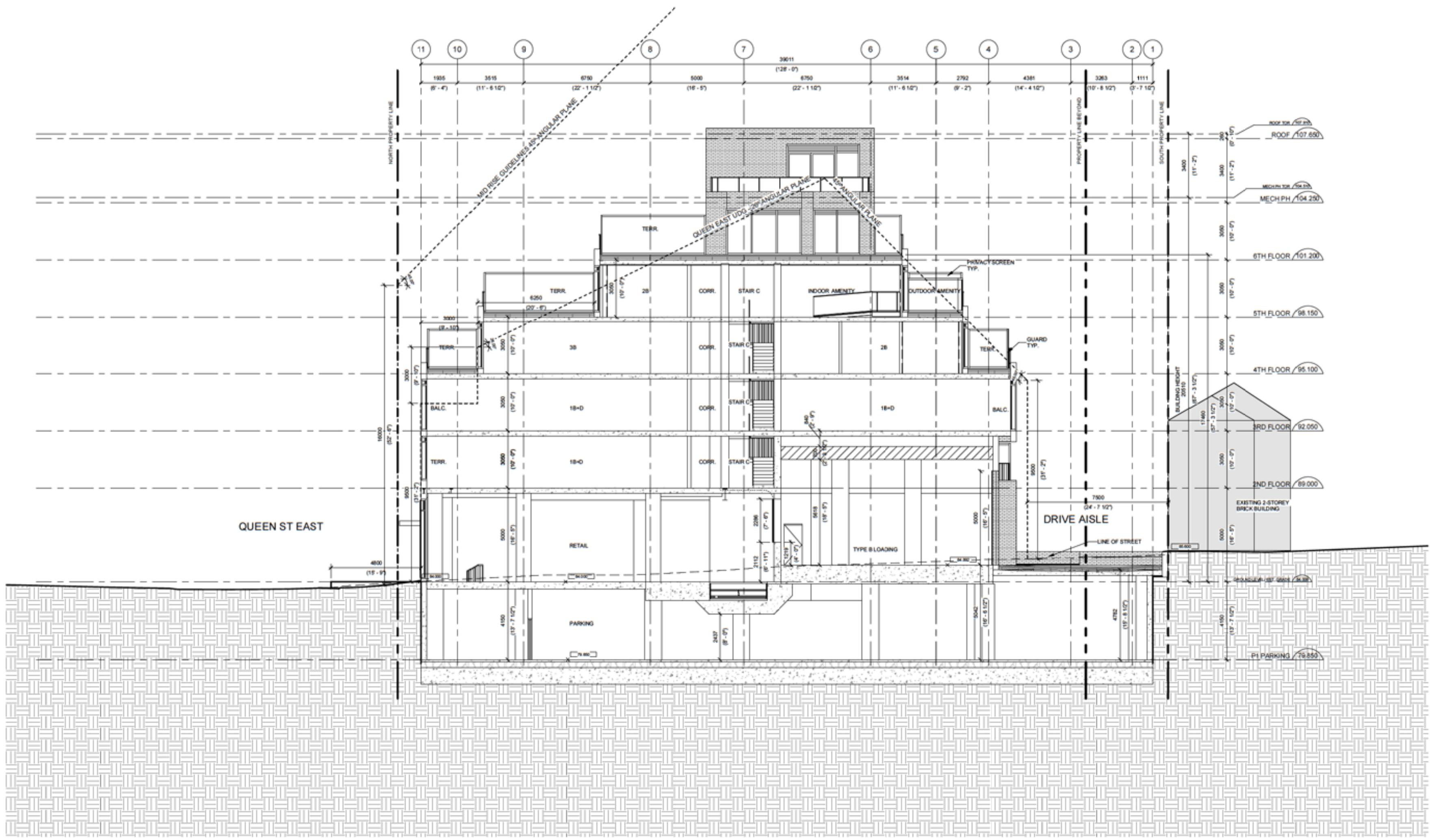
THE BEACHES

CROMBIE REIT

ROOF PLAN

SCALE: 1 : 100

A208



ISSUE RECORD

REVISION RECORD

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2026 QUEEN ST E
TORONTO

THE BEACHES

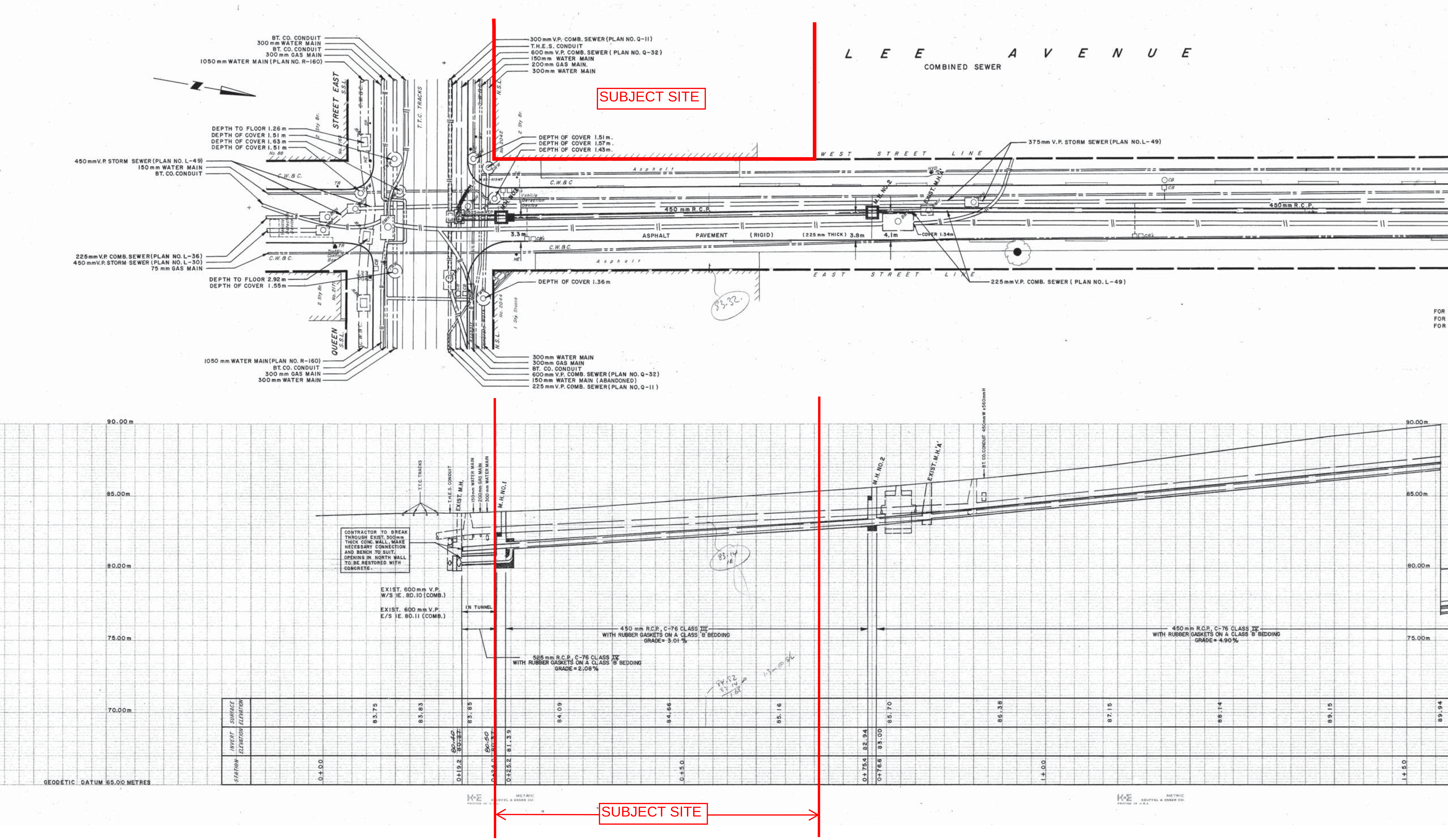
CROMBIE REIT

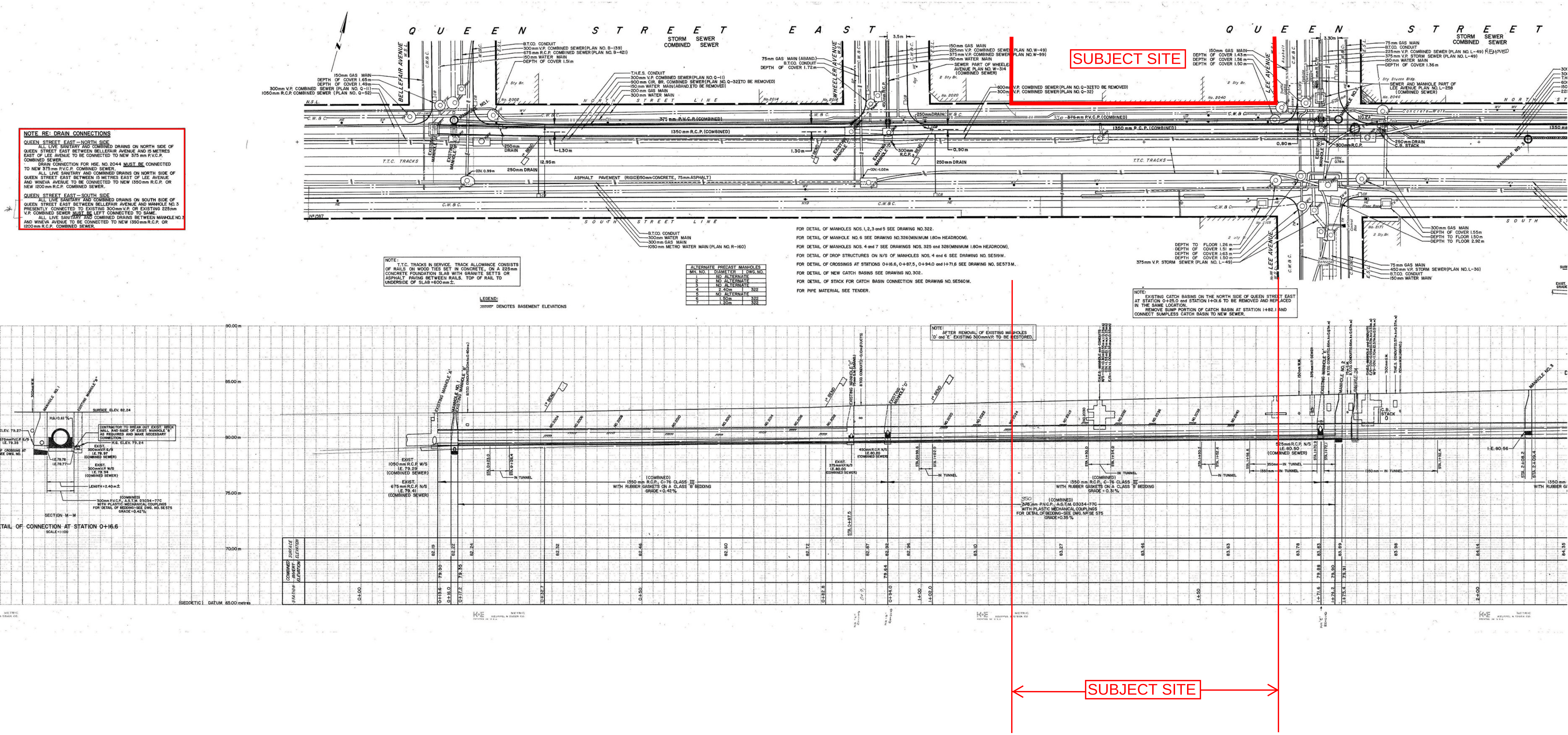
BUILDING SECTION

SCALE: 1 : 100

A501

APPENDIX B


L E E A V E N U E
COMBINED SEWER
SUBJECT SITE
BT. CO. CONDUIT
300 mm WATER MAIN
BT. CO. CONDUIT
300 mm GAS MAIN
1050 mm WATER MAIN (PLAN NO. R-160)
300 mm V.P. COMB. SEWER (PLAN NO. Q-11)
T.H.E.S. CONDUIT
600 mm V.P. COMB. SEWER (PLAN NO. Q-32)
150 mm WATER MAIN
200 mm GAS MAIN.
300 mm WATER MAIN
STREET EAST
QUEEN
T.T.C. TRACKS
DEPTH TO FLOOR 1.26 m
DEPTH OF COVER 1.51 m
DEPTH OF COVER 1.63 m
DEPTH OF COVER 1.51 m
DEPTH OF COVER 1.51 m.
DEPTH OF COVER 1.57 m.
DEPTH OF COVER 1.43 m.
450 mm V.P. STORM SEWER (PLAN NO. L-49)
150 mm WATER MAIN
BT. CO. CONDUIT
375 mm V.P. STORM SEWER (PLAN NO. L-49)
WEST STREET LINE
EAST STREET LINE
450 mm R.C.P.
ASPHALT PAVEMENT (RIGID) (225 mm THICK)
3.3 m
3.8 m
4.1 m
COVER 1.34 m
M.H. NO. 2
EXIST. M.H. 'A'
225 mm V.P. COMB. SEWER (PLAN NO. L-36)
450 mm V.P. STORM SEWER (PLAN NO. L-30)
75 mm GAS MAIN
DEPTH TO FLOOR 2.92 m
DEPTH OF COVER 1.55 m
DEPTH OF COVER 1.36 m
225 mm V.P. COMB. SEWER (PLAN NO. L-49)
1050 mm WATER MAIN (PLAN NO. R-160)
BT. CO. CONDUIT
300 mm GAS MAIN
300 mm WATER MAIN
300 mm WATER MAIN
300 mm GAS MAIN
BT. CO. CONDUIT
600 mm V.P. COMB. SEWER (PLAN NO. Q-32)
150 mm WATER MAIN (ABANDONED)
225 mm V.P. COMB. SEWER (PLAN NO. Q-11)
90.00 m
85.00 m
80.00 m
75.00 m
70.00 m
CONTRACTOR TO BREAK THROUGH EXIST. 300 mm THICK CONC. WALL, MAKE NECESSARY CONNECTION AND BENCH TO SUIT. OPENING IN NORTH WALL TO BE RESTORED WITH CONCRETE.
EXIST. 600 mm V.P. W/S IE. 80.10 (COMB.)
EXIST. 600 mm V.P. E/S IE. 80.11 (COMB.)
IN TUNNEL
450 mm R.C.P., C-76 CLASS III WITH RUBBER GASKETS ON A CLASS 'B' BEDDING GRADE = 3.01 %
525 mm R.C.P., C-76 CLASS IV WITH RUBBER GASKETS ON A CLASS 'B' BEDDING GRADE = 2.08 %
450 mm R.C.P., C-76 CLASS IV WITH RUBBER GASKETS ON A CLASS 'B' BEDDING GRADE = 4.90 %
M.H. NO. 1
EXIST. M.H.
SURFACE ELEVATION
INVERT ELEVATION
STATION
83.75
83.83
83.85
84.09
84.66
85.16
85.70
86.38
87.15
88.14
89.15
89.94
81.39
82.94
83.00
0+00
0+19.2
0+25.2
0+50
0+75.4
0+76.6
1+00
1+50
GEODETIC DATUM 65.00 METRES
SUBJECT SITE



QUEEN STREET EAST
SUBJECT SITE
BELLEFAIR AVENUE
WHEELER AVENUE
LEE AVENUE
NORTH STREET LINE
SOUTH STREET LINE
T.T.C. TRACKS
375 mm P.V.C.P. (COMBINED)
1350 mm R.C.P. (COMBINED)
NOTE RE: DRAIN CONNECTIONS
