1299 RICHMOND ROAD – SERVICING AND STORMWATER MANAGEMENT REPORT

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Prepared for:
11034936 Canada Inc.

Prepared by:
Stantec Consulting Ltd.

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Prepared by: _____
Signature

Peter Mott, EIT
Printed Name

Reviewed by: _____
Signature

Kris Kilborn
Printed Name

Approved by: _____
Signature

Peter Moroz, P.Eng.
Printed Name

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1.0 Introduction

Stantec Consulting Ltd. has been commissioned by 11034936 Canada Inc. to prepare the following Servicing and Stormwater Management Report in support of a Major Zoning By-law Amendment and Site Plan Control application for the proposed development located at 1299 Richmond Road in the City of Ottawa.

The site is 0.453 ha in area and is situated along the north side of Richmond Road, the south and west sides of Starflower Lane, and the east side of Assaly Road. The site is currently zoned AM10 and consists of an existing commercial strip mall and surface parking lot. The site is bounded by Richmond Road to the south, Starflower Lane to the east and north, and Assaly Road to the west, as shown in **Figure 1-1** below.

Figure 1-1: Key Plan of Site

The proposed development will comprise of a single mixed-use building with a 5-storey podium, two residential high-rise towers, and a 0.04 ha park along the west boundary of the site. The proposed buildings will include 1 bachelor unit, 313 one-bedroom units, 2 one-bedroom units with dens, 39 two-bedroom units, 214 two-bedroom units with dens, 21 three-bedroom units, and 735 m^2 of retail space. Quadrangle Architects Ltd. has prepared a site plan which defines the proposed development (see **Appendix B**).

1.1 Objective

This site servicing and stormwater management (SWM) report presents a servicing scheme that is free of conflicts, provides on-site servicing in accordance with City of Ottawa Design Guidelines, and uses the existing municipal infrastructure in accordance with any limitations communicated during consultation with the City of Ottawa staff. Details of the existing infrastructure located within the Richmond Road, Assaly Road, and Starflower Lane right of ways (ROW) were obtained from available as-built drawings and site topographic survey.

Criteria and constraints provided by the City of Ottawa have been used as a basis for the servicing design of the proposed development. Specific and potential development constraints to be addressed are as follows:

- **Potable Water Servicing**
    - Estimated water demands to characterize the proposed feed(s) for the proposed development which will be serviced from either the existing 305 mm diameter watermain within the Richmond Road ROW or the existing 152 mm diameter watermain within the Assaly Road ROW, or both.
    - Watermain servicing for the development is to be able to provide average day and maximum day (including peak hour) demands (i.e., non-emergency conditions) at pressures within the acceptable range of 345 to 552 kPa (50 to 80 psi)
    - Under fire flow (emergency) conditions, the water distribution system is to maintain a minimum pressure greater than 140 kPa (20 psi)
- **Wastewater (Sanitary) Servicing**
    - Define and size the sanitary service lateral which will be connected to the existing 300 mm diameter sanitary sewer within the Richmond Road ROW.
- **Storm Sewer Servicing**
    - Define major and minor conveyance systems in conjunction with the proposed grading plan.
    - Determine the stormwater management storage requirements to meet the allowable release rate for the site.
    - Define and size the proposed storm service lateral that will be connected to the existing 450 mm diameter municipal storm sewer within the Richmond Road ROW.
- Prepare a grading plan in accordance with the proposed site plan and existing grades.

The accompanying drawings included in **Appendix G** of this report illustrate the proposed internal servicing scheme for the site.

2.0 Background

Documents referenced in preparing of this stormwater and servicing report for the 1299 Richmond Road development include:

- *City of Ottawa Sewer Design Guidelines* (SDG), City of Ottawa, October 2012, including all subsequent technical bulletins
- *City of Ottawa Design Guidelines – Water Distribution*, City of Ottawa, July 2010, including all subsequent technical bulletins
- *Design Guidelines for Drinking Water Systems*, Ministry of the Environment, Conservation, and Parks (MECP), 2008
- *Fire Protection Water Supply Guideline* for Part 3 in the Ontario Building Code, Office of the Fire Marshal (OFM), October 2020
- *Water Supply for Public Fire Protection*, Fire Underwriters Survey (FUS), 2020
- *Geotechnical Investigation, Proposed Multi-Storey Building – 1299 Richmond Road, Ottawa, Ontario,* Patterson Group, April 25, 2023
- *Stormwater Management Guidelines for the Pinecrest Creek/Westboro Area Final Report,* Prepared for Planning and Infrastructure, City of Ottawa by J.F. Sabourin and Associates Inc., May 2019.

3.0 Water Servicing

3.1 Background

The proposed building is in Pressure Zone 1W of the City of Ottawa's Water Distribution System. The existing watermains along the boundaries of the site consists of a 305 mm diameter cast iron watermain within Richmond Road and a 152 mm diameter cast iron watermain within Assaly Road. There are existing fire hydrants on Richmond Road and Assaly Road. The existing strip mall on site is presently serviced by a building service lateral connection to the 305 mm diameter watermain on Richmond Road, which would be removed by City Staff and blanked at the watermain as shown in the Existing Conditions and Removals Plan (see **Drawing EX-1** in **Appendix G**).

3.2 Water Demands

3.2.1 POTABLE (DOMESTIC) WATER DEMANDS

Water demands were estimated based on the unit mix of the site plan provided by Quadrangle Architects Ltd. (see **Appendix B**) The development comprises of two mixed-use high rise towers, one 28-storey and the other 32-storey, with a five-storey podium and consists of 1 bachelor unit, 313 one-bedroom units, 2 one-bedroom units with dens, 39 two-bedroom units, 214 two-bedroom units with dens, 21 three-bedroom units, and 735 m^2 of retail space.

The City of Ottawa Water Distribution Guidelines (July 2010) and ISTB 2021-03 Technical Bulletin were used to determine water demands based on projected population densities for residential areas and peaking factors. The population was estimated using an occupancy of 1.4 persons per unit for bachelor and one-bedroom apartments, 2.1 persons per unit for one-bedroom apartments with den and two-bedroom apartments, and 3.1 persons per unit for two-bedroom apartments with den and three-bedroom apartments.

A daily rate of 280 L/cap/day has been used to estimate average daily (AVDY) potable water demand for the residential units, and 28,000 L/gross ha/day for the commercial spaces. Maximum day (MXDY) demands were determined by multiplying the AVDY demands by a factor of 2.5 for residential areas and 1.5 for commercial areas. Peak hourly (PKHR) demands were determined by multiplying the MXDY by a factor of 2.2 for residential areas and 1.8 for commercial areas. The estimated demands for each commercial and residential plot are summarized in **Table 3-1** below.

Table 3-1: Estimated Water Demands

	Comm. Area (m^2)	Total Apartment Units	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Building	735.0	590	1254	4.09	10.20	22.42

3.2.2 FIRE FLOW DEMANDS

Based on the site plan, the fire flow requirement was calculated in accordance with Fire Underwriters Survey (FUS) methodology. Through correspondence with the architect (see **Appendix A.4**), the building will be constructed out of cast-in-place concrete with non-combustible cladding, sprinklered, and the vertical separations between the floors will be equipped with all the necessary fire separations required by code.

As such, they were estimated based on a building of non-combustible construction type with two-hour fire rated structural members, and full protections of all vertical openings (one hour fire rating), and the final sprinkler design to conform to the NFPA 13 standard. The gross floor area of the largest floor + 25 % of the gross floor area of two additional floors were used in the FUS calculation for the two high-rises, as per Page 22 of the *Fire Underwriters Survey's Water Supply for Public Fire Protection* (2020).

Therefore, the building's required fire flow was determined to be 100.0 L/s (6,000 L/min). Detailed fire flow calculations per the FUS methodology are provided in **Appendix A.2**.

3.3 Level of Servicing

3.3.1 BOUNDARY CONDITIONS

The estimated domestic potable water demands, and fire flow demands, were used to define the level of servicing required for the proposed development from the municipal watermain and hydrants within the Richmond Road ROW. **Table 3-2** outlines the boundary conditions provided by the City of Ottawa on May 18, 2023 (See **Appendix A.3** for correspondence).

Table 3-2: Boundary Conditions

Connection	Richmond Road
Min. HGL (m)	108.3
Max. HGL (m)	115.9
MXDY + FF (100.0 L/s) (m)	109.4

3.3.2 ALLOWABLE DOMESTIC PRESSURES

The desired normal operating pressure range in occupied areas as per the City of Ottawa 2010 Water Distribution Design Guidelines is 345 kPa to 552 kPa (50 psi to 80 psi) under a condition of maximum daily flow and no less than 276 kPa (40 psi) under a condition of maximum hourly demand. Furthermore, the maximum pressure at any point in the water distribution should not exceed 689 kPa (100 psi) as per the Ontario Building/Plumbing Code; pressure reducing measures are required to service areas where pressures greater than 552 kPa (80 psi) are anticipated in occupied areas.

The proposed finished floor elevation at the ground floor of 70.2 m will serve as the ground floor elevation for the calculation of residual pressures at ground level. As per the boundary conditions, the on-site pressures are expected to range from 374.0 kPa to 448.5 kPa (54.2 psi to 65.0 psi) under normal operating conditions, which are within the normal operating pressure range defined by the City of Ottawa design

guidelines as within 276 kPa to 552 kPa (40 psi to 80 psi). It is anticipated that booster pumps will be required to service the upper floors of the townhouses and towers.

Calculations of the residual pressures under normal operating conditions have been provided in **Appendix A.5**.

3.3.3 ALLOWABLE FIRE FLOW PRESSURES

The boundary conditions provided by the City of Ottawa indicate that the 305 mm diameter watermain within Richmond Road is expected to maintain a residual pressure of 39.25 m equivalent to 384.8 kPa (55.8 psi) under the worst-case fire flow conditions. This demonstrates that with the upsizing and complete looping, the watermains and nearby hydrants can provide the required fire flows while maintaining a residual pressure of 20 psi.

3.3.4 FIRE HYDRANT COVERAGE

The building will be sprinklered and a Siamese (fire department) connection is to be provided to the right of the main entrance. There are three fire hydrants in the proximity of the proposed development site, as shown in **Figure 3-1**. The distance of each hydrant from the proposed building is less than 115 m.

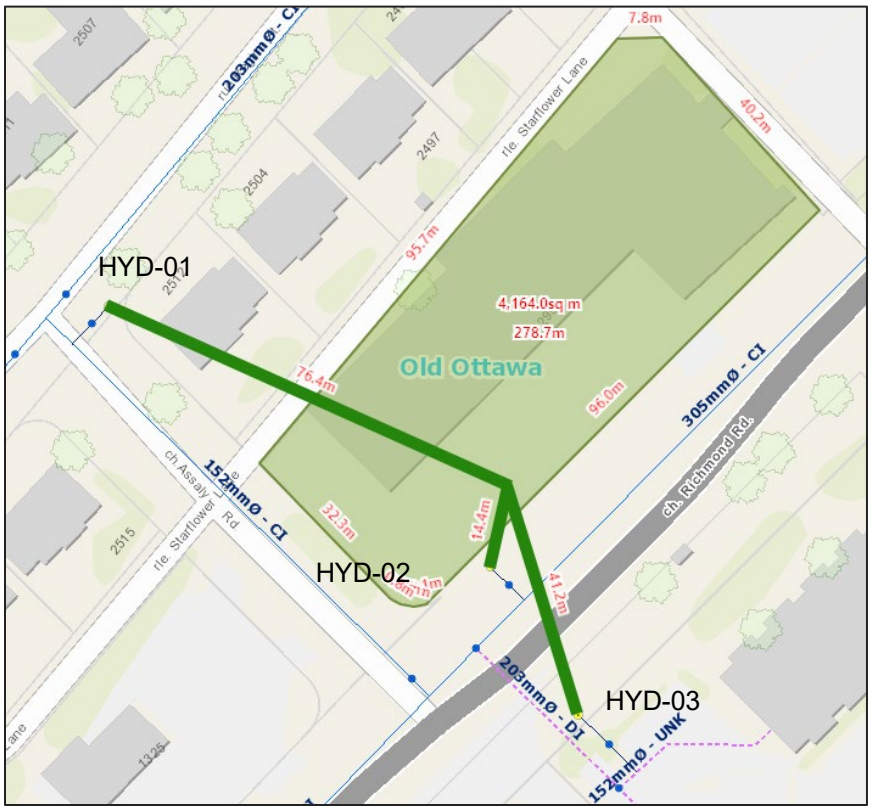

Figure 3-1: Fire Hydrant Coverage Sketch

According to the NFPA 1 Table 18.5.4.3 and as referenced in Technical Bulletin ISTB-2018-02 by the City of Ottawa, a hydrant situated less than 76 m away from a building can supply a maximum capacity of 5,678 L/min. Hence, the required fire flow for this site (6,000 L/min) can be achieved with HYD-02 and HYD-03. See **Appendix A.6** for fire hydrant coverage table calculations and NFPA 1 Table 18.5.4.3.

As per Section 3.2.5.16 of the Ontario Building Code, the distance between the fire department connection and hydrant must be unobstructed and cannot be more than 45 m. As such, the site is will served by HYD-02, which can provide the adequate fire flows from an unobstructed distance less than 45 m to the fire department connection and meet the OBC requirements.

3.4 Proposed Water Servicing

The development will be serviced by two 200 mm building service connections to the 305 mm diameter watermain on Richmond Road. The sizing of the service connections is to be confirmed by the mechanical consultant. The proposed water servicing is shown on **Drawing SSP-1** contained in **Appendix G**. Based on the City of Ottawa Water Design Guidelines, the 305 mm diameter watermain on Richmond Road can provide adequate fire and domestic flows for the subject site.

Thermal insulation is required on the water service laterals in the connection to the watermain on Richmond Road, as there is less than 2.4 m cover provided per W22. Booster pumps are required for the towers. The mechanical consultant or plumbing contractor will ultimately be responsible to confirm building pressures are adequate to meet building code requirements.

4.0 Wastewater Servicing

The site will be serviced from the existing 300 mm diameter concrete sanitary sewer on Richmond Road. The existing strip mall on site is presently serviced by a sanitary service lateral connected to the existing 300 mm diameter sanitary sewer. The lateral and manholes will be decommissioned, capped, and abandoned at the property line per City Standard S11.4, as shown in Existing Conditions and Removals Plan (see **Drawing EX-1** in **Appendix G**).

4.1 Design Criteria

As outlined in the City of Ottawa Sewer Design Guidelines and the MECP Design Guidelines for Sewage Works, the following criteria were used to calculate the estimated wastewater flow rates and to determine the size and location of the sanitary service lateral:

- Minimum velocity = 0.6 m/s (0.8 m/s for upstream sections)
- Maximum velocity = 3.0 m/s
- Manning roughness coefficient for all smooth wall pipes = 0.013
- Minimum size of sanitary sewer service = 135 mm
- Minimum grade of sanitary sewer service = 1.0 % (2.0 % preferred)
- Average wastewater generation = 280 L/person/day (per City Design Guidelines)
- Peak Factor = based on Harmon Equation; maximum of 4.0 (residential)
- Harmon correction factor = 0.8
- Infiltration allowance = 0.33 L/s/ha (per City Design Guidelines)
- Minimum cover for sewer service connections – 2.0 m
- Population density for bachelor and one-bedroom apartments – 1.4 persons/apartment
- Population density for one-bedroom with den and two-bedroom apartments – 2.1 persons/apartment
- Population density for two-bedroom with den and three-bedroom apartments – 3.1 persons/apartment
- Average commercial wastewater generation – 28,000 L/ha/day of building space

4.2 Wastewater Generation and Servicing Design

A sanitary sewer design sheet was prepared and is included in **Appendix C.1.** The estimated wastewater flows to be generated are based on the current site plan and consists of 1 bachelor unit, 313 one-bedroom units, 2 one-bedroom units with dens, 39 two-bedroom units, 214 two-bedroom units with dens, 21 three-bedroom units, and 735 m^2 of retail space. The peak wastewater flows were calculated to be 15.35 L/s for the entire site. The anticipated wastewater peak flow generated from the proposed development is summarized in **Table 4-1** - Estimated Total Wastewater Peak Flow below:

The lands to be conveyed to the proposed park do not form part of the site plan development and were not considered as part of the sanitary sewage calculations.

Table 4-1 - Estimated Total Wastewater Peak Flow

Residential Units				Commercial Areas			Infiltration Flow (L/s)	Total Peak Flow (L/s)
Number of Units	Population	Peak Factor	Peak Flow (L/s)	Area (ha)	Peak Factor	Peak Flow (L/s)		
590	1254	3.73	15.18	0.074	1.5	0.04	0.14	15.35
Total Estimated Wastewater Peak Flow (L/s):								**15.35**

1. Design residential flow based on 280 L/p/day and design commercial flow based on 28,000 L/ha/day.
2. Peak factor for residential units calculated using Harmon's formula and taken as 1.50 for commercial areas.
3. Residential population estimated based on 1.4 persons/unit for bachelor and one-bedroom apartments, 2.1 persons/unit for one-bedroom units with dens and two-bedroom units, and 3.1 persons/unit for two-bedroom units with dens and three-bedroom units.
4. Infiltration design flow equals 0.33 L/s/ha.

Design of internal plumbing and associated mechanical systems for the buildings on site is to be completed by the buildings' mechanical engineer. A backflow preventer will be required for the proposed building in accordance with the City of Ottawa Sewer Design Guidelines. This requirement will be coordinated with the building's mechanical engineer.

The anticipated peak wastewater flows for the proposed development were provided to the City of Ottawa staff to conduct a capacity analysis of the sanitary sewer system in the vicinity of the site and downstream system. Assuming that the sanitary sewers in the area are pumped from the Lincoln Heights Sanitary Pumping Station, which is currently at capacity, the site approval will be conditioned on the completion of the pumping station upgrades before it can take on the additional sanitary flows from the site.

4.3 Proposed Sanitary Servicing

A 300 mm diameter sanitary building service, complete with full port backwater valve as per City standard S14.1, is recommended to service the proposed development. The sanitary lateral is be equipped with a sanitary monitor manhole, anchored as per S.P. No. F-4070, before connecting to the sewer main with a riser pipe as per City standard S11.1. The proposed sanitary servicing is shown on **Drawing SSP-1** and **Drawing SA-1** in **Appendix G**.

Existing connections are to be abandoned and full port backwater valves installed on the proposed sanitary service within the site to prevent any surcharge from the downstream sewer main from impacting the proposed property. A sump pump will be required for sewage discharge from the mechanical room. Sizing of the service lateral, sump pit, and sump pump are to be confirmed by the mechanical consultant.

5.0 Stormwater Management and Servicing

5.1 Objectives

The goal of this stormwater servicing and stormwater management (SWM) plan is to determine the measures necessary to control the quantity and quality of stormwater released from the proposed development to meet the criteria established during the consultation process with City of Ottawa staff, and to provide sufficient details required for approval.

5.2 Stormwater Management (SWM) Criteria

The Stormwater Management (SWM) criteria were established by combining current design practices outlined by the City of Ottawa Sewer Design Guidelines (SDG) (October 2012), review of project pre-consultation notes with the City of Ottawa, and through consultation with City of Ottawa staff and SWM Guidelines for the Pinecrest Creek/Westboro Area (PWSG). The following summarizes the criteria, with the source of each criterion indicated in brackets:

General

- Use of the dual drainage principle (City of Ottawa SDG)
- Wherever feasible and practical, site-level measures should be used to reduce and control the volume and rate of runoff (City of Ottawa SDG)
- Assess impact of 100-year event outlined in the City of Ottawa Sewer Design Guidelines on the major and minor drainage systems (City of Ottawa SDG)

Storm Sewer & Inlet Controls

- Discharge for each storm event to be restricted to a 2-year storm event pre-development rate with a maximum pre-development C coefficient of 0.5 (PWSG, **Appendix E**, and City of Ottawa pre-consultation, **Appendix F**)
- Peak flows generated from events greater than the 2-year and including the 100-year storm must be detained on site (PWSG, **Appendix E**, and City of Ottawa pre-consultation, **Appendix F**)
- The preferred stormwater system outlet for this site is the 450 mm diameter storm sewer within the Richmond Road ROW. (City of Ottawa pre-consultation, **Appendix F**)
- The foundation drainage system is to be independently connected to sewer main unless being pumped with appropriate back up power, sufficient sized pump, and backflow prevention. (City of Ottawa pre-consultation, **Appendix F**)
- T_c should be not less than 10 minutes since IDF curves become unrealistic at less than 10 min (PWSG, **Appendix E**, and City of Ottawa SDG).