QUEEN STREET EAST – NORTH SIDE
ALL LIVE SANITARY AND COMBINED DRAINS ON NORTH SIDE OF QUEEN STREET EAST BETWEEN BELLEFAIR AVENUE AND 15 METRES EAST OF LEE AVENUE TO BE CONNECTED TO NEW 375 mm P.V.C.P. COMBINED SEWER.
DRAIN CONNECTION FOR HSE. NO. 2044 MUST BE CONNECTED TO NEW 375 mm P.V.C.P. COMBINED SEWER.
ALL LIVE SANITARY AND COMBINED DRAINS ON NORTH SIDE OF QUEEN STREET EAST BETWEEN 15 METRES EAST OF LEE AVENUE AND WINEVA AVENUE TO BE CONNECTED TO NEW 1350 mm R.C.P. OR NEW 1200 mm R.C.P. COMBINED SEWER.
QUEEN STREET EAST – SOUTH SIDE
ALL LIVE SANITARY AND COMBINED DRAINS ON SOUTH SIDE OF QUEEN STREET EAST BETWEEN BELLEFAIR AVENUE AND MANHOLE NO. 3 PRESENTLY CONNECTED TO EXISTING 300 mm V.P. OR EXISTING 225 mm V.P. COMBINED SEWER MUST BE LEFT CONNECTED TO SAME.
ALL LIVE SANITARY AND COMBINED DRAINS BETWEEN MANHOLE NO. 3 AND WINEVA AVENUE TO BE CONNECTED TO NEW 1350 mm R.C.P. OR 1200 mm R.C.P. COMBINED SEWER.
DETAIL OF CONNECTION AT STATION 0+16.6
SUBJECT SITE

APPENDIX C

HYDRANT FLOW TEST REPORT

HydraTek & Associates

PROJECT INFORMATION

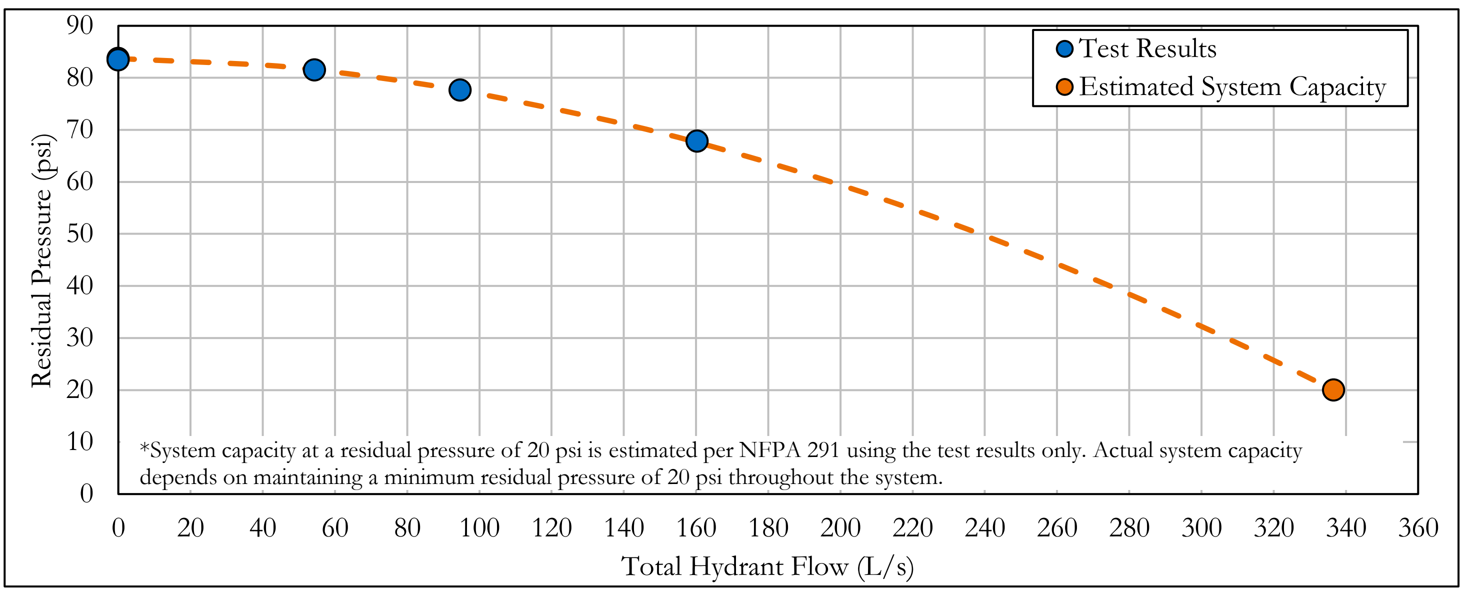
HydraTek Project No.:	22117
Project Name:	2026 Queen Street East Development, City of Toronto
Client/Owner Name:	Crombie REIT
Date and Time of Test:	24 October 2022 at 12:05 PM
Fire Hydrant Use Permit:	11703931-2689
Test Conducted By:	N. Tomic and S. Genser
Design Flow:	tbd
Municipality:	City of Toronto
Pressure Zone/District:	Pressure District 1 (PD1)

TEST INFORMATION

Residual Hydrant:	2000 Queen Street East, 1st W of Bellefair Avenue, N side, ID HY21479
Flow Hydrant No. 1:	2036 Queen Street East, 1st W of Lee Avenue, N side, ID HY1353802
Flow Hydrant No. 2:	n/a
Watermain Size / Material:	300 mm (12-in) Cast Iron (1907)

TEST RESULTS

Test No.	No. Ports and Size	Pitot Pressure (psi) Flow Hydrant			Total Flow (L/s)	Residual Pressure (psi)
		Port 1	Port 2	Port 3		
1	Initial Static	-	-	-	0	83.7
2	1 x 2.0-in	30.5	-	-	54	81.5
3	2 x 2.0-in	24.0	22.3	-	95	77.6
4	2 x 2.0-in + 1 x 4.0-in	11.9	11.0	15.6	160	67.8
5	Final Static	-	-	-	0	83.4
				Estimated System Capacity (L/s):	336	20



TEST COMMENTS

-With a single flow hydrant the test was <u>not</u> able to achieve a minimum 25% drop in residual pressure per NFPA 291 test requirements.

-Estimated system capacity is based on NFPA 291 using the test results only. Actual system capacity depends on the maximum flow that can be withdrawn subject to maintaining a minimum residual pressure of 20 psi (14.3 m; 140 kPa) at the test location and throughout the rest of the water system.

PREPARED BY:

Name: Nikola Tomic Signature:

2026 Queen Street East - Mixed Use Development
Water Demand Calculations

Designed By: **Alessandro Stefenatti**, B.Eng
Checked By: **Angela Mokin**, P.Eng.
File No.: 22117
Date: 11 December 2023

Domestic Water Supply Demands:
Per City of Toronto Design Criteria for Water Distribution Systems
- assume Average Day demand is 190 L/capita/day for multi-unit residential uses
- assume Population Density (see chart)

Unit Type	Population Density
Office	3.3 Pers / 100 m^2
1-Bed	1.4 Pers / Unit
2-Bed	2.1 Pers / Unit
3-Bed	3.1 Pers / Unit

Building	Building Data		Population	Ave. Day Flow	Peak Hour, ADxPH [1]	Max. Day, ADxMD [2]
	Units	(sq.m)	pers	(L/s)	(L/s)	(L/s)
Office	n/a	0	0	0.00	0.00	0.00
Retail	n/a	1,140	13	0.03	0.03	0.03
1-Bed	40	n/a	56	0.12	0.31	0.16
2-Bed	14	n/a	30	0.07	0.16	0.09
3-Bed	6	n/a	19	0.04	0.10	0.05
Total	60		118	0.26	0.61	0.33

[1] Peak Hour Demand, PHD, is 2.5 for residential and 1.20 for commercial
[2] Max Day Demand, MDD, is 1.3 for residential and 1.10 for commercial

Fire Protection Supply Demands:
Per Water Supply for Public Fire Protection Manual, 2020, by the Fire Underwriters Survey

STEP 1: Calculate Fire Flow

C = Coefficient related to type of construction:
= 1.5 for Type V (Wood Frame Construction)
= 0.8 for Type IV-A (Encapsulated Mass Timber Construction and structural elements with min. 2 hr rating and roof has min. 1 hr rating)
= 0.9 for Type IV-B (Rated Mass Timber Construction and all elements have a min. 1 hr rating)
= 1.0 for Type IV-C (Ordinary Mass Timber Construction and structural elements with min. 1 hr rating and roof has no rating)
= 1.5 for Type IV-D (Un-Rate Mass Timber Construction and exterior walls have <1 hr rating regardless of other element ratings)
= 1.0 for Type III (Ordinary Construction - exterior masonry walls with min. 1 hr rating, combustible floor and interior)
= 0.8 for Type-II (Non Combustible Construction - structural elements, walls, floors, roofs with min. 1 hr rating)
= 0.6 for Type I (Fire Resistive Construction - structural elements, walls, floors, roofs with min. 2 hr rating)

C =	0.6		
Largest Floor Area =	1372	m^2	
Floor Area Above =	1112	m^2	
Floor Area Below =	1337	m^2	
A =	1,984	m^2	Largest Floor + 25% x (Floor Above + Floor Below)
F =	5,880	L/min	
F =	6,000	L/min	Round to the nearest 1000

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 = 5,100 L/min

2026 Queen Street East - Mixed Use Development
Water Demand Calculations

STEP 3: Decrease F1 if building contains fire suppression 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 =	-40%	
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 =	N 7.5m / E 15m / S 18m / W 0.31m	
Factors =	20% + 15% + 15% + 25%	
Factor =	75%	(max 75%)
F3 = F1 x Factor =	3,825	L/min

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

Fire Flow =	F1 - F2 + F3		
F1 =	5,100	L/min	
- F2 =	2,040	L/min	
+ F3 =	3,825	L/min	
Fire Flow =	6,885	L/min	
Fire Flow =	7,000	L/min	Round to the nearest 1000
Fire Flow =	116.7	L/s	

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

Recall Max Day Demand (from chart above) =	0.33	L/s
TOTAL Fire Demand =	117.0	L/s

2026 Queen Street East - Mixed Use Development

Supply Line Head Loss Calculations

Designed By: Alessandro Stefenatti, B.Eng
Checked By: Angela Mokin, P.Eng.
File No.: 22117
Date: 11 December 2023

Total Fire Demand = 117.0 L/s (Taken From Fire Flow Spreadsheet)

Hydrant Flow Test Results

Flow (gpm)	Flow (L/s)	Pressure (psi)	Pressure (kPa)
0	0	83.7	577
856	54	81.5	562
1506	95	77.6	535
2536	160	67.8	467
1,854	117	75	516
5,349	338	20	140

2022 10 24 Queen Street Hydrant Flow Test Results

Estimated available Fire Flow calculated 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= 1.9 m
D= 100 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)
0.6	0.00	0.0	0.0	0.00	0.00	83.7	577	**83.7**	**577**

[1] Residual pressure taken from above

New 200 mm Fire Line

L= 1.9 m
D= 150 mm
C= 110

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)
117.0	0.12	0.65	25.7	0.93	6.41	74.3	512	**73.4**	**506**

[1] Residual pressure taken from above

APPENDIX D

2026 Queen Street East - Mixed Use Development

City of Toronto - Engineering & Construction Services

SANITARY SEWER DESIGN SHEET

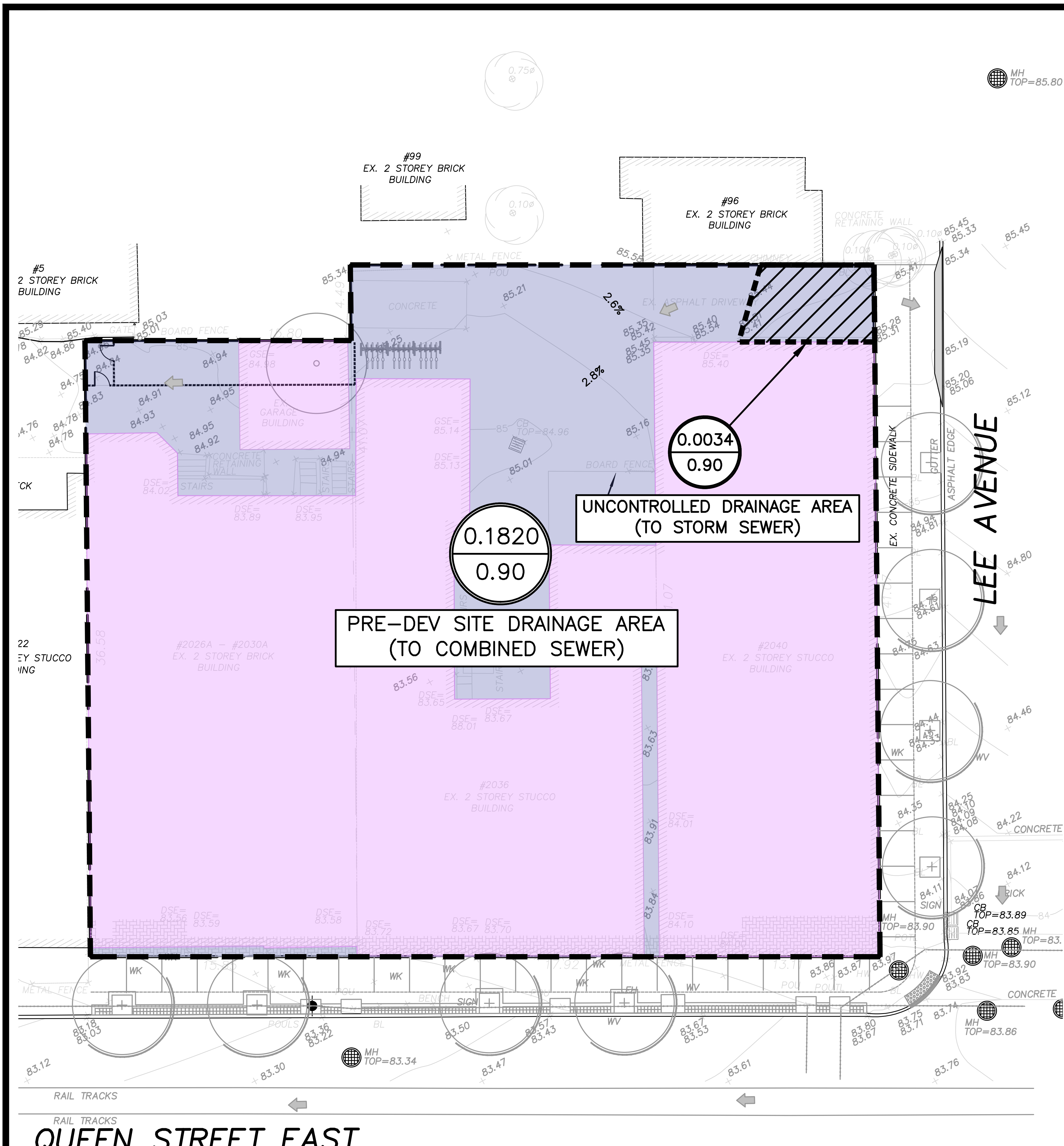
NOTES

Pre-development residential sewage flow based upon 240 Lpcd (residential) and 250 Lpcd (industrial/commercial).
Pre-development office population density of 3.3 people/100 sqm.
Post-development domestic sewage flow based upon a unit flow of 450.0 Lpcd.
Infiltration flow based upon a unit flow of 0.26 L/s/ha.
Maximum flow velocity for pipe flowing full = 3.0 m/s.
Minimum flow velocity for pipe flowing partially full (actual flow) = 0.6 m/s.

Designed By: **Angela Mokin**, P.Eng.
Checked By: **Angela Mokin**, P.Eng.
File No.: 22117
Date: 20 November 2023

			DESIGN FLOW CALCULATIONS										SEWER DESIGN & ANALYSIS							
	from M.H.	to M.H.	Area (ha) or No. Units	Density (p/unit)	Population	Cumulative Area (ha)	Cumulative Population	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)	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
DESIGN FLOWS																				
PRE-DEVELOPMENT																				
Subject Site	Retail		2750	0.011	31	0.185	31		0.09	0.05		**0.1**								
POST-DEVELOPMENT										**Based on 250 Lpcd**										
Subject Site	Retail		1140	0.011	13		13		0.0	0.19		0.2								
	1-Bed		40	1.4	56															
	2-Bed		14	2.1	30					**Based on 240 Lpcd**										
	3-Bed		6	3.1	19															
	Total Units		60				105	4.24	1.2			1.2								
	Total Site		0.185			0.185	118					**1.5**	Post-Development flow for sewer capacity analysis							
										Total Increase in Flow		**1.3**								
										Based on 450 Lpcd (Used for Sewer Design)										
	Total Site		0.185			0.185	118	4.22	2.6	0.05	0.0	**2.6**	150	2.0%	3.5	22.5	1.23	11.8%	0.8	Self Cleansing Ok

APPENDIX E



LEGEND

SITE DRAINAGE AREA

UNCONTROLLED DRAINAGE AREA

ROOF DRAINAGE AREA

HARD SURFACE DRAINAGE AREA

2.518 — SITE AREA (Ha)

0.82 — SITE RUNOFF COEFFICIENT

DWN. BY:	A.M.
DESIGNED BY:	A.M.
CHECKED BY:	R.F.
SCALE:	1:300
DATE:	NOV 2023
SHEET NO:	1 OF 2

fabian papa & partners
A Division of FP&P HydraTek Inc.
3901 Highway 7, Suite 500
Vaughan, Ontario, L4L 8L5
t: 905-264-2420
www.fabianpapa.com