Surface Storage & Overland Flow

- Building openings to be a minimum of 0.30 m above the 100-year water level (City of Ottawa SDG)

- Maximum depth of flow under either static or dynamic conditions shall be less than 0.30 m (City of Ottawa SDG)
- Provide adequate emergency overflow conveyance off-site with a minimum vertical clearance of 15 cm between the spill elevation and the ground elevation at the building envelope in the proximity of the flow route or ponding area (City of Ottawa SDG)

Quality Control

- Site must provide quality control measures that meet 80 % TSS Removal (PWSG, **Appendix E**)

5.3 Existing Conditions

The existing site (0.413 ha) is dominated by roofs and asphalt pavement with only around 131.9 m^2 in soft area, as such the overall site pre-development runoff coefficient was established to be C=0.88, in which the hard surface areas use a coefficient of 0.90 while soft surface areas have a coefficient of 0.20. This exceeds the maximum permissible pre-development runoff coefficient of C=0.5 identified in the City of Ottawa pre-consultation for this site. Therefore, the pre-development runoff coefficient of 0.5 was used for the site analysis.

The pre-development release rates for the site have been determined using the rational method and the drainage characteristics identified above. A time of concentration for the pre-development area (10 minutes) was assigned based on the relatively small area, and its proximity to the stormwater outfall. The peak pre-development flow rates shown in **Table 5-1** have been calculated using the rational method as follows:

$$Q = 2.78\,(C)(I)(A)$$

Where:
$Q = peak\ flow\ rate, L/s$
$C = site\ runoff\ coefficient$
$I = rainfall\ intensity, mm/hr\ (per\ City\ of\ Ottawa\ IDF\ curves)$
$A = drainage\ area, ha$

Table 5-1: Peak Pre-Development Flow Rates

Design Storm	Pre-Development Flow Rate (L/s) for C=0.5, A=0.37 ha, t_c = 10 min
2-year	**39.8**
100-year	**92.5**

5.4 Stormwater Management Design

The site will be serviced by the 450 mm diameter storm sewers in Richmond Road. The site has been subdivided into catchment areas to effectively collect, store, and convey runoff at flowrates not exceeding the target release rate established by consultation with the City of Ottawa (refer to **Drawing SD-1** in **Appendix G** for drainage areas).

A stormwater cistern located in the underground parking area is proposed to attenuate peak flows from the rooftop areas from the towers and the common areas. Area drains will convey stormwater runoff from the surface to the stormwater cistern via the internal plumbing of the buildings. The stormwater cistern will be pumped at controlled rates to monitor manholes which outlets to the 450 mm diameter storm sewer on Richmond Road via 300 mm diameter lateral. The stormwater cistern's location will be coordinated by building's architect in conjunction with mechanical and structural engineers.

Footing drainage will be independent of the internal stormwater cistern quantity control system while sharing the same outlet. The mechanical design for the weeping tile system will include dedicated storm pits and duplex pumps to pump the weeping tile drainage to the storm main downstream of the cistern.

The proposed site plan, drainage areas and proposed storm sewer infrastructure are shown on **Drawing SD-1** and **SSP-1** in **Appendix G**.

5.4.1 ALLOWABLE RELEASE RATE

Based on consultation with City of Ottawa staff, the peak post-development discharge from the subject site must be limited to the discharge resulting from the 2-year storm event due to capacity restrictions of the downstream municipal stormwater infrastructure. As per **Section 5.3**, the maximum pre-development runoff coefficient of C=0.5 was utilized for the site. C coefficient values have been increased by 25 % for the post-development 100-year storm event based on the MTO Drainage Manual recommendations.

The pre-development 2-year release rate for the site, as determined in **Table 5-1**, was calculated using the rational method to be 39.8 L/s. Consequently, the target release rate for the site under all events up to and including the 100-year event will be 39.8 L/s, as shown in **Table 5-2** below.

Table 5-2: Target Release Rate

Design Storm	**Target Flow Rate (L/s)**
All Events	39.8

5.4.2 QUANTITY CONTROL: STORAGE REQUIREMENTS

The Modified Rational Method (MRM) was used to assess the flow rate and volume of runoff generated under post-development conditions. The site was subdivided into sub-catchments tributary to separate quantity control measures and subject to different inlet controls. **Drawing SD-1** shows the delineated sub-

catchment areas. The MRM spreadsheet is included in **Appendix D.1** and the Storm Sewer Design Sheet is included in **Appendix D.2**.

The following assumptions were made in the creation of the storm drainage plan and accompanying MRM spreadsheet:

- Excess run-off that cannot be captured as surface storage due to grading constraints is to sheet flow uncontrolled to the adjacent roadways (areas UNC-1, UNC-2, and UNC-3).
- Stormwater cistern equipped with mechanical pump to attenuate peak flows from the cistern will be used to manage stormwater flows from the site.

5.4.2.1 Uncontrolled Areas

Uncontrolled areas represent drainage areas that cannot be graded to enter the storm sewer system and are not captured by the proposed storm cistern. As such, they will sheet drain off the site to the adjacent roadways (see **Drawing SD-1**).

Table 5-3: Peak Uncontrolled 2- and 100-Year run-off

Area IDs	Area (ha)	2-Yr uncontrolled peak flow (L/s)	100-Yr uncontrolled peak flow (L/s)
UNC-1	0.03	5.0	12.9
UNC-2	0.01	2.0	5.2
UNC-3	0.02	3.6	9.2

5.4.2.2 Stormwater Cistern

As part of the stormwater management design of the site development, a stormwater cistern located in the underground parking area and equipped with mechanical pumps is proposed to attenuate peak flows from drainage areas CIST 1-1 to CIST 1-3. The final location of the cistern within the proposed building is to be coordinated by the architect with mechanical and structural engineers.

The cistern is to be designed to provide a minimum active storage volume of 140 m^3 with a maximum controlled release rate of 12.5 L/s. The stormwater cistern is to discharge at the specified controlled release rate using a pump.

Table 5-4 summarizes the respective flow rates and volume of retained stormwater in the 2-year and 100-year storm events.

Table 5-4: Proposed Cistern 2 and 100-Year Storage Requirement

Storm Return Period	Area IDs	Drainage Area (ha)	$Q_{release}$ (L/s)	$V_{required}$ (m^3)	$V_{available}$ (m^3)
2-year	CIST 1-1 to CIST 1-3	0.32	12.5	35.0	140.00
100-year				133.0	

5.4.2.3 Results

The proposed stormwater management plan provides adequate attenuation to meet the target release rate for the 2 and 100-year storm events as shown in **Table 5-5** below.

Table 5-5: Estimated Post-Development Discharge

Area Type	2-Year (L/s)	100-Year (L/s)	Target (L/s)
Uncontrolled	10.6	27.3	39.8
Controlled Areas	12.5		
Total Flow to Sewer	**39.8**		

5.4.3 QUALITY CONTROL

As per the Pinecrest Creek/Westboro Stormwater Design Guidelines, the site will be required to provide Enhanced level of quality control (80 % TSS Removal). Correspondence with the RVCA regarding the quality control requirements for the site can be found in **Appendix D.3**, however, further correspondence in addition to the pre-consultation notes provided will be required to determine if any further quality control measures should be implemented. To meet the quality control requirement, storm runoff from the surface areas of the development will be captured into the site storm sewer system and directed to the proposed oil/grit separator unit. It should be noted that the SWM design criteria for the Pinecrest Creek/Westboro Study Area states the requirement to infiltrate the first 10 mm of rainfall, which will not be feasible based on the proposed buildings and underground parking layout. Best management practices are to be employed and the quality control provided by the OGS unit is a substantial improvement from existing conditions. Quality control design criteria excerpts have been provided in **Appendix D.3**.

The Stormceptor sizing software has been used to size the required unit to provide up to 90% TSS removal as shown in the Stormceptor sizing design sheet and standard detail included in **Appendix D.4.**

A Stormceptor EF 04 or an approved equivalent designed to provide up to > 80% TSS removal has been proposed to collect and treat storm runoff from the site before discharging into the existing 450 mm diameter storm sewer in Richmond Road as shown on **Drawing SSP-1** in **Appendix G.**

5.5 Proposed Stormwater Servicing

A single 300 mm diameter stormwater building service, complete with full port backwater valve as per City standard S14.1, is proposed for the storm service discharge, as per **Drawing SSP-1** and **Drawing SD-1** in **Appendix G**. A stormwater sump and pump are required for the proposed foundation drain, and the roof drains are to be connected to the cistern.

The combined foundation drain, roof drain, and subdrain flows will outlet to the cistern, which then pumps the discharge at a controlled rate and to the existing 450 mm diameter storm sewer within the Richmond Road ROW. The lateral is to connect to the main as per City standard S11.1 via monitoring manholes. The proposed stormwater servicing is shown on **Drawing SSP-1** and **SD-1** in **Appendix G**.

6.0 Site Grading

The proposed re-development site, excluding the park, measures approximately 0.41 ha in area and consists of an existing commercial strip mall and asphalt parking area, with very small patches of grassland. The topography across the site generally slopes from the middle, close to the east, towards the Assaly Road ROW at the west and the Starflower Lane ROW at the east.

A detailed grading plan (see **Drawing GP-1** in **Appendix G**) has been provided to satisfy the stormwater management requirements, as detailed in **Section 5.0**, adhere to any grade raise restrictions for the site, and provide for minimum cover requirements for storm and sanitary sewers where possible. Site grading has been established to provide emergency overland flow routes required for stormwater management.

The proposed grading plan provides an adequate overland flow route and maintains the existing drainage conditions for the perimeter locations adjacent to the public right of way. No retaining walls are required for the proposed development. As identified on the grading plan, various depressed curbs and associated sidewalks will be removed and replaced with full height barrier curbs and sidewalks in accordance with Ottawa standards.