PROJECT NAME:
2026 QUEEN STREET EAST
MIXED USE DEVELOPMENT

DRAWING TITLE:
PRE–DEV STORMWATER MANAGEMENT DRAINAGE PLAN

PROJECT NO:	DWG NO:	REV NO:
22117	SWM–1	0

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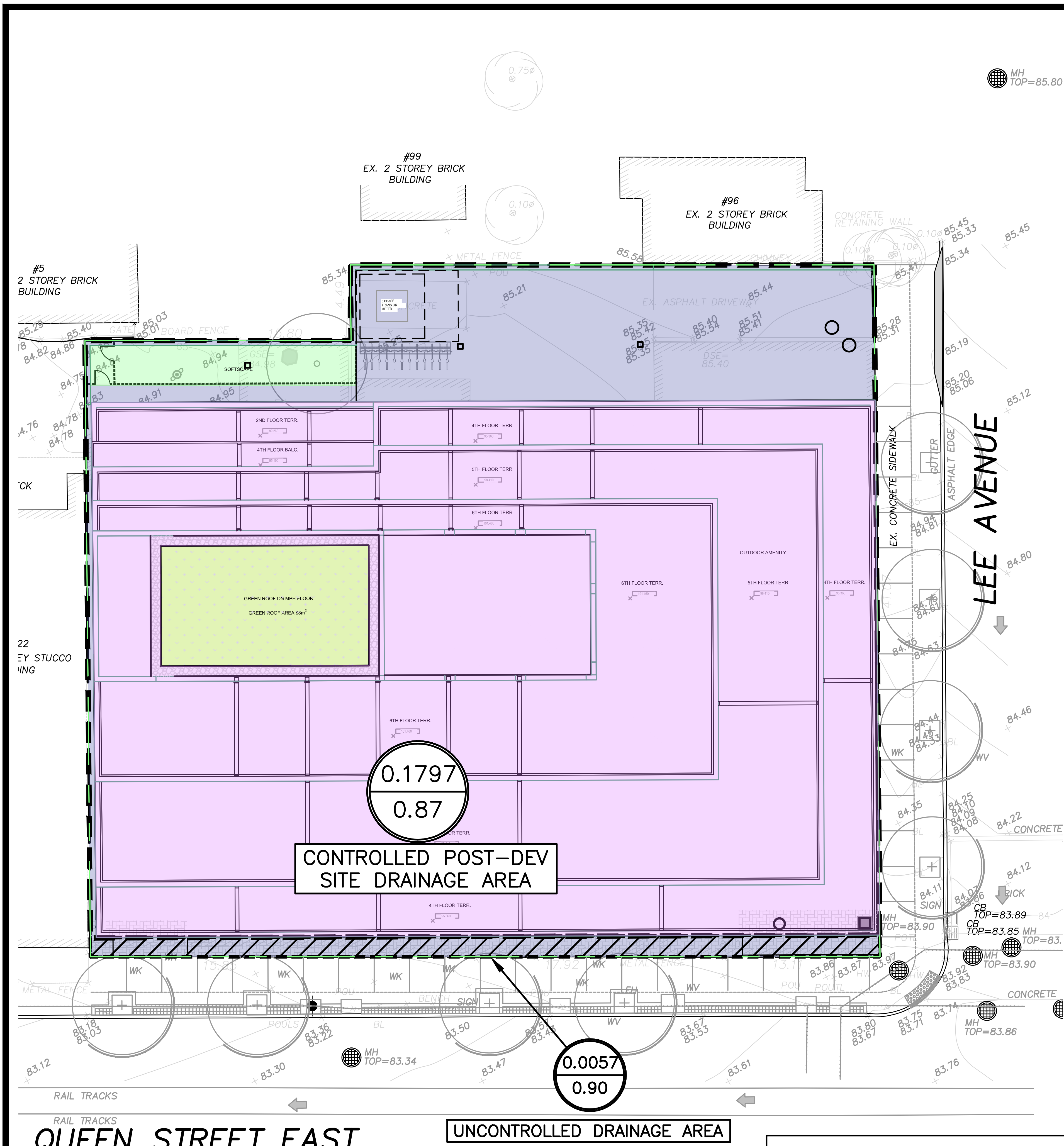


LEGEND

- SWM TANK DRAINAGE AREA
- UNCONTROLLED SHEET DRAINAGE AREA
- GREENROOF AREA
- ROOF DRAINAGE AREA
- HARD SURFACE AREA
- LANDSCAPED AREA

2.518 — DRAINAGE AREA (Ha)
0.82 — SITE RUNOFF COEFFICIENT

DWN. BY:	A.M.
DESIGNED BY:	A.M.
CHECKED BY:	R.F.
SCALE:	1:300
DATE:	DEC 2023
SHEET NO:	2 OF 2

fabian papa & partners
A Division of FP&P HydraTek Inc.
3901 Highway 7, Suite 500
Vaughan, Ontario, L4L 8L5
t: 905-264-2420
www.fabianpapa.com

PROJECT NO:	DWG NO:	REV NO:
22117	SWM–2	0

PROJECT NAME:
2026 QUEEN STREET EAST
MIXED USE DEVELOPMENT

DRAWING TITLE:
POST–DEV STORMWATER MANAGEMENT DRAINAGE PLAN

2026 Queen Street East - Mixed Use Development

Weighted Run-Off Coefficient Calculations

Based on WWFMG - City of Toronto

Designed By: **Angela Mokin**, P.Eng.
Checked By: **Angela Mokin**, P.Eng.
File No. 22117
Date: 12 December 2023

Pre-Dev. To Combined Sewer				
Roof Bare	1398.1	76.8%	0.90	0.69
Green Roof	0.0	0.0%	0.50	0.00
Landscape	0.0	0.0%	0.25	0.00
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	422.1	23.2%	0.90	0.21
	1820.2	100%		0.90

Pre-Dev. Uncontrolled Sheet Drainage to Lee Avenue				
Roof Bare	0.0	0.0%	0.90	0.00
Green Roof	0.0	0.0%	0.50	0.00
Landscape	0.0	0.0%	0.25	0.00
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	33.8	100.0%	0.90	0.90
	33.8	100%		0.90

Total Pre-Dev. Site Drainage				
Roof Bare	1398.1	75.4%	0.90	0.68
Green Roof	0.0	0.0%	0.50	0.00
Landscape	0.0	0.0%	0.25	0.00
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	455.9	24.6%	0.90	0.22
	1854.0	100%		0.90

Drainage Direct to the SWM Tank				
Roof Bare	1389.2	77.3%	0.90	0.70
Green Roof	88.0	4.9%	0.50	0.02
Landscape	37.5	2.1%	0.25	0.01
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	282.5	15.7%	0.90	0.14
	1797.2	100%		0.87

Uncontrolled Sheet Drainage to Queen St E				
Roof Bare	0.0	0.0%	0.90	0.00
Green Roof	0.0	0.0%	0.50	0.00
Landscape	0.0	0.0%	0.25	0.00
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	56.8	100.0%	0.90	0.90
	56.8	100%		0.90

Total Proposed Development				
Roof Bare	1389.2	74.9%	0.90	0.67
Green Roof	88.0	4.7%	0.50	0.02
Landscape	37.5	2.0%	0.25	0.01
Permeable	0.0	0.0%	0.50	0.00
Hard Surface	339.3	18.3%	0.90	0.16
	1854.0	100%		0.87

2026 Queen Street East - Mixed Use Development

City of Toronto - Engineering & Construction Services

2 / 100 YEAR STORM SEWER DESIGN SHEET

fp&p

2-Year IDF Curve $I_{2-Yr} = \frac{21.8}{T^{0.78}}$

100-Year IDF Curve $I_{100-Yr} = \frac{59.7}{T^{0.80}}$

Designed By: **Angela Mokin**, P.Eng.
Checked By: **Angela Mokin**, P.Eng.
File No.: 22117
Date: 12 December 2023

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
2-YEAR PRE-DEVELOPMENT ACTUAL FLOW (TO COMBINED SEWERS)																			
2026 Queen Street East	Roof Bare	Sewer	0.1398	0.90	0.126	0.126	10.0	88.2	30.8										
	Hard Surface	Sewer	0.0422	0.90	0.038	0.038	10.0	88.2	9.3										
		Total	0.1820	0.90					40.1	Existing Drainage Tributary to Combined Sewer on Queen Avenue									
2-YEAR PRE-DEVELOPMENT ACTUAL FLOW (TO STORM SEWER - Lee Avenue)																			
	Un-Controlled	Lee Ave Stm	0.0034	0.90	0.003	0.003	10.0	88.2	0.7	Existing Drainage Tributary to Storm Sewer on Lee Avenue									
2026 Queen Street East	Site	Total	0.1854	0.90															
2-YEAR PRE-DEVELOPMENT ALLOWABLE FLOW - Calculated Drainage																			
2026 Queen Street East		Total	0.1820	0.50	0.091	0.091	10.0	88.2	22.3	Maximum per WWFMG, & Max Discharge to Combined Sewer System									
100-YEAR PRE-DEVELOPMENT ACTUAL FLOW - Calculated Drainage																			
2026 Queen Street East		Total	0.1820	0.90	0.164	0.164	10.0	250.3	113.9										
100-YEAR POST-DEVELOPMENT CONDITIONS																			
2026 Queen Street East	Roof Bare	STM Tank	0.1389	0.90	0.125	0.125	10.0	250.3	86.9										
	Green Roof	STM Tank	0.0088	0.50	0.004	0.004	10.0	250.3	3.1										
	Landscape	STM Tank	0.0038	0.25	0.001	0.001	10.0	250.3	0.7										
	Permeable	STM Tank	0.0000	0.50	0.000	0.000	10.0	250.3	0.0										
	Hard Surface	STM Tank	0.0283	0.90	0.025	0.025	10.0	250.3	17.7										
	Total	STM Tank	0.1797	0.87		0.156			108.3										
										Controlled via orifice (see storage calculation sheet)									
Uncontrolled Sheet Flow to Queen	Site	Queen St E	0.0057	0.90	0.005	0.005	10.0	250.3	3.6										
Controlled Tank Discharge	SWM Tank	Ex. Sewer	0.1797	0.87	0.156				17.1	200	2.00	48.4	1.48	1.35	4.7	0.1	0.1	35.4%	
Total Site Discharge			0.1854	0.87	0.161				20.7	92.7% of Allowable Discharge									

2026 Queen Street East - Mixed Use Development
Stormwater Storage Calculations using Rational Method
100-year Storm - City of Toronto IDF Data

SWM Tank Design

Project No.	22117	Area (ha)	0.1797
Analysis By:	**Angela Mokin**	Total Runoff Coefficient	0.87
Last Revised:	**12 December 2023**	Maximum Site Discharge (L/s)	17.1

Time (min)	Intensity (mm/hr)	Q-100 (cu.m/s)	Q-stored (cu.m/s)	Storage Volume (cu.m)
0	0.0	0.000	0.000	0.000
10	250.3	0.108	0.091	54.718
20	143.8	0.062	0.045	54.104
30	103.9	0.045	0.028	50.133
40	82.6	0.036	0.019	44.650
50	69.1	0.030	0.013	38.286
60	59.7	0.026	0.009	31.338
70	52.8	0.023	0.006	23.972
80	47.4	0.021	0.003	16.290
90	43.2	0.019	0.002	8.360
100	39.7	0.017	0.000	0.230
110	36.8	0.016	0.000	0.000
120	34.3	0.015	0.000	0.000
130	32.2	0.014	0.000	0.000
140	30.3	0.013	0.000	0.000
150	28.7	0.012	0.000	0.000
160	27.2	0.012	0.000	0.000
170	25.9	0.011	0.000	0.000
180	24.8	0.011	0.000	0.000
190	23.7	0.010	0.000	0.000
200	22.8	0.010	0.000	0.000
210	21.9	0.009	0.000	0.000
220	21.1	0.009	0.000	0.000
230	20.4	0.009	0.000	0.000
240	19.7	0.009	0.000	0.000
250	19.1	0.008	0.000	0.000
260	18.5	0.008	0.000	0.000
270	17.9	0.008	0.000	0.000
280	17.4	0.008	0.000	0.000
290	16.9	0.007	0.000	0.000
300	16.5	0.007	0.000	0.000
310	16.0	0.007	0.000	0.000
320	15.6	0.007	0.000	0.000
330	15.3	0.007	0.000	0.000
340	14.9	0.006	0.000	0.000
350	14.6	0.006	0.000	0.000
360	14.2	0.006	0.000	0.000

Storage Volume Required (cu.m)	**54.7**
Storage Volume Provided (cu.m)	**79.2**
Depth at Outlet (m)	**1.97**
Maximum Discharge Flow (L/s)	**17.1**
Outlet Type	**75mm Orifice**

2026 Queen Street East - Mixed Use Development

Water Quality, Initial Abstraction and Water Balance Calculations

Based on WWFMG - City of Toronto

Designed By: **Angela Mokin**, P.Eng.
Checked By: **Angela Mokin**, P.Eng.
File No. 22117
Date: 12 December 2023

Water Quality Management

Inferred Water Quality

			% TSS Removal	Overall
Roof Bare	1389.2	74.9%	80	59.9
Green Roof	88.0	4.7%	80	3.8
Landscape	37.5	2.0%	80	1.6
Permeable	0.0	0.0%	80	0.0
Hard Surface	339.3	18.3%	0	0.0
	1854.0	100%		65.4

* Treatment unit required.

Water Balance Management

Volume Required to be Retained

Required Water Balance (mm):	5
Site Area (m^2):	1,854
Water Balance Volume to be Retained (m^3):	9.3

Post-Development Initial Abstraction

	Area (m^2)		mm	Vol (m^3)	Vol (mm)
Roof Bare	1389.2	74.9%	1	1.4	0.75
Green Roof	88.0	4.7%	5	0.4	0.24
Landscape	37.5	2.0%	5	0.2	0.10
Permeable	0.0	0.0%	5	0.0	0.00
Hard Surface	339.3	18.3%	1	0.3	0.18
	1854.0	100%		2.4	1.27

Initial abstraction (see above) (m^3)	2.4
Water Re-Use: Irrigation & Grey Water System (m^3):	6.9
Provided Volume (m^3):	9.3

Irrigation Volume

Area of Tank	m^2		27.8
Depth of Sump	m		0.3
Total Volume Provided	m^3		8.3

Determining Number of Cartridges for Flow Based Systems

Date	**23/11/2022**	**Black Cells =**	**Calculation**

Site Information

Project Name	**2026 Queen Street E**		
Project Location	**Toronto, ON**		
OGS ID	**OGS**		
Drainage Area, Ad	**0.08**	ac	(0.0305 ha)
Impervious Area, Ai	**0.08**	ac	
Pervious Area, Ap	**0.00**		
% Impervious	**100%**		
Runoff Coefficient, Rc	**0.90**		
Treatment storm flow rate, Q_{treat}	**0.05**	cfs	(1.5 L/s)
Peak storm flow rate, Q_{peak}	**0.73**	cfs	(20.7 L/s)

Filter System

Filtration brand	**StormFilter**	
Cartridge height	**12**	in
Specific Flow Rate	**2.00**	gpm/ft^2
Flow rate per cartridge	**10.00**	gpm

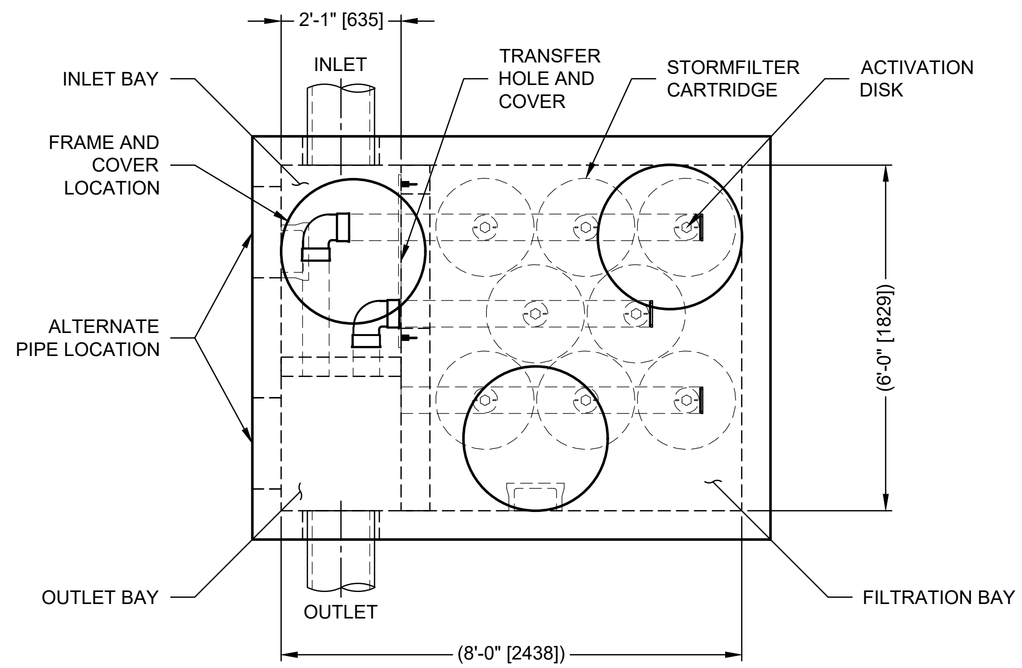
SUMMARY

Number of Cartridges	3
Media Type	Perlite

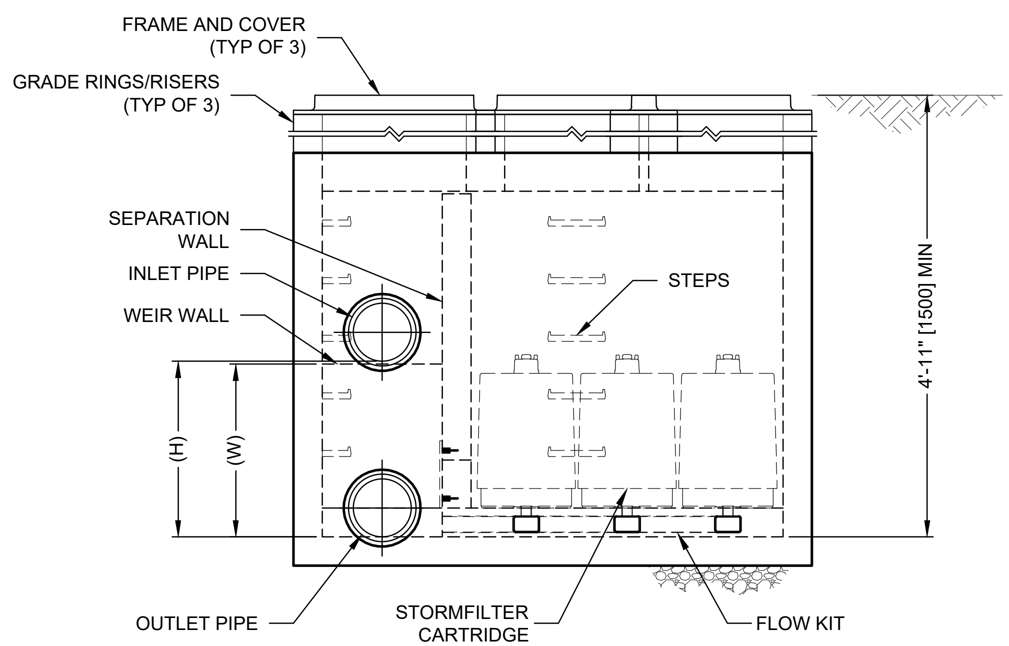
Event Mean Concentration (EMC)	**150**	mg/L
Annual TSS Removal	**80%**	
Percent Runoff Capture	**90%**	

Recommend SFPD0608 vault or CIP

conteches.com
200 Enterprise Drive
Scarborough, ME 04074
Phone 877-907-8676
Fax 207-885-9825



PLAN



ELEVATION

The Stormwater Management StormFilter®

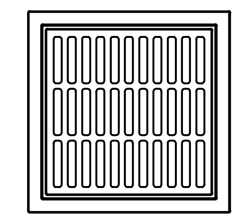
THIS PRODUCT MAY BE PROTECTED BY ONE OR MORE OF THE FOLLOWING U.S. PATENTS: 5,322,629; 5,524,576; 5,707,527; 5,985,157; 6,027,639; 6,649,048; RELATED FOREIGN PATENTS, OR OTHER PATENTS PENDING.

STORMFILTER DESIGN NOTES

- STORMFILTER TREATMENT CAPACITY VARIES BY CARTRIDGE COUNT AND LOCALLY APPROVED SURFACE AREA SPECIFIC FLOW RATE. PEAK CONVEYANCE CAPACITY TO BE DETERMINED BY ENGINEER OF RECORD
- A 6' x 8' [1829 x 2438] PEAK DIVERSION STYLE STORMFILTER IS SHOWN WITH THE MAXIMUM NUMBER OF CARTRIDGES (8) AND IS AVAILABLE IN A LEFT INLET (AS SHOWN) OR A RIGHT INLET CONFIGURATION
- ALL PARTS AND INTERNAL ASSEMBLY PROVIDED BY CONTECH UNLESS NOTED OTHERWISE

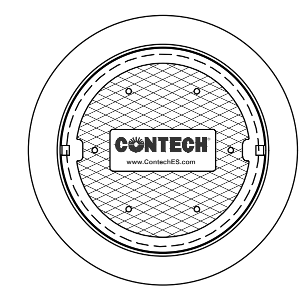
CARTRIDGE SIZE (in. [mm])	27 [686]			18 [457]			LOW DROP		
RECOMMENDED HYDRAULIC DROP (H) (ft. [mm])	3.05 [930]			2.3 [701]			1.8 [549]		
HEIGHT OF WEIR (W) (ft. [mm])	3.00 [914]			2.25 [686]			1.75 [533]		
SPECIFIC FLOW RATE (gpm/sf [L/s/m^2])	2 [1.36]	1.67* [1.13]*	1 [0.68]	2 [1.36]	1.67* [1.13]*	1 [0.68]	2 [1.36]	1.67* [1.13]*	1 [0.68]
CARTRIDGE FLOW RATE (gpm [L/s])	22.5 [1.42]	18.79 [1.19]	11.25 [0.71]	15 [0.95]	12.53 [0.79]	7.5 [0.47]	10 [0.63]	8.35 [0.53]	5 [0.32]

* 1.67 gpm/sf [1.13 L/s/m^2] SPECIFIC FLOW RATE IS APPROVED WITH PHOSPHOSORB® (PSORB) MEDIA ONLY



FRAME AND GRATE

(24" SQUARE)
(NOT TO SCALE)



FRAME AND COVER

(30" ROUND)
(NOT TO SCALE)

SITE SPECIFIC DATA REQUIREMENTS

STRUCTURE ID	
WATER QUALITY FLOW RATE (cfs [L/s])	
PEAK FLOW RATE (cfs [L/s])	
RETURN PERIOD OF PEAK FLOW (yrs)	
CARTRIDGE FLOW RATE	
CARTRIDGE SIZE (27, 18, LOW DROP (LD))	
MEDIA TYPE (PERLITE, ZPG, PSORB)	
NUMBER OF CARTRIDGES REQUIRED	
INLET BAY RIM ELEVATION	
FILTER BAY RIM ELEVATION	

PIPE DATA:	INVERT	MATERIAL	DIAMETER
INLET PIPE 1			
INLET PIPE 2			
OUTLET PIPE			

NOTES/SPECIAL REQUIREMENTS:

PERFORMANCE SPECIFICATION

FILTER CARTRIDGES SHALL BE MEDIA-FILLED, PASSIVE, SIPHON ACTUATED, RADIAL FLOW, AND SELF CLEANING. **RADIAL MEDIA DEPTH SHALL BE 7" [178].** FILTER MEDIA CONTACT TIME SHALL BE AT LEAST **37 SECONDS**. SPECIFIC FLOW RATE SHALL BE **2 GPM/SF [1.36 L/s/m^2] (MAXIMUM)**. SPECIFIC FLOW RATE IS THE MEASURE OF THE FLOW (GPM) DIVIDED BY THE MEDIA SURFACE CONTACT AREA (SF). MEDIA VOLUMETRIC FLOW RATE SHALL BE **6 GPM/CF [13.39 L/s/m^3] OF MEDIA (MAXIMUM).**

GENERAL NOTES

1. CONTECH TO PROVIDE ALL MATERIALS UNLESS NOTED OTHERWISE.
2. DIMENSIONS MARKED WITH ( ) ARE REFERENCE DIMENSIONS. ACTUAL DIMENSIONS MAY VARY.
3. ALTERNATE DIMENSIONS ARE IN MILLIMETERS [mm] UNLESS NOTED OTHERWISE.
4. FOR FABRICATION DRAWINGS WITH DETAILED STRUCTURE DIMENSIONS AND WEIGHTS, PLEASE CONTACT YOUR CONTECH REPRESENTATIVE. www.ContechES.com
5. STORMFILTER WATER QUALITY STRUCTURE SHALL BE IN ACCORDANCE WITH ALL DESIGN DATA AND INFORMATION CONTAINED IN THIS DRAWING. CONTRACTOR TO CONFIRM STRUCTURE MEETS REQUIREMENTS OF PROJECT.
6. STRUCTURE SHALL MEET AASHTO HS20 LOAD RATING, ASSUMING EARTH COVER OF 0' - 10' [3048] AND GROUNDWATER ELEVATION AT, OR BELOW, THE OUTLET PIPE INVERT ELEVATION. ENGINEER OF RECORD TO CONFIRM ACTUAL GROUNDWATER ELEVATION. CASTINGS SHALL MEET AASHTO M306 AND BE CAST WITH THE CONTECH LOGO.

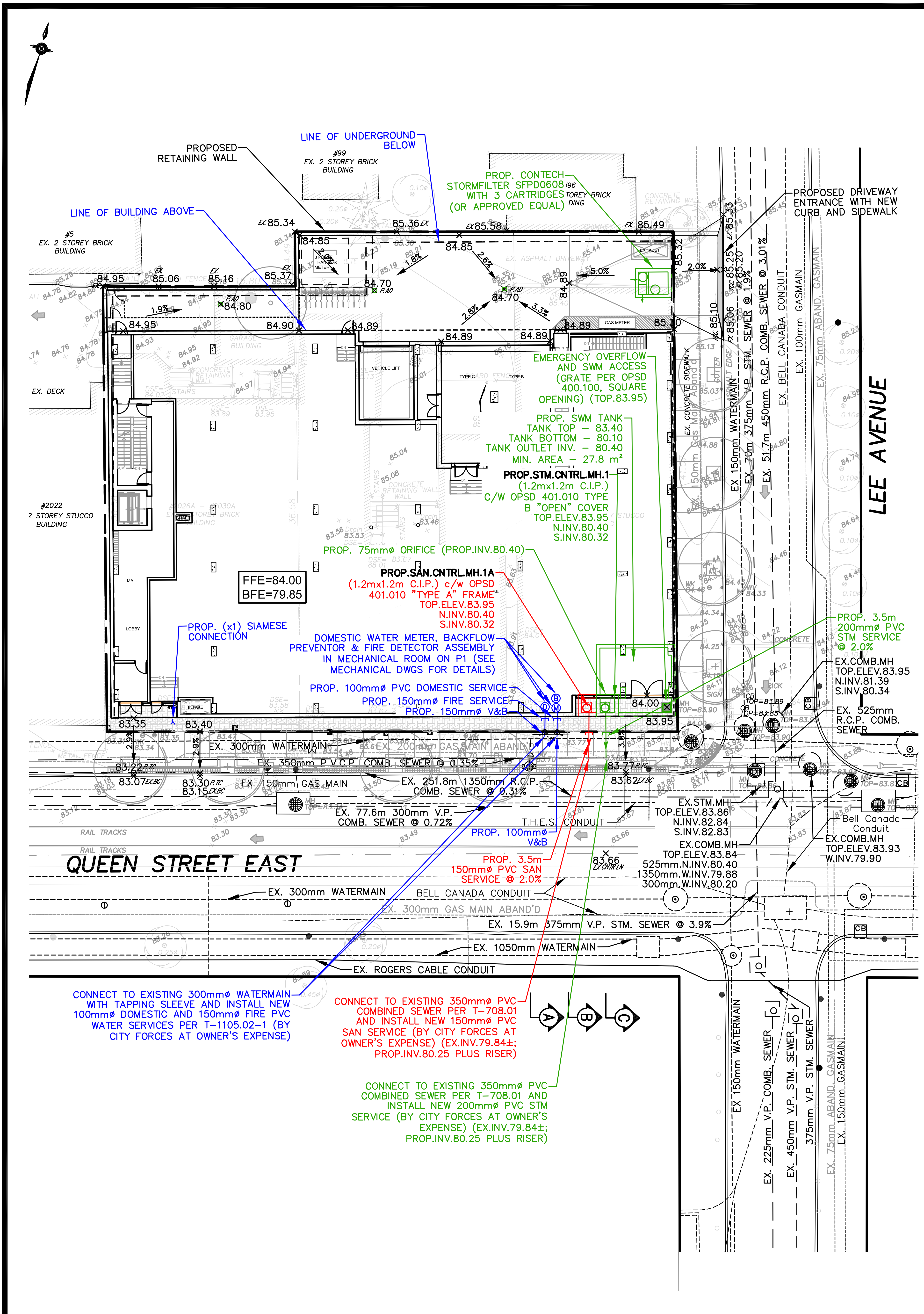
INSTALLATION NOTES

A. ANY SUB-BASE, BACKFILL DEPTH, AND/OR ANTI-FLOTATION PROVISIONS ARE SITE-SPECIFIC DESIGN CONSIDERATIONS AND SHALL BE SPECIFIED BY ENGINEER OF RECORD.
B. CONTRACTOR TO PROVIDE EQUIPMENT WITH SUFFICIENT LIFTING AND REACH CAPACITY TO LIFT AND SET THE STORMFILTER STRUCTURE.
C. CONTRACTOR TO INSTALL JOINT SEALANT BETWEEN ALL SECTIONS AND ASSEMBLE STRUCTURE.
D. CONTRACTOR TO PROVIDE, INSTALL, AND GROUT PIPES. MATCH OUTLET PIPE INVERT WITH OUTLET BAY FLOOR.
E. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT CARTRIDGES FROM CONSTRUCTION-RELATED EROSION RUNOFF.
F. CONTRACTOR TO REMOVE THE TRANSFER OPENING COVER WHEN THE SYSTEM IS BROUGHT ONLINE.

CONTECH®
ENGINEERED SOLUTIONS LLC
www.ContechES.com
9025 Centre Pointe Dr., Suite 400, West Chester, OH 45069
800-338-1122 513-645-7000 513-645-7993 FAX

SFPD0608 (6' x 8')
PEAK DIVERSION STORMFILTER
STANDARD DETAIL

APPENDIX F



DWN. BY:	A.M.
DESIGNED BY:	A.M.
CHECKED BY:	R.F.
SCALE:	1:300
DATE:	DEC 2023
SHEET NO:	1 OF 2

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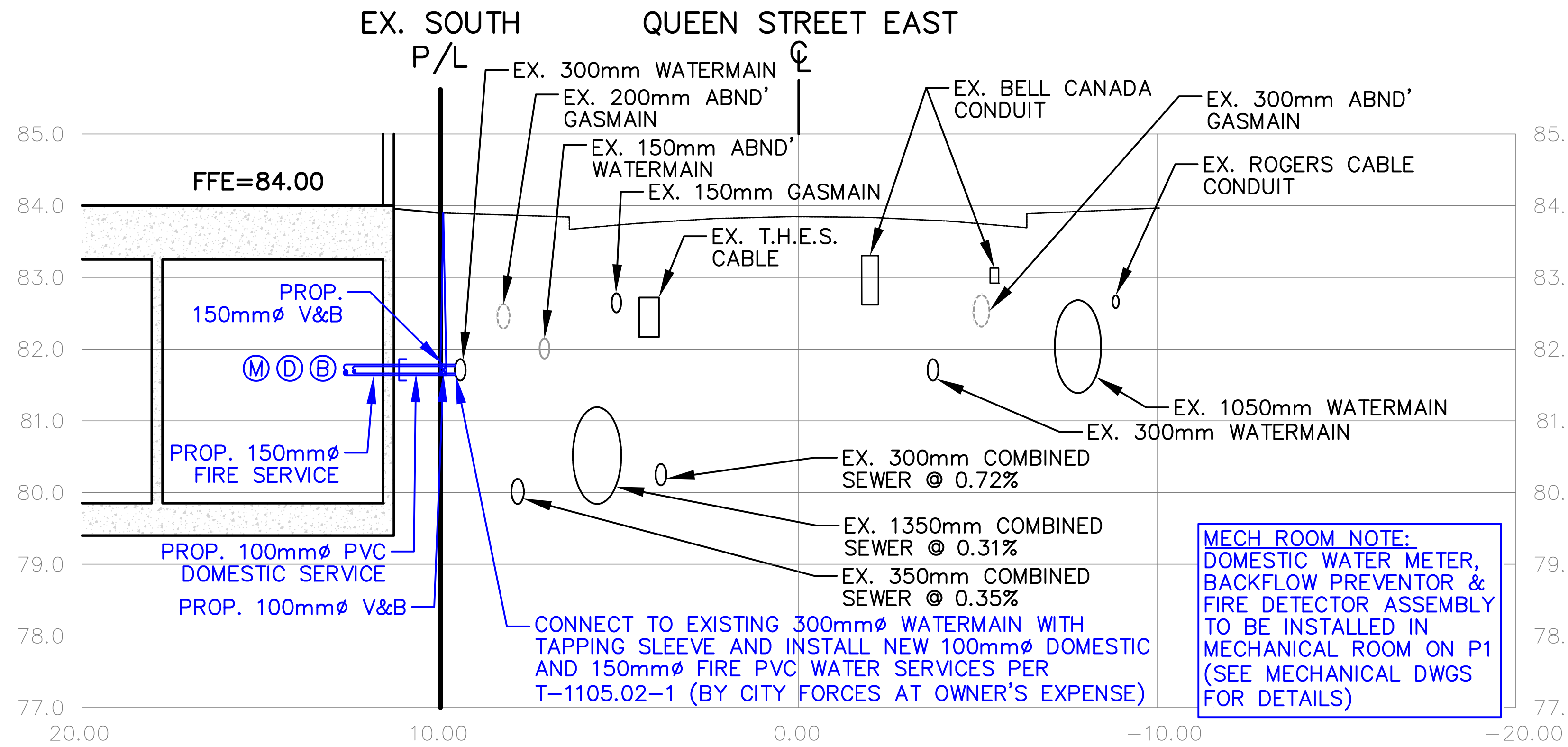
PROJECT NO:	DWG NO:	REV NO:
22117	SSGS-1	0