7.0 Utilities

Overhead (OH) hydro-wires run east-west on the north side of Starflower Lane and the east side of Assaly Road, terminating at a utility pole located at the northwest corner of the site. An existing underground hydro duct bank runs east-west within Richmond Road along the south property line of the site. The existing utility poles and duct banks within the public right of way are to be protected during construction.

As the site is surrounded by existing residential and commercial development, Hydro Ottawa, Bell, Rogers, and Enbridge servicing is readily available through existing infrastructure to service this site. The exact size, location, and routing of utilities will be finalized after design circulation. Existing overhead wires and utility plants may need to be temporarily moved/reconfigured to allow sufficient clearance for the movement of heavy machinery required for construction. The relocation of existing utilities will be coordinated with the individual utility providers upon design circulation.

8.0 Approvals

The proposed development lies on a private site under singular ownership; therefore, the site will not require an Environmental Compliance Approval (ECA) from the Ministry of the Environment, Conservation and Parks (MECP) under O.Reg. 525/98.

For ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). It is possible that groundwater may be encountered during the foundation excavation on this site. A minimum of two to four weeks should be allotted for completion of the EASR registration and the preparation of the Water Taking and Discharge Plan by a Qualified Person as stipulated under O.Reg. 63/16. An MECP Permit to Take Water (PTTW), which is required for dewatering volumes exceeding 400,000L/day, is not anticipated for the site.

9.0 Erosion and Sediment Control During Construction

To protect downstream water quality and prevent sediment build-up in catch basins and storm sewers, erosion and sediment control measures must be implemented during construction. The following recommendations will be included in the contract documents and communicated to the Contractor.

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit the extent of the exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with geotextiles, geogrid, or synthetic mulches.
6. Install silt barriers/fencing around the perimeter of the site as indicated in **Drawing ECDS-1** in **Appendix G** to prevent the migration of sediment offsite.
7. Install trackout control mats (mud mats) at the entrance/egress to prevent migration of sediment into the public ROW.
8. Provide sediment traps and basins during dewatering works.
9. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
10. Schedule the construction works at times which avoid flooding due to seasonal rains.

The Contractor will also be required to complete inspections and guarantee the proper performance of their erosion and sediment control measures at least after every rainfall. The inspections are to include:

- Verification that water is not flowing under silt barriers.
- Cleaning and changing the sediment traps placed on catch basins.

Refer to **Drawing ECDS-1** in **Appendix G** for the proposed location of silt fences, sediment traps, and other erosion control measures.

10.0 Geotechnical Investigation

A geotechnical investigation report was prepared by Paterson Group on April 25, 2023 to provide an assessment of the subsurface conditions found at the site. Five (5) boreholes were advanced on site, in which two were advanced to a maximum depth of 13.2 metres below the existing ground surface (BGS), and the remainder to a maximum depth of 16.5 metres BGS in the investigation carried out from March 13 to March 15, 2023. The information obtained from the field investigation will guide the detailed design of the site and identify development constraints.

The subsurface profile encountered at the test holes BH 1-23 to BH 4-23 are characterized primarily by a layer of concrete or asphaltic concrete underlain by crushed stone with silty sand fill extending to an approximate depth of 0.4 m to 1 m below the existing ground surface, with brown silty clay with some sand and gravel observed underlying the crushed stone at BH 2-23 and BH 4-23 up to 1.6 m BGS. The profile at BH 5-23 is observed to comprise of 100 mm of topsoil, underlain by brown sandy silt with some clay, traces of gravel and organics.

Underlying the fill, loose brown sandy silt with trace to some clay was observed, with stiff to firm brown silty clay layer encountered under the fill and silty. Underlaying the silty clay deposit was a grey glacial till comprising of sand in gravel in a silty clay soil matrix and was noted to be water bearing and highly permeable on site.

Based on the core samples acquired in the boreholes, the bedrock generally consists of excellent grey quartz sandstone of the Ottawa formation at depths ranging from 12 m to 14 m. Groundwater levels are expected to be 4.5 metres to 6.0 metres below the existing ground surface, though as groundwater levels are subject to seasonal fluctuations, they could vary at the time of construction.

Based on Paterson's recommendations, the site is suitable for the proposed development. It is expected that the building be founded on the bedrock. Alternatively, a near vertical, zero entry trench be excavated to the surface of the bedrock will be filled with lean concrete for the footings to avoid excavating the entire building footprint to the bedrock level. A total of five foundation options are presented.

The recommended rigid pavement structure is further presented in **Table 10-1** below.

Table 10-1: Recommended Pavement Structure

Material	Light Vehicle Parking	Access Lanes, Local Roadways and Heavy Vehicle Parking
Wear Course – Superpave 12.5-FC2 Asphaltic Concrete	40 mm	
Binder Course –Superpave 19.0 Asphaltic Concrete	50 mm	
BASE – OPSS Granular A Crushed Stone	150 mm	
SUBBASE – OPSS Granular B Type II	300 mm	400 mm

Refer to the full geotechnical report attached in **Appendix E** for further details.

11.0 Conclusions

11.1 Water Servicing

Based on the supplied boundary conditions for existing watermains and calculated domestic and fire flow demands for the subject site, the adjacent watermain on Richmond Road has sufficient capacity to sustain both the required domestic and emergency fire flow demands for the development. The proposed development will be serviced by two 150 mm diameter water service laterals to connect to the 305 mm diameter watermain on Richmond Road. Booster pumps will be required to provide adequate pressure to the towers' upper stories. Sizing of the water service and requirements for booster pump(s) are to be confirmed by the mechanical consultant.

11.2 Sanitary Servicing

The proposed sanitary sewer service will consist of a 300 mm diameter sanitary service lateral, a sanitary sump pit, a monitor manhole, and sump pump directing wastewater to the existing 250 mm diameter sanitary sewer on Richmond Road. Existing connections are to be abandoned and full port backwater valves installed on the proposed sanitary service within the site to prevent any surcharge from the downstream sewer main from impacting the proposed property. A sump pump will be required for sewage discharge from the mechanical room. Sizing of the service lateral, sump pit, and sump pump are to be confirmed by the mechanical consultant.

11.3 Stormwater Servicing and Management

A cistern in the underground parking has been proposed to limit the stormwater discharge rate for all rainfall events up to and including the 100-year event to a peak 2-year predevelopment release rate. The remaining site area drains uncontrolled to the adjacent surrounding ROWs as per existing conditions.

A 300 mm diameter storm service lateral is proposed for the building's foundation drain and internal storm sewer system, which is to be mechanically pumped and include a full port backwater valve. The roof drains and ramp drain are to be connected through internal plumbing to the cistern, which will pump discharge at a controlled rate through the service lateral and the backwater valve to the 450 mm diameter municipal storm sewer in the Richmond Road ROW. Sizing of the service lateral, cistern, and foundation drain pump are to be confirmed by the mechanical consultant. A Stormceptor EF 04 or an approved equivalent designed to provide up to > 80% TSS removal has been proposed to collect and treat storm runoff from the site before discharging into the existing 450 mm diameter storm sewer in Richmond Road.

11.4 Grading

Site grading has been designed to provide an adequate emergency overland flow route and respect the existing grades at the property lines. The northeast, east, west, and south sides will continue to drain as per existing conditions.

11.5 Erosion and Sediment Control During Construction

Erosion and sediment control measures and best management practices outlined in this report and included in the drawing set, will be implemented during construction to reduce the impact on adjacent properties, the public ROW, and existing facilities.

11.6 Geotechnical Investigation

Based on the geotechnical investigation, the site is considered suitable for the proposed building. It is expected that the building be founded on the bedrock. Alternatively, a near vertical, zero entry trench be excavated to the surface of the bedrock will be filled with lean concrete for the footings to avoid excavating the entire building footprint to the bedrock level. The report did not identify any grade-rise restrictions for the site.

11.7 Utilities

The site is situated within an established neighbourhood, hence existing utility infrastructure is readily available to service the proposed development. Overhead lines within the Starflower Lane and Assaly Road ROWs will need to be protected and accommodated during construction. Utility infrastructure exists within overhead lines running parallel to the north and west sides of the property, and underground hydro duct bank within the Richmond Road ROW. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities will be finalized after design circulation.

11.8 Approvals

This site will not be subjected to the Ministry of the Environment, Conservation and Parks (MECP) Environmental Compliance Approval (ECA) process under O.Reg. 525/98. For the expected dewatering needs of 50,000 to 400,000 L/day, the proponent will need to register on the MECP's Environmental Activity and Sector Registry (EASR). A Permit to Take Water, for dewatering needs in excess of 400,000 L/day, is not anticipated for this site.

APPENDICES

Appendix A Water Demands

A.1 Domestic Water Demands

1299 Richmond Road - Domestic Water Demand Estimates
Site Plan provided by Quadrangle Architects Ltd.
Project Number: 160401697

Population densities as per Table 4.1 of the City of Ottawa Water Design Guidelines:		
1 Bedroom	1.4	ppu
2 Bedroom	2.1	ppu
3 Bedroom	3.1	ppu

Demand conversion factors as per Table 4.2 of the Ottawa Design Guidelines - Water Distribution[5]:		
Residential	280	L/cap/day
Commercial	28000	L/gross ha/day

Building ID	Area (ha)	Bachelor	1 Bedroom	1 Bedroom + Den[4]	2 Bedroom	2 Bedroom + Den[4]	3 Bedroom	Population	Avg. Day Demand		Max. Day Demand [1, 2]		Peak Hour Demand [1, 2]	
									(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Podium Retail	0.0735	-	-	-	-	-	-	-	1.4	0.02	2.1	0.04	3.9	0.06
Commercial Total	**0.0735**								**1.4**	**0.02**	**2.1**	**0.04**	**3.9**	**0.06**
A Residential	-		162			108		562	109.2	1.82	273.0	4.55	600.6	10.01
B Residential	-		138			92		478	93.0	1.55	232.6	3.88	511.6	8.53
Podium Residential	-	1	13	2	39	14	21	214	41.7	0.69	104.1	1.74	229.1	3.82
Residential Total		**1**	**313**	**2**	**39**	**214**	**21**	**1254**	**243.9**	**4.06**	**609.7**	**10.16**	**1341.3**	**22.35**
Total Site :	**0.073**	**1**	**313**	**2**	**39**	**214**	**21**	**1254**	**245.3**	**4.09**	**611.8**	**10.20**	**1345.2**	**22.42**

Notes:
1 Water demand criteria used to estimate peak demand rates for residential areas are as follows:
maximum day demand rate = 2.5 x average day demand rate
peak hour demand rate = 2.2 x maximum day demand rate (as per Technical Bulletin ISD-2010-02)
2 Water demand criteria used to estimate peak demand rates for commercial/amenity areas are as follows:
maximum day demand rate = 1.5 x average day demand rate
peak hour demand rate = 1.8 x maximum day demand rate (as per Technical Bulletin ISD-2010-02)
3 Number of apartment units as per **Quadrangle Architects Ltd. GFA Suite** statistics table.
4 Assumption that "1 bedroom with den" has density of 2.1 ppu, "2-bedroom with den" has density of 3.1 ppu
5 As per Table 4-2 from the City of Ottawa Water Design Guidelines and Technical Bulletin ISTB-2021-03, the average daily rate of water demand for residential areas: 280 L/cap/day

A.2 Fire Flow Demands (FUS 2020)

FUS Fire Flow Calculation Sheet - 2020 FUS Guidelines

Stantec Project #: 160401697
Project Name: 1299 Richmond Road
Date: 2023-05-01
Fire Flow Calculation #: 1
Description: 5-Storey Podium with 28-Storey and 32-Storey High-Rise Towers
Podium Footprint: 2581.5 m^2. Tower Footprint: 1500 m^2.

Notes: Podium footprint area per sketch on AutoCAD from April 28, 2023 site plan; Tower footprints as per Quadrangle Site Plan provided April 28, 2023

Step	Task	Notes										Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type II - Noncombustible Construction / Type IV-A - Mass Timber Construction										0.8	-
2	Determine Effective Floor Area	Sum of Largest Floor + 25% of Two Additional Floors					Vertical Openings Protected?					YES	-
		2581.5	2405	2405								3784	-
3	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min										-	11000
4	Determine Occupancy Charge	Limited Combustible										-15%	9350
5	Determine Sprinkler Reduction	Conforms to NFPA 13										-30%	-4675
		Standard Water Supply										-10%	
		Fully Supervised										-10%	
		% Coverage of Sprinkler System										100%	
6	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	Firewall / Sprinklered ?				-	-
		North	20.1 to 30	81.79	3	> 100	Type V	NO				10%	935
		East	> 30	0	0	0-20	Type V	NO				0%	
		South	> 30	0	0	0-20	Type V	NO				0%	
		West	> 30	0	0	0-20	Type V	NO				0%	
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min											6000
		Total Required Fire Flow in L/s											100.0
		Required Duration of Fire Flow (hrs)											2.00
		Required Volume of Fire Flow (m^3)											720

A.3 Hydraulic Boundary Conditions (City of Ottawa)

Wu, Michael

From: Rathnasooriya, Shika <Thakshika.Rathnasooriya@ottawa.ca>
Sent: Thursday, 18 May, 2023 12:56
To: Wu, Michael
Cc: Kilborn, Kris; Ford, Matthew
Subject: RE: 1299 Richmond Road Follow-up
Attachments: 1299 Richmond Road May 2023.pdf

Hi Michael,

Well I received a response much quicker than I expected!

Please find BC below:

******The following information may be passed on to the consultant, but do NOT forward this e-mail directly.******

The following are boundary conditions, HGL, for hydraulic analysis at 1299 Richmond Road, (zone 1W) assumed to be a dual connection to the 305 mm watermain on Richmond Road (see attached PDF for location).

Min HGL: 108.3 m

Max HGL: 115.9 m

Max Day + FF (100 L/s): 109.4 m

Regards,
Shika

From: Rathnasooriya, Shika
Sent: May 18, 2023 12:47 PM
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>; Ford, Matthew <Matthew.Ford@stantec.com>
Subject: RE: 1299 Richmond Road Follow-up

Hi Michael,

The typical turn around time for boundary conditions is 3 weeks and given that they were submitted on May 4th you should receive them by May 25th. I'll send a message over to our water resources team requesting a statues update.

If you do not receive an answer before then, please go ahead with the first submission and mention in the report that boundary conditions and sanitary sewer capacity constraints will be included in the second submission.

Regards,
Shika Rathnasooriya, P.Eng
Project Manager
Planning, Real Estate and Economic Development Department - West Branch
City of Ottawa
110 Laurier Avenue West Ottawa, ON
613.580.2424 ext. 23433

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: May 18, 2023 12:10 PM
To: Rathnasooriya, Shika <Thakshika.Rathnasooriya@ottawa.ca>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>; Ford, Matthew <Matthew.Ford@stantec.com>
Subject: 1299 Richmond Road Follow-up

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Good afternoon, Shika:

Just wanted to do a quick follow up on the status of the hydraulic boundary conditions and confirmation of sanitary sewer capacity request for 1299 Richmond Road.

We are looking to have the site servicing and stormwater management report submitted by next Friday (May 26th), as such, in the event the boundary conditions and sanitary sewer capacity confirmation cannot be received prior to that day, I was wondering if it will be possible for us to make the first submission without the boundary conditions and then have them added in as part of the comments following the first submission. There are a few sites in which we were allowed to make the first submission without the boundary conditions.

On a side note, if you received the boundary conditions and sanitary sewer capacity confirmation before next Friday, please be sure to forward them to Matt Ford, who I have cc'd in this email, as I will be away on vacation for the next two weeks.

Thanks,

Michael Wu, EIT
Civil Engineering Intern, Community Development

Work: (613) 738-6033
Mobile: (613) 858-0548
michael.wu@stantec.com

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

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Vacation Alert: I will be away on vacation from May 19th to June 2nd

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A.4 Correspondence With Architect on Construction Type and Vertical Opening Protections

Mott, Peter

From: Kilborn, Kris
Sent: Tuesday, May 30, 2023 12:45 PM
To: Mott, Peter
Subject: FW: 1299 Richmond Road Building Construction and Fire Protection Confirmation

See below for the Building construction questions
Kris

From: Bhagyashri Sakhare <BSakhare@bdpquadrangle.com>
Sent: Tuesday, May 2, 2023 5:45 PM
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>; Sharp, Mike <Mike.Sharp@stantec.com>; Ford, Matthew <Matthew.Ford@stantec.com>; Ryan Lupien <rlupien@bdpquadrangle.com>
Subject: RE: 1299 Richmond Road Building Construction and Fire Protection Confirmation

Hi Michael,
Here are the answers to your questions.

1. It's a non- combustible construction with non-combustible cladding and we will be using cast in place concrete structure.
2. Yes, we will be following the two codes. Air shafts will be enclosed in two-hour fire rated assembly and fire dampers will be installed at the air vents as specified my mechanical consultants. And the building will be required to be sprinklered.
3. We would like to locate it near main entrance lobby on Richmond road side. Can you confirm the fire hydrant location?

Let me know if you need anything else.

Thanks

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: Tuesday, May 2, 2023 9:16 AM
To: Bhagyashri Sakhare <BSakhare@bdpquadrangle.com>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>; Sharp, Mike <Mike.Sharp@stantec.com>; Ford, Matthew <Matthew.Ford@stantec.com>
Subject: 1299 Richmond Road Building Construction and Fire Protection Confirmation

You don't often get email from michael.wu@stantec.com. Learn why this is important

Good morning, Bhagyashri:

I was wondering if you could provide us the following information for the proposed building at 1299 Richmond Road? We would need them for requesting the hydraulic boundary conditions from the City.

1. Construction type.
2. Confirmation that the vertical openings (between floors) are going to be **protected** per the fire code requirements outlined in the Ontario and National Building Codes and whether the building will be sprinklered.
3. Locations of the fire department connections.

Thanks,

Michael Wu, EIT
Civil Engineering Intern, Community Development

Work: (613) 738-6033
Mobile: (613) 858-0548
michael.wu@stantec.com

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

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A.5 Hydraulic Analysis

Stantec	Project:	**1299 Richmond Road**	No. **160401697**
	SITE PLAN HYDRAULIC ANALYSIS		
	Revision:	01	Prepared By: MW
	Revision Date:	18-May-2023	Checked By:

BOUNDARY CONDITIONS (BC)	
Connection at Richmond Road	
Site Plan Revision Date	28-Apr-2023
Min. HGL (m)	108.3
Max. HGL (m)	115.9
Max. Day + Fire Flow (100 L/s)	109.4

Ground Floor Elevation (GFE) (Level 01) (m)	70.15

GROUND FLOOR (GF) PRESSURE RANGE				
	GF HGL (m)	**GF Pressure (kPa)**	**GF Pressure (psi)**	**Outcome**
	= BC HGL (m) - FFE (m)	= GF HGL (m) x 9.804 (kPa/m)	= GF Pressure (kPA) x 0.145 (psi/kPa)	If min <50 psi: booster pump If max >100 psi: pressure reducer
Minimum Normal	38.15	374.0	54.2	No Booster Pump Required
Maximum Normal	45.75	448.5	65.0	No Pressure Reducer Required

Number of Floors Above Ground	32
Approximate Height of One Storey (m)	3
Pressure Drop Per Floor (kPa)	29.4
Pressure Drop Per Floor (psi)	4.3

RESIDUAL PRESSURE RANGE IN MULTI-LEVEL BUILDINGS			
	Residual Pressure (kPa)	**Residual Pressure (psi)**	**Outcome**
Top Floor Min	-537.8	-78.0	Booster Pump Required
Top Floor Max	-463.2	-67.2	
Maximum Number of Floors Above Ground at Minimum Pressure	3		

RESIDUAL PRESSURE UNDER FIRE FLOW CONDITIONS			
	Residual HGL (m)	**Residual Pressure (kPa)**	**Residual Pressure (psi)**
Ground Floor	39.25	384.8	55.8
Top Floor	-53.75	-527.0	-76.4

PRESSURE CHECK		
	Pressure (kPa)	**Pressure (psi)**
UNDER NORMAL OPERATING CONDITIONS		
Pressure Below Minimum	<276	<40
Pressure Below Normal	276-345	40-50
Pressure Within Normal Range	345-552	50-80
Pressure Above Normal Range	552-690	80-100
Pressure Above Maximum	>690	>100
UNDER FIRE FLOW CONDITIONS		
Pressure Below Minimum	<140	<20
Acceptable Pressure	≥140	≥20

A.6 Fire Hydrant Coverage Calculations

Stantec	Project:	**1299 Richmond Road**	160401697
	TABLE 1: FIRE HYDRANT COVERAGE TABLE		
	Revision:	1	Prepared By: MW
	Revision Date:	5/18/2023	Checked By:

Description	Hydrants[1]			Total Available Fire Flow (L/min)	Total Required Fire Flow[2] (L/min)
	HYD-01	**HYD-02**	**HYD-03**		
265 Catherine Street					
Distance from fire department connection (m)	76.4	14.4	41.2	-	-
Maximum fire flow capacity[3] (L/min)	3,785	5,678	5,678	**15,141**	6,000

NFPA 1 Table 18.5.4.3	
Distance to Building (m)	Maximum Capacity (L/min)
≤ 76	**5,678**
> 76 and ≤ 152	**3,785**
> 152 and ≤ 305	**2,839**

Notes:

1. Hydrant locations as per GeoOttawa accessed May 18, 2023. Refer to fire hydrant coverage sketch (Figure 3-1).
2. See OBC Calculations, Appendix A.2 for fire flow requirements.
3. See NFPA 1 Table 18.5.4.3 in Appendix I of the City of Ottawa Technical Bulletin ISTB-2018-02 for maxiumim fire flow capacity of hydrants by distance to building.

Appendix B Site Plan by Quadrangle Architects Ltd. (April 28, 2023)

ELEVATION NOTES
1. Elevations shown are geodetic, derived from the Vertical Control Monument 2016-0135 having an elevation of 68.405 metres and are referred to the CGVD28 geodetic datum.
2. It is the responsibility of the user of this information to verify that the job benchmark has not been altered or disturbed and that it's relative elevation and description agrees with the information shown on this drawing.

UTILITY NOTES
1. This drawing cannot be accepted as acknowledging all of the utilities and it will be the responsibility of the user to contact the respective utility authorities for confirmation.
2. Only visible surface utilities were located.
3. A field location of underground plant by the pertinent utility authority is mandatory before any work involving breaking ground, probing, excavating etc.

LOT 27 LOT 28 LOT 29 PART 6 PLAN 4R-1031

STARFLOWER LANE (Formerly Regina Lane) (By-Law 2020-250, Inst. oc2251257)

ruelle STARFLOWER LANE pin 04282 - 0662

PROPERTY LINE

3m SETBACK LINE

N39°21'50"E

PARKING GARAGE AIR SHAFT WITH GRATE POROSITY OF LESS THAN 20mm X 20mm (or 40mm X 10mm)

PARKING GARAGE ENTRANCE (RAMP TO UNDERGROUND)

4 STOREY TERRACE

BALCONY

AMENITY PATIO

LANDSCAPED TERRACE 1 STOREY

GARBAGE STAGING AREA

MAIL DROP OFF AREA

ELEVATOR MACHINE ROOM

CACF

TOWER A
32 STOREY RESIDENTIAL BUILDING
FFL: 70.35

5 STOREY PODIUM (AMENITY)

4 STOREY PODIUM

TOWER B
28 STOREY RESIDENTIAL BUILDING
FFL: 70.35

PARKING GARAGE RAMP

ELEVATOR + MPH ROOM

GAS METER

HYDRO VAULT AIR SHAFT WITH GRATE POROSITY OF LESS THAN 20mm X 20mm (or 40mm X 10mm)

PARKING GARAGE AIR SHAFT WITH GRATE POROSITY OF LESS THAN 20mm X 20mm (or 40mm X 10mm)

OUTLINE OF STORM WATER STORAGE TANK

GARAGE OUTLINE

FIRE- DEPARTMENT CONNECTION (E)

FIRE-HYDRANT (E)

PROPOSED PARK LAND (AREA = 414.2 sq. metres)

PARK LAND (10% OF TOTAL SITE AREA)

3728.1 m² (site) + 414.2 m² (park) = 4142.3 m²

ASSAILY ROAD pin 04282 - 0664

RICHMOND ROAD

ROAD WIDENING

N 44° 35' 50" E (Reference Bearing)

N44°35'50"E 96.54 (P3)&Meas.

PART 1 PLAN 5R-9366 (By-Law 2020-250, Inst. N339171)

N72°55'50"E 31.98 (P3)&Meas.

N42°46'10"W 1.10(P3)&Set

JOB BENCHMARK TOP OF SPINDLE Elev.=71.02

Approximate Crown of Road

pin 04282 - 0679

FIRE ROUTE

UP with Transformer/ LS

Southwest Face of Curb 0.1 Northeast

Southeast Face of Curb 0.1 Southeast

Southwest Face of Curb 0.4 Northeast

Southeast Face of Curb 0.5 Southeast

Concrete Curb

Concrete Sidewalk

Depressed Sidewalk

Asphalt Parking

Asphalt

Grass

Shrub

Planter

Concrete Pad

Interlock 0.05 Northeast

Interlock 0.4 Northeast

Interlock 0.05 Southeast

Edge of Asphalt

Paint Line

Guy Wire

MH-ST T/G=68.90

MH-H T/G=69.40

MH-T T/G=70.30

Sign 0.4 Southeast Concrete Pad 0.6 Southeast

CB T/G=68.98

CB T/G=69.72

CB T/G=69.51

VC T/G=70.40

VC T/G=70.29

VC T/G=6

WV

SIB(1236)

SIB(RMOO)(WIT) 0.05 North (P3)&Set

SSIB(RMOO)(WIT) 0.05 South 0.03 West

SIB(CC(AOG))

CC(AOG) 69.05

CC(AOG) 68.94

MAIN PRIVATE RESIDENTIAL ENTRANCE

RESIDENTIAL ENTRANCE

MAIN RESIDENTIAL ENTRANCE

COMMERCIAL ENTRANCE

EXIT

WIDTH OF DRIVEWAY

DISTANCE FROM ENTRANCE TO CURB

DISTANCE FROM FIRE HYDRANT ENTRANCE TO CURB

DISTANCE FROM MAIN ENTRANCE TO CURB

SLOPED WALKWAY

CANOPY

Top of foundation wall

Anchor to basement

Door Sill Elev.=69.46

Door Sill Elev.=69.44

Brick 0.07 Southeast

Brick 0.07 Southwest

Door Sill Elev.=69.46

Transformer Pad

Northwest Face WRW ± to Line

Edge of Asphalt ±to Line

BRW ± to Line

C/L Bollard 0.2 South

WRW 0.22 Southeast

Northeast Face WRW 0.04 Southwest

Northeast Face WRW 0.05 Southwest

Northeast Face CRW 0.07 Southwest

Southeast Face CRW 0.03 Southwest

Edge of Asphalt

Edge of Grass

Edge of Concrete

Edge of Planter

Edge of Pad

Edge of Curb

Edge of Sidewalk

Edge of Paint Line

Edge of Guy Wire

Edge of Sign

Edge of Shrub

Edge of Interlock

Edge of Transformer Pad

Edge of Door Sill

Edge of Brick

Edge of Northwest Face WRW

Edge of Northeast Face WRW

Edge of Northeast Face CRW

Edge of Southeast Face CRW

Edge of Southwest Face CRW

Edge of Southwest Face WRW

Edge of Southeast Face WRW

Edge of Southeast Face of Curb

Edge of Southwest Face of Curb

Edge of Northeast Face of Curb

Edge of Northwest Face of Curb

Appendix C Sanitary

C.1 Sanitary Calculation Sheet

Stantec

SUBDIVISION:

1299 Richmond Road

DATE: 5/3/2023
REVISION: 1
DESIGNED BY: MW
CHECKED BY: PM

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401697

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 l/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 l/ha/day	MANNINGS n	0.013
PEAKING FACTOR (ICI >20%):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / 1 BEDROOM	1.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 2 BEDROOM	2.1	INFILTRATION	0.33 l/s/Ha	HARMON CORRECTION FACTOR	0.8
PERSONS / 3 BEDROOM	3.1				

LOCATION			RESIDENTIAL AREA AND POPULATION								COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA	UNITS 1 BEDROOM	2 BEDROOM	3 BEDROOM	POP.	CUMULATIVE AREA	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW	AREA	ACCU. AREA	AREA	ACCU. AREA	AREA	ACCU. AREA	AREA	ACCU. AREA	AREA	ACCU. AREA	PEAK FLOW	TOTAL AREA	ACCU. AREA	INFILT. FLOW	FLOW	LENGTH	DIA	MATERIAL	CLASS	SLOPE	CAP. (FULL)	CAP. V PEAK FLOW	VEL. (FULL)	VEL. (ACT.)
			(ha)					(ha)			(l/s)	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)	(ha)	(l/s)	(ha)	(ha)	(l/s)	(l/s)	(m)	(mm)			(%)	(l/s)	(%)	(m/s)	(m/s)
BUILDING	Building	MH 25331	0.258	314	41	235	1254	0.258	1254	3.73	15.18	0.074	0.074	0.000	0.000	0.000	0.000	0.000	0.000	0.083	0.083	0.04	0.414	0.414	0.14	15.35	19.8	300	PVC	SDR 35	1.00	96.0	**15.99%**	1.36	0.82

C.2 Correspondence with City on Sanitary Sewer Capacity

Mott, Peter

From: Kilborn, Kris
Sent: Tuesday, May 30, 2023 11:57 AM
To: Mott, Peter
Subject: FW: 1299 Richmond Road Sanitary Sewer Capacity Confirmation and SWM Quality Control Measures

---

From: Rathnasooriya, Shika <Thakshika.Rathnasooriya@ottawa.ca>
Sent: Tuesday, May 23, 2023 11:56 AM
To: Kilborn, Kris <kris.kilborn@stantec.com>; Ford, Matthew <Matthew.Ford@stantec.com>
Cc: Wu, Michael <Michael.Wu@stantec.com>
Subject: FW: 1299 Richmond Road Sanitary Sewer Capacity Confirmation and SWM Quality Control Measures

Hi Kris,

Please see below.

Thanks,
Shika

From: Tousignant, Eric <Eric.Tousignant@ottawa.ca>
Sent: May 23, 2023 11:43 AM
To: Rathnasooriya, Shika <Thakshika.Rathnasooriya@ottawa.ca>
Subject: RE: 1299 Richmond Road Sanitary Sewer Capacity Confirmation and SWM Quality Control Measures

Hi Shika

No concerns with that flow. The pipe is also steep there and it's the end of the run, so there is quite a bit of spare capacity.

Eric

Eric Tousignant, P.Eng.
Senior Water Resources Engineer/ Ingénieur principal en resources hydriques
Infrastructure and Water Services / *services d'infrastructure et d'eau*
City of Ottawa
613-580-2424 ext 25129

From: Rathnasooriya, Shika <Thakshika.Rathnasooriya@ottawa.ca>
Sent: May 16, 2023 8:50 AM
To: Tousignant, Eric <Eric.Tousignant@ottawa.ca>
Subject: FW: 1299 Richmond Road Sanitary Sewer Capacity Confirmation and SWM Quality Control Measures

Hi Eric,

Can you please confirm if the capacity of the 300mm diameter sanitary sewer within Richmond Road can accommodate an additional 15.4L/s?

Thank you,
Shika

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: May 15, 2023 4:50 PM
To: Rathnasooriya, Shika <Thakshika.Rathnasooriya@ottawa.ca>
Cc: Ford, Matthew <Matthew.Ford@stantec.com>; Kilborn, Kris <kris.kilborn@stantec.com>
Subject: 1299 Richmond Road Sanitary Sewer Capacity Confirmation and SWM Quality Control Measures

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Good afternoon, Shika:

In addition to the hydraulic boundary conditions, as part of the servicing for the proposed development on 1299 Richmond Road, we would like to confirm if there is sufficient capacity downstream of the 300 mm diameter sanitary sewer in Richmond Road to receive an additional peak flow of 15.4 L/s from the proposed development.

Please find our sanitary design sheet and location map attached for your information. Furthermore, it is my understanding that the City now provides the SWM quality control criteria. Given that the site drains directly to the Ottawa River as indicated in the 2020 Pinecrest Creek/Westboro SWM Design Criteria report, are there any additional quality control measures applicable to the site, besides the 80 % TSS Removal?

On a side note, I will be away on vacation for two weeks starting this Friday afternoon. If you have any questions or have received the boundary conditions, please don't hesitate to reach out to let Kris and Matt, who I have cc'd in this email.

Thanks,

Michael Wu, EIT
Civil Engineering Intern, Community Development

Work: (613) 738-6033
Mobile: (613) 858-0548
michael.wu@stantec.com

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

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Vacation Alert: I will be away on vacation from May 19th to June 2nd

'

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Appendix D Stormwater Servicing

D.1 Modified Rational Method Sheet

Stormwater Management Calculations

File No: **160401697**
Project: **1299 Richmond Road**
Date: **30-May-23**

SWM Approach:
Post-development to Pre-development flows

Post-Development Site Conditions:

Overall Runoff Coefficient for Site and Sub-Catchment Areas

Runoff Coefficient Table

Catchment Type	Sub-catchment Area ID / Description		Area (ha) "A"		Runoff Coefficient "C"	"A x C"		Overall Runoff Coefficient
Uncontrolled - Non-Tributary	PARK	Hard	0.02		0.9	0.022		
		Soft	0.02		0.2	0.003		
	Subtotal			0.04			0.025	0.61
Uncontrolled - Tributary	CIST 1-3	Hard	0.05		0.9	0.041		
		Soft	0.00		0.2	0.000		
	Subtotal			0.05			0.041	0.90
Uncontrolled - Tributary	CIST 1-2	Hard	0.01		0.9	0.009		
		Soft	0.00		0.2	0.000		
	Subtotal			0.01			0.009	0.80
Controlled - Tributary	CIST 1-1	Hard	0.26		0.9	0.235		
		Soft	0.00		0.2	0.000		
	Subtotal			0.26			0.235	0.90
Uncontrolled - Non-Tributary	UNC-3	Hard	0.02		0.9	0.017		
		Soft	0.00		0.2	0.000		
	Subtotal			0.02			0.017	0.90
Uncontrolled - Non-Tributary	UNC-2	Hard	0.01		0.9	0.009		
		Soft	0.00		0.2	0.000		
	Subtotal			0.01			0.009	0.90
Uncontrolled - Non-Tributary	UNC-1	Hard	0.03		0.9	0.023		
		Soft	0.00		0.2	0.000		
	Subtotal			0.03			0.023	0.90
Total				**0.373**			**0.334**	
Overall Runoff Coefficient= C:								**0.90**

Total Roof Areas	**0.000**	**ha**
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.318**	**ha**
Total Tributary Area to Outlet	**0.318**	**ha**
Total Uncontrolled Areas (Non-Tributary)	**0.055**	**ha**
Park	**0.041**	**ha**
Total Site (Exclude Park)	**0.373**	**ha**

Stormwater Management Calculations

Project #160401697, 1299 Richmond Road
Modified Rational Method Calculations for Storage

2 yr Intensity
City of Ottawa

$I = a/(t + b)^c$

a =	732.951
b =	6.199
c =	0.81

t (min)	I (mm/hr)
10	76.81
20	52.03
30	40.04
40	32.86
50	28.04
60	24.56
70	21.91
80	19.83
90	18.14
100	16.75
110	15.57
120	14.56

2 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 0.373
C: 0.50

Typical Time of Concentration

tc (min)	I (2 yr) (mm/hr)	Qtarget (L/s)
10	76.81	**39.80**

2 YEAR Modified Rational Method for Entire Site

Subdrainage Area: PARK — Uncontrolled - Non-Tributary
Area (ha): 0.04
C: 0.61

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	5.40	5.40		
20	52.03	3.66	3.66		
30	40.04	2.81	2.81		
40	32.86	2.31	2.31		
50	28.04	1.97	1.97		
60	24.56	1.73	1.73		
70	21.91	1.54	1.54		
80	19.83	1.39	1.39		
90	18.14	1.27	1.27		
100	16.75	1.18	1.18		
110	15.57	1.09	1.09		
120	14.56	1.02	1.02		

Subdrainage Area: CIST 1-3 — Uncontrolled - Tributary
Area (ha): 0.05
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	8.67	8.67		
20	52.03	5.87	5.87		
30	40.04	4.52	4.52		
40	32.86	3.71	3.71		
50	28.04	3.16	3.16		
60	24.56	2.77	2.77		
70	21.91	2.47	2.47		
80	19.83	2.24	2.24		
90	18.14	2.05	2.05		
100	16.75	1.89	1.89		
110	15.57	1.76	1.76		
120	14.56	1.64	1.64		

Subdrainage Area: CIST 1-2 — Uncontrolled - Tributary
Area (ha): 0.01
C: 0.80

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	1.98	1.98		
20	52.03	1.34	1.34		
30	40.04	1.03	1.03		
40	32.86	0.85	0.85		
50	28.04	0.72	0.72		
60	24.56	0.63	0.63		
70	21.91	0.57	0.57		
80	19.83	0.51	0.51		
90	18.14	0.47	0.47		
100	16.75	0.43	0.43		
110	15.57	0.40	0.40		
120	14.56	0.38	0.38		

Subdrainage Area: CIST 1-1 — Controlled - Tributary
Area (ha): 0.26
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	60.81	12.48	48.33	29.00
20	52.03	41.20	12.48	28.72	34.46
30	40.04	31.70	12.48	19.23	**34.61**
40	32.86	26.02	12.48	13.54	32.50
50	28.04	22.20	12.48	9.72	29.17
60	24.56	19.44	12.48	6.97	25.08
70	21.91	17.35	12.48	4.87	20.46
80	19.83	15.70	12.48	3.22	15.47
90	18.14	14.36	12.48	1.89	10.19
100	16.75	13.26	12.48	0.78	4.68
110	15.57	12.33	12.33	0.00	0.00
120	14.56	11.53	11.53	0.00	0.00

Notes:
1) All flows from subcatchment areas CIST 1-1 to CIST 1-3 outlet to the cistern
2) Discharge to be controlled to 12.48 L/s

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
2-year Water Level	-	-	12.48	34.61	140.00	OK

Subdrainage Area: UNC-3 — Uncontrolled - Non-Tributary
Area (ha): 0.02

Project #160401697, 1299 Richmond Road
Modified Rational Method Calculations for Storage

100 yr Intensity
City of Ottawa

$I = a/(t + b)^c$

a =	1735.688
b =	6.014
c =	0.820

t (min)	I (mm/hr)
10	178.56
20	119.95
30	91.87
40	75.15
50	63.95
60	55.89
70	49.79
80	44.99
90	41.11
100	37.90
110	35.20
120	32.89

100 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 0.373
C: 0.50

Estimated Time of Concentration after Development

tc (min)	I (100 yr) (mm/hr)	Q100yr (L/s)
10	178.56	**92.52**

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: PARK — Uncontrolled - Non-Tributary
Area (ha): 0.04
C: 0.76

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	15.68	15.68		
20	119.95	10.54	10.54		
30	91.87	8.07	8.07		
40	75.15	6.60	6.60		
50	63.95	5.62	5.62		
60	55.89	4.91	4.91		
70	49.79	4.37	4.37		
80	44.99	3.95	3.95		
90	41.11	3.61	3.61		
100	37.90	3.33	3.33		
110	35.20	3.09	3.09		
120	32.89	2.89	2.89		

Subdrainage Area: CIST 1-3 — Uncontrolled - Tributary
Area (ha): 0.05
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	22.38	22.38		
20	119.95	15.04	15.04		
30	91.87	11.52	11.52		
40	75.15	9.42	9.42		
50	63.95	8.02	8.02		
60	55.89	7.01	7.01		
70	49.79	6.24	6.24		
80	44.99	5.64	5.64		
90	41.11	5.15	5.15		
100	37.90	4.75	4.75		
110	35.20	4.41	4.41		
120	32.89	4.12	4.12		

Subdrainage Area: CIST 1-2 — Uncontrolled - Tributary
Area (ha): 0.01
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	5.76	5.76		
20	119.95	3.87	3.87		
30	91.87	2.97	2.97		
40	75.15	2.43	2.43		
50	63.95	2.06	2.06		
60	55.89	1.80	1.80		
70	49.79	1.61	1.61		
80	44.99	1.45	1.45		
90	41.11	1.33	1.33		
100	37.90	1.22	1.22		
110	35.20	1.14	1.14		
120	32.89	1.06	1.06		

Subdrainage Area: CIST 1-1 — Controlled - Tributary
Area (ha): 0.26
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	157.72	12.48	145.24	87.15
20	119.95	105.95	12.48	93.47	112.17
30	91.87	81.15	12.48	68.67	123.61
40	75.15	66.38	12.48	53.90	129.36
50	63.95	56.49	12.48	44.01	132.04
60	55.89	49.37	12.48	36.89	**132.82**
70	49.79	43.98	12.48	31.50	132.31
80	44.99	39.74	12.48	27.26	130.86
90	41.11	36.31	12.48	23.84	128.71
100	37.90	33.48	12.48	21.00	126.01
110	35.20	31.09	12.48	18.62	122.87
120	32.89	29.06	12.48	16.58	119.36

Notes:
1) All flows from subcatchment areas CIST 1-1 to CIST 1-3 outlet to the cistern
2) Discharge to be controlled to 12.48 L/s

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	-	-	12.48	132.82	140.00	OK
					7.18	

Subdrainage Area: UNC-3 — Uncontrolled - Non-Tributary
Area (ha): 0.02

Stormwater Management Calculations

Project #160401697, 1299 Richmond Road
Modified Rational Method Calculations for Storage

C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	3.55	3.55		
20	52.03	2.40	2.40		
30	40.04	1.85	1.85		
40	32.86	1.52	1.52		
50	28.04	1.30	1.30		
60	24.56	1.13	1.13		
70	21.91	1.01	1.01		
80	19.83	0.92	0.92		
90	18.14	0.84	0.84		
100	16.75	0.77	0.77		
110	15.57	0.72	0.72		
120	14.56	0.67	0.67		

Subdrainage Area: UNC-2 Uncontrolled - Non-Tributary
Area (ha): 0.01
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	2.02	2.02		
20	52.03	1.37	1.37		
30	40.04	1.05	1.05		
40	32.86	0.86	0.86		
50	28.04	0.74	0.74		
60	24.56	0.64	0.64		
70	21.91	0.58	0.58		
80	19.83	0.52	0.52		
90	18.14	0.48	0.48		
100	16.75	0.44	0.44		
110	15.57	0.41	0.41		
120	14.56	0.38	0.38		

Subdrainage Area: UNC-1 Uncontrolled - Non-Tributary
Area (ha): 0.03
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	5.01	5.01		
20	52.03	3.39	3.39		
30	40.04	2.61	2.61		
40	32.86	2.14	2.14		
50	28.04	1.83	1.83		
60	24.56	1.60	1.60		
70	21.91	1.43	1.43		
80	19.83	1.29	1.29		
90	18.14	1.18	1.18		
100	16.75	1.09	1.09		
110	15.57	1.02	1.02		
120	14.56	0.95	0.95		

SUMMARY TO OUTLET

			Vrequired	Vavailable*	
Tributary Area	0.3	ha			
Total 2yr Flow to Cistern	60.8	L/s	34.61	140.00 m^3	Ok
Total 2yr Flow from Cistern	12.5	L/s			
Non-Tributary Area	0.1	ha			
Total 2yr Flow Uncontrolled	10.6	L/s			
Park Area	0.0	ha			
Total 2yr Flow from Park	5.4	L/s			
Total Area (Excluding Park)	0.4	ha			
Total 2yr Flow	17.9	L/s			
Target	39.8	L/s			

Project #160401697, 1299 Richmond Road
Modified Rational Method Calculations for Storage

C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	9.17	9.17		
20	119.95	6.16	6.16		
30	91.87	4.72	4.72		
40	75.15	3.86	3.86		
50	63.95	3.28	3.28		
60	55.89	2.87	2.87		
70	49.79	2.56	2.56		
80	44.99	2.31	2.31		
90	41.11	2.11	2.11		
100	37.90	1.95	1.95		
110	35.20	1.81	1.81		
120	32.89	1.69	1.69		

Subdrainage Area: UNC-2 Uncontrolled - Non-Tributary
Area (ha): 0.01
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	5.21	5.21		
20	119.95	3.50	3.50		
30	91.87	2.68	2.68		
40	75.15	2.19	2.19		
50	63.95	1.87	1.87		
60	55.89	1.63	1.63		
70	49.79	1.45	1.45		
80	44.99	1.31	1.31		
90	41.11	1.20	1.20		
100	37.90	1.11	1.11		
110	35.20	1.03	1.03		
120	32.89	0.96	0.96		

Subdrainage Area: UNC-1 Uncontrolled - Non-Tributary
Area (ha): 0.03
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	12.94	12.94		
20	119.95	8.69	8.69		
30	91.87	6.66	6.66		
40	75.15	5.45	5.45		
50	63.95	4.63	4.63		
60	55.89	4.05	4.05		
70	49.79	3.61	3.61		
80	44.99	3.26	3.26		
90	41.11	2.98	2.98		
100	37.90	2.75	2.75		
110	35.20	2.55	2.55		
120	32.89	2.38	2.38		

SUMMARY TO OUTLET

			Vrequired	Vavailable*	
Tributary Area	0.3	ha			
Total 100yr Flow to Cistern	157.7	L/s	132.82	140.00 m^3	Ok
Total 100yr Flow from Cistern	12.5	L/s			
Non-Tributary Area	0.1	ha			
Total 100yr Flow Uncontrolled	27.3	L/s			
Park Area	0.0	ha			
Total 100yr Flow from Park	15.7	L/s			
Total Area (Excluding Park)	0.4	ha			
Total 100yr Flow	39.8	L/s			
Target	39.8	L/s			

D.2 Storm Sewer Design Sheet

Stantec

1299 Richmond Road

DATE: 2023-05-18
REVISION: 1
DESIGNED BY: MW
CHECKED BY: PM

FILE NUMBER: 160401697

STORM SEWER DESIGN SHEET
(City of Ottawa)

DESIGN PARAMETERS

$I = a / (t+b)^c$ (As per City of Ottawa Guidelines, 2012)

	1:2 yr	1:5 yr	1:10 yr	1:100 yr
a =	732.951	998.071	1174.184	1735.688
b =	6.199	6.053	6.014	6.014
c =	0.810	0.814	0.816	0.820

MANNING'S n = 0.013 BEDDING CLASS = B
MINIMUM COVER: 2.00 m
TIME OF ENTRY 10 min

LOCATION			DRAINAGE AREA																					PIPE SELECTION										
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (5-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (100-YEAR) (ha)	AREA (ROOF) (ha)	C (2-YEAR) (-)	C (5-YEAR) (-)	C (10-YEAR) (-)	C (100-YEAR) (-)	A x C (2-YEAR) (ha)	ACCUM AxC (2YR) (ha)	A x C (5-YEAR) (ha)	ACCUM. AxC (5YR) (ha)	A x C (10-YEAR) (ha)	ACCUM. AxC (10YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)	T of C (min)	$I_{2\text{-YEAR}}$ (mm/h)	$I_{5\text{-YEAR}}$ (mm/h)	$I_{10\text{-YEAR}}$ (mm/h)	$I_{100\text{-YEAR}}$ (mm/h)	$Q_{CONTROL}$ (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)	LENGTH (m)	PIPE WIDTH OR DIAMETEI (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE %	Q_{CAP} (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	TIME OF FLOW (min)
CISTERN	Building	100	0.32	0.00	0.00	0.00	0.00	0.90	0.00	0.00	0.00	0.287	0.287	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	61.2	7.3	300	300	CIRCULAR	CONCRETE	-	1.00	96.2	**63.68%**	1.37	0.10
																				10.10																	

D.3 SWM Quality Control Criteria Correspondence

Mott, Peter

From: Eric Lalande <eric.lalande@rvca.ca>
Sent: Tuesday, May 30, 2023 2:34 PM
To: Mott, Peter
Cc: Kilborn, Kris
Subject: RE: 1299 Richmond Road - Water Quality Control

Hi Peter,

While the RVCA likely would not have had any requirements, water quality control is now being handled by the City of Ottawa's infrastructure, I would direct you to them for review and comments.

Cheers,

Eric Lalande, MCIP, RPP
Planner, RVCA
613-692-3571 x1137

From: Mott, Peter <Peter.Mott@stantec.com>
Sent: Tuesday, May 30, 2023 2:07 PM
To: Eric Lalande <eric.lalande@rvca.ca>
Cc: Kilborn, Kris <kris.kilborn@stantec.com>
Subject: 1299 Richmond Road - Water Quality Control

Hi Eric,

We've been retained to help provide a design for a development containing a single mixed-use building with a 5-storey podium and two residential high-rise towers at 1299 Richmond Road in Ottawa. The site is shown in the attached SSP drawing.

We are looking to confirm if quality control measures are required on-site. Please review the site servicing plan attached and confirm if quality treatment is required for the site. If you need any other information, feel free to reach out.

Best regards,

Peter Mott EIT
Engineering Intern, Community Development

Mobile: +1 (613) 897-0445
Teams: +1 (613) 724-4370
Peter.Mott@stantec.com
Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

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Stormwater Management Design Criteria

for the Pinecrest Creek/Westboro Area

City of Ottawa

Final – May 2020

Table 1: SWM Design Criteria for the Pinecrest Creek / Westboro Study Area

	Development Type	Runoff Volume Reduction	Water Quality	Water Quantity	
			TSS Removal	Flood Control	Erosion Control
All Locations					
Residential Development *not* subject to Plan of Subdivision or Site Plan Control approval(s)					
1	all soil infiltration rates	Direction/re-direction of downspouts/roof drainage to discharge to pervious surfaces, where possible, to reduce runoff, while meeting all other City of Ottawa