PROJECT NAME:
2026 QUEEN STREET EAST
MIXED USE DEVELOPMENT

DRAWING TITLE:
SITE SERVICING & GRADING SCHEMATIC



SECTION A–A

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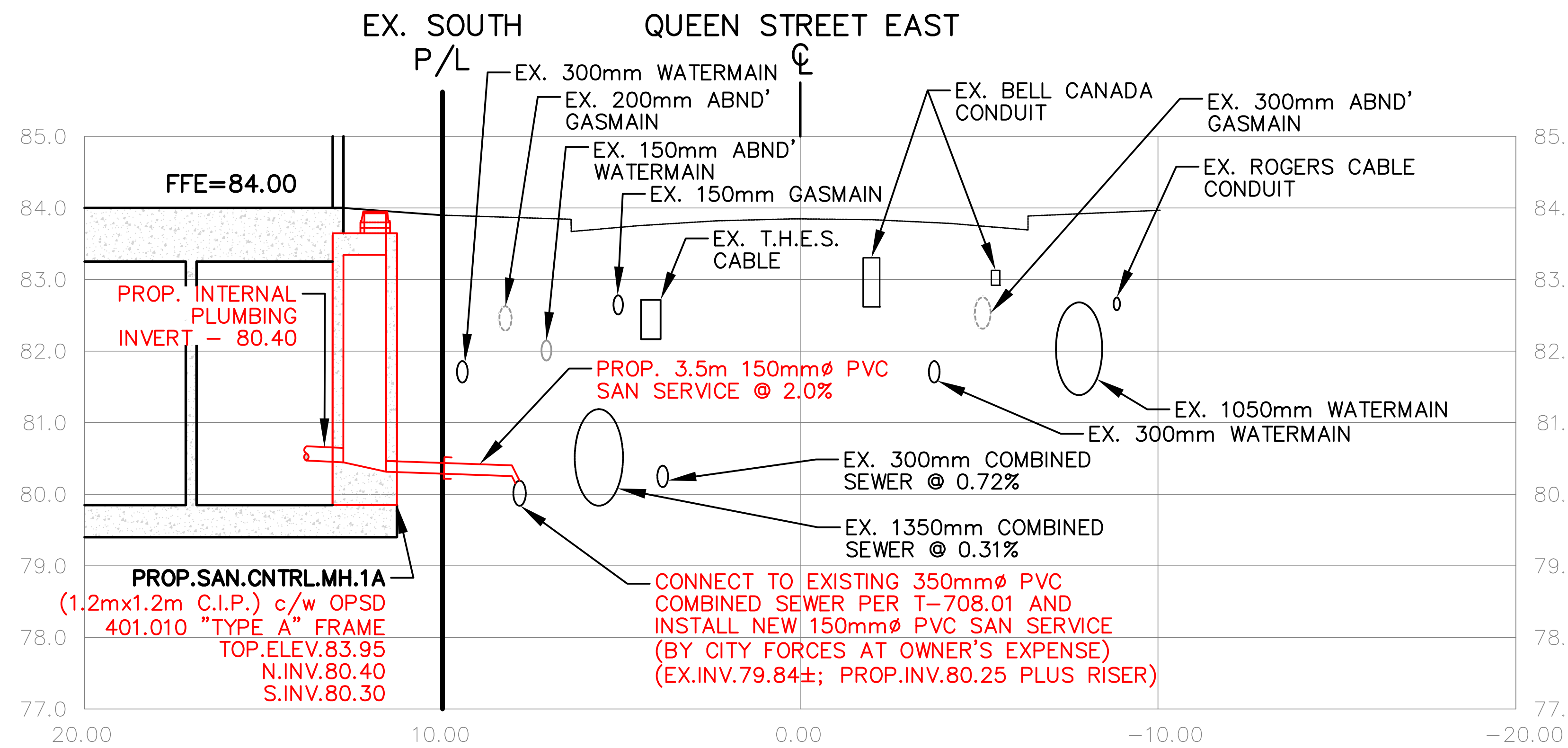

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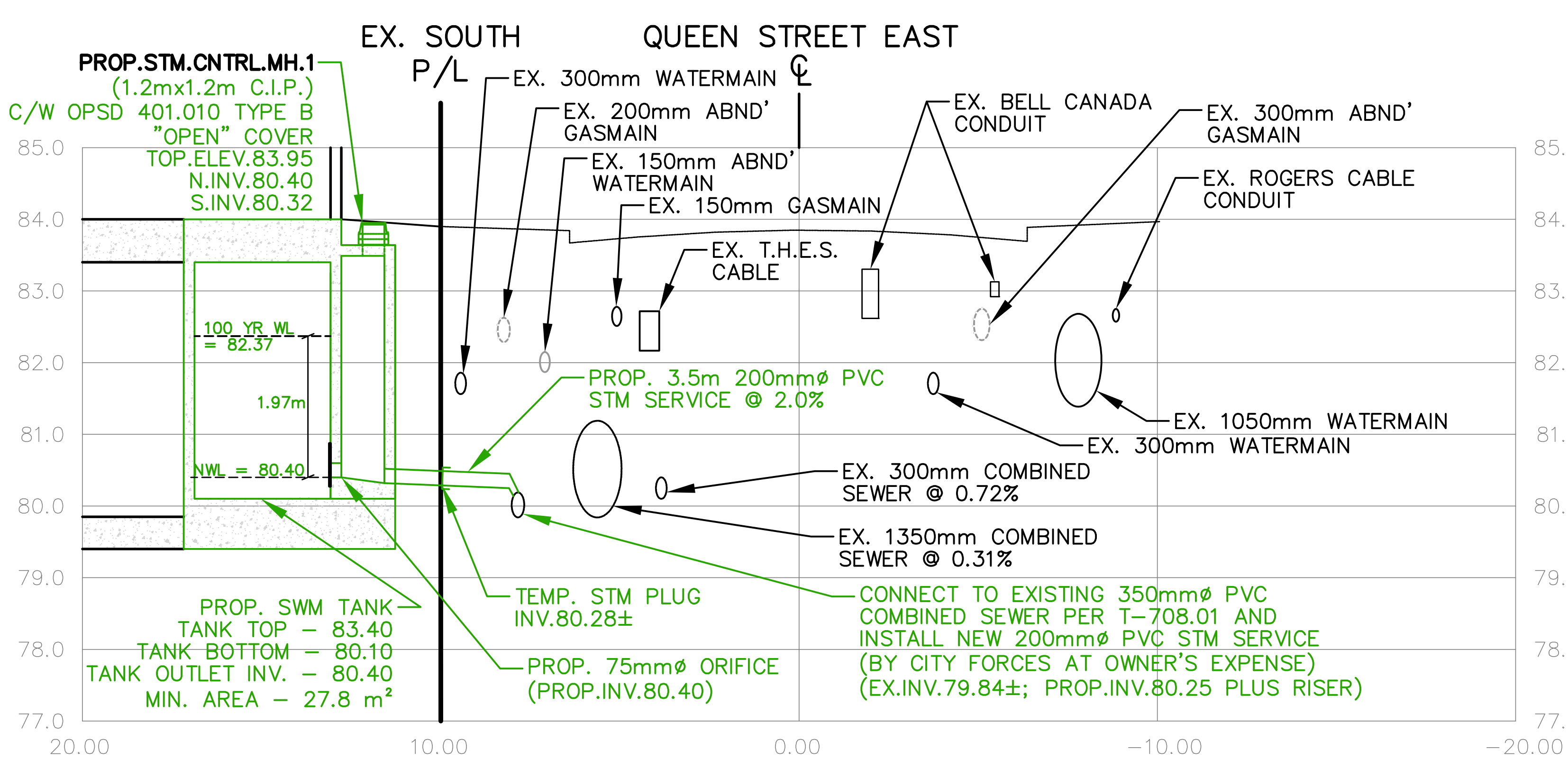

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DWN. BY:	A.M.
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PROJECT NO:	DWG NO:	REV NO:
22117	CS-1	0

PROJECT NAME:
2026 QUEEN STREET EAST
MIXED USE DEVELOPMENT

DRAWING TITLE:
CROSS-SECTION DETAILS

APPENDIX G

Hydrogeology Assessment – Proposed Mixed-use Development

2026 to 2042 Queen Street East, Toronto, Ontario

Prepared for:

Crombie REIT
610 East River Road, Suite 200
New Glasgow, Nova Scotia B2H 3S2

December 12, 2023

Pinchin File: 317395.003

Issued to:	**Crombie REIT**
Issued on:	**December 12, 2023**
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December 12 2023

4.3 Hydraulic Conductivity Estimates

The hydraulic conductivity (K-value) of the soils was estimated based on results obtained from in-situ rising head hydraulic conductivity tests.

Rising head hydraulic conductivity tests were conducted at two monitoring wells (BH/MW101 and BH/MW109) on January 3, 2023. The hydraulic conductivity (K) tests were conducted at five additional on-Site monitoring wells on January 25 and 26, 2023.

The hydraulic conductivity test curves and estimates are provided in Appendix III, and are also summarized below.

MW Notation	Screen Interval	Screened Soils	K-Estimate (cm/sec)
BH/MW101	2.1 - 5.2 (mbgs)	Sand; Silty Sand	7.1×10^{-5}
BH/MW102	0.9 – 4.0 (mbfs)	Silt; Silty Sand	7.5×10^{-6}
BH/MW104	0.6 – 3.7 (mbfs)	Sandy Silt; Silty Sand; Sandy Silt to Silty Sand	1.4×10^{-5}
BH/MW105	0.6 – 3.7 (mbfs)	Sand; Silty Sand; Sandy Silt to Silty Sand	5.4×10^{-6}
BH/MW106	0.9 – 4.0 (mbfs)	Sandy Silt to Silty Sand	5.4×10^{-6}
BH/MW108	1.5 – 4.6 (mbfs)	Sandy Silt to Silty Sand; Silt	5.1×10^{-6}
BH/MW109	10.0 - 13.1 (mbgs)	Silt	3.3×10^{-5}
Geometric mean			1.2×10^{-5}

Note: mbgs = metre below ground surface; mbfs = metre below basement floor surface

The K-values estimated for the cohesionless soils at the Site (including sand, silty sand, silty sand to sandy silt, and silt) ranged from 5.1×10^{-6} cm/sec to 7.1×10^{-5} cm/sec, with a geometric mean of 1.2×10^{-5} cm/sec.

5.0 DEWATERING ASSESSMENT

As stated above in Section 1.2, the proposed development consists of a 6-storey building with one level (P1) of underground parking facility. Based on the design, the established ground floor elevation is at 84 m above sea level (masl) and the P1 Level is at the elevation of 79.85 masl.

Based on the groundwater monitoring completed between January and June 2023, the measured groundwater levels ranged from 81.16 masl to 82.69 masl, which is higher than the P1 level elevation. Therefore, groundwater control to lower the groundwater levels shall be considered during the construction phase and during the building operations.

It should be noted that The City of Toronto has enacted a new Foundation Drainage Policy (Policy) and published Foundation Drainage Guidelines (Guidelines), which came into effect on January 1, 2022. The new Policy does not allow long-term discharge of foundation drainage into the City's sewer system.

The Guidelines provide minimum groundwater monitoring, groundwater level & flow estimation analysis and submission requirements to support development application review by the City.

Based on the guidelines, Maximum Anticipated Groundwater Level (GWL) shall be calculated by adding a fluctuation allowance to the peak measured static GWL according to the following methods:

- Option 1 – Flexible, Year-round Monitoring Approach (a minimum of three (3) static GWL measurements, taken every two weeks), upon which a fluctuation allowance for a measurement month shall be added to the highest measured static GWL.
- Option 2 – Peak Season Monitoring Approach (a minimum of six (6) static GWL measurements, taken every two weeks, within the months of April, May and June), upon which a fluctuation allowance of 0.8 m shall be added to the peak measured static GWL.

As Peak Season groundwater monitoring has been conducted capturing the highest groundwater levels of 82.69 masl, the dewatering calculations below will use Option 2 methodology and the Maximum Anticipated Groundwater Level will thus be 83.49 masl.

It should be noted that because long-term discharge of groundwater from a subdrainage system is not allowed by the City of Toronto, a water-proof foundation will have to be considered in the building design to avoid the long-term discharge.

5.1 Short-Term Dewatering Estimates

5.1.1 Groundwater Inflow

As indicated, the Peak Season option was used in the dewatering assessment for the Site. Based on the design and the anticipated construction, groundwater dewatering during construction was estimated using the following parameters and assumptions:

- The excavation/dewatering area is 1,854 m^2 (whole site area assumed);
- The Maximum Anticipated Groundwater Level is 83.49 masl;
- The excavation bottom for water-proof structure construction is assumed to be 3 m below the P1 level, which will be 76.85 masl, and the target water level will be at 76.35 masl (dewatering assumed to extend to 0.5 m below the excavation bottom);
- The hydraulic conductivity is 7.1 x 10^{-5} cm/sec (the highest hydraulic conductivity estimated at the Site).

Based on the above assumptions, the short-term construction dewatering rate and zone of influence were estimated and are presented below.

Dewatering Area (m^2)	Initial Water Level (masl)	Target Water Level (masl)	K-Estimate (cm/sec)	Estimated Maximum Zone of Influence (m from Edge of Excavation)	Dewatering Rate (without Safety Factor) (L/day)	Dewatering Rate Estimate with Safety Factor of 2 or 100% (L/day)
1,854	83.49	76.35	7.1×10^{-5}	17	18,177	36,354

It should be noted that the application of a Safety Factor provides a more conservative assessment for planning purposes to account for potential variabilities in the hydraulic conductivities in the soil across the Site. In addition, during the initial stages of the construction dewatering, the dewatering volumes would be greater than those under a steady state condition, because the water stored in the soils is also being removed.

It is understood that water taking from groundwater in Ontario is governed under Part II.2 of the Environmental Protection Act (EPA or Act), Ontario Regulation (O. Reg.) 245/11 (Registrations Under II.2 of the Act – General) and O. Reg. 63/16 (Registrations under II.2 of the Act – Water Taking), Ontario Regulation 387/04 and Ontario Water Resources Act (OWRA).

5.1.2 Stormwater Inflow

It should be noted that a significant amount of the dewatering demand from any construction project is the volume of water that is derived from stormwater that can accumulate in the excavation area during and after precipitation events.

For planning purposes, dewatering estimates are developed assuming the potential occurrence of “Design Storm” events, which are based upon events that have an observed “return period” or period of recurrence. Based on the historical climatic data for 1981 to 2010 for Toronto Pearson Airport weather station, the number of days when the precipitation is greater than 25 mm per day is observed to be 0.9 days/year. For the Design Storm inflow estimation, precipitation is assumed to be 30 mm/day.

The Design Storms represent singular events which will entail the handling of larger volumes of water over a short period of time. In the case of the proposed development, the handling stormwater over the excavation area will likely be required.

The volumes of stormwater that can be generated within the excavation for the underground footprint area were estimated for a 30 mm/day Design Storm event. The estimated stormwater inflow is summarized below:

Excavation Area (m^2)	Precipitation Depth (mm)	Stormwater Volume (L/day)
1,854	30.0	55,620

The dewatering requirement from a high-precipitation storm with a rate of 30 mm/day is estimated to be 55,620 L/day. It should be noted that the above estimate does not take into account any infiltration or evaporation in the excavation area. However, for infrequent extreme storm events, the great majority of the generated stormwater becomes run-off or accumulates in the excavation area, due to the fixed assimilative capacity of the soils and the minimal evaporation until the cessation of the event.

5.1.3 Summary of Construction Dewatering Estimates

Based on the short-term construction dewatering calculations discussed above, the estimated maximum construction phase dewatering rates are summarized below.

Construction Dewatering	Total Volume without Safety Factor for Groundwater (L/day)	Total Volume with Safety Factor of 2 for Groundwater (L/day)
Discharge of Groundwater	18,177	36,354
Discharge of Stormwater	55,620	55,620
Discharge of Groundwater and Stormwater	73,797	91,974

Based on the dewatering calculations as described above, the estimated dewatering inflows from groundwater and Design Storm surface water inflows, taking into account a Safety Factor of 2, or 100%, are above the threshold for an Environmental Activity Sector Registration (EASR) requirement for construction dewatering of more than 50,000 L/day (50 m^3/day) bit are below the threshold limit of 400,000 L/day (400 m^3/day) for a Permit-to-Taka-Water (PTTW) requirement. An EASR registration will be required for the construction of the proposed building

5.2 Long-Term Dewatering Estimate - Operations

Because discharge of groundwater from a subdrainage system is not allowed by the City of Toronto, a water-proof foundation will have to be considered in the building design to avoid long-term discharge. Therefore, long-term dewatering from groundwater is not anticipated and a PTTW will not be required for the building operations.

A standby internal weeping tile below the P1 level slab may be proposed as a precautionary measure in the event of a leak through some part of the water-proofing over the life of the building.

5.3 Hydraulic Uplift Estimate

It is understood that the proposed building would be constructed using a raft/watertight structure, in which no dewatering will be required. However, given the relatively high groundwater elevation, the building will have to be designed to resist hydrostatic uplift.

The magnitude of the hydrostatic uplift may be calculated using the following formula:

$$P = \gamma \times d$$

Where:

P = hydrostatic uplift pressure acting on the base of the structure (kPa)

γ = unit weight of water (9.8 kN/m^3)

d = depth of base of structure below the design high water level (m)

The estimated hydraulic uplift value for the proposed structure is presented as below.

Design Groundwater Elevation (masl)	Assumed Structure Base Elevation (masl)	Depth to Structure Base (m)	Calculated Hydraulic Uplift (kPa)
83.49	76.85	6.64	65.07

6.0 GROUNDWATER QUALITY

The City requires the proponent to characterize the expected dewatering discharge quality in order to evaluate options for the discharge water.

An unfiltered groundwater sample was obtained from one of the on-site monitoring wells at BH/MW101 (sample ID: MW101) on January 4, 2023 to evaluate the water quality with reference to the City of Toronto Sewer Use By-Law parameter criteria, for storm sewer, sanitary sewer, and/or combined sewer discharge. In addition, a lab-filtered water sample was analyzed for selected dissolved metals including copper, manganese, nickel and zinc to assess the effect of filtration treatment of sediment. Moreover, resampling was conducted at BH/MW101 on January 25, 2023 for analysis of total polycyclic aromatic hydrocarbons (18 PAHs).

The groundwater samples were submitted to and analyzed by Bureau Veritas (BV), which is a lab accredited by Canadian Association For Laboratory Accreditation Inc. (CALA). The Certificates of Analysis are presented in Appendix IV.

The analytical results were compared with the relevant standards (City of Toronto Sewer Use bylaw – Sanitary and Storm Sewer Discharge Limits). Exceedances of the Storm Sewer Discharge Limits were measured in the initial water samples for five parameters, including total suspended solids (TSS), copper manganese (total and dissolved manganese), nickel, zinc and PAHs. The identified exceedances are presented in Table below.

Monitoring Well	Sample ID	Parameter	Unit	Storm Water Guideline Value	Sanitary Sewer Guideline Value	Measured Concentration
BH/MW101	MW101	TSS	mg/L	**15**	350	48
		Copper	mg/L	**0.04**	2	0.1
		Manganese	mg/L	**0.050**	5	0.11
		Nickel	mg/L	**0.080**	2	0.12
		Zinc	mg/L	**0.040**	2	0.19
		Total PAHs (18 PAHs)	ug/L	**2**	5	3
		Dissolved Manganese	mg/L	**0.050**	5	0.092

It should be noted that no exceedances were found for the total PAHs in the re-sampled well water or for the dissolved metals in the filtered sample, except for dissolved manganese. Elevated manganese is a common occurrence in the shallow groundwater in the Greater Toronto Area. The individual PAH compounds were all measured below the laboratory detection limit in the re-sampled water. It is concluded that the initial MW-1 sample for PAHs was contaminated in some fashion.

Based on the chemical testing, the groundwater at the Site may not meet the City's Storm Sewer bylaw standards because of the elevated concentration of manganese, and the water generated during the construction dewatering cannot be discharged into the local sewer system without adequate treatment prior to discharge. Filtration and special treatment to minimise the concentration of manganese will need to be considered if the storm sewer system is selected as a discharge receiver during the construction phase.

7.0 CONCLUSIONS

Pinchin provides the following conclusions arising out of the Hydrogeology Assessment activities to date:

- The Site is located in the Lake Ontario Waterfront Subwatershed within the jurisdiction of the Toronto and Region Conservation Authority (TRCA). No open water body is present on or near the Site. Lake Ontario is located approximately 550 m south of the Site;

- The Site is located on the Iroquois Plain Physiographic Region, and the Sand Plains Physiographic Landform and deposited with coarse-textured glaciolacustrine deposits or foreshore and basinal deposits of sand, gravel, minor silt and clay, and is underlain by Georgian Bay Formation bedrock;
- In general, the soil stratigraphy at the Site comprises upper cohesionless soils under the pavement structure and fill material, which consist of sand, silt sand and silt, underlain by cohesive soils of clayey silt and/or silty clay till, then by lower cohesionless soils of sand, overlying shale bedrock. The shale bedrock was encountered at the depths ranging from approximately 24.4 mbgs to 25.6 mbgs;
- The groundwater levels measured at the Site in from January to June 2023 ranged from 0.12 mbfs to 3.14 mbgs, and groundwater elevations ranged from 81.16 masl to 82.69 masl, with the highest groundwater level measured in April 2023 during the Peak Season between April and June. The inferred groundwater flow direction was generally towards the south/southeast;
- The K-values estimated for the cohesionless soils at the Site ranged from 5.1×10^{-6} cm/sec to 7.1×10^{-5} cm/sec, with a geometric mean of 1.2×10^{-5} cm/sec;
- The dewatering assessments indicated that the maximum short-term construction dewatering, including groundwater and stormwater inflows, is anticipated to be 91,974 L/day, with a safety factor of 2 considered for groundwater inflow. An EASR registration will be required for the construction dewatering activities;
- Because discharge of groundwater from a subdrainage system is not allowed by the City of Toronto, a water-proof foundation will have to be considered in the building design. As such, long-term dewatering from groundwater is not anticipated and a PTTW will not be required during the building operations; and
- A groundwater quality assessment completed as per the City Sewer Use Bylaw indicated the presence of exceedances in the tested water samples. Should the excess water be discharged into the local sewer system, filtration and appropriate treatment for manganese will be required, prior to discharge.

8.0 REFERENCES

1) L.J. Chapman and D.F. Putnam, 2007. *Physiography of Southern Ontario*; Ontario Geological Survey, Miscellaneous Release--Data 228.

2) Ontario Geological Survey 2010. *Surficial Geology of Southern Ontario*; Ontario Geological Survey, Miscellaneous Release--Data 128-REV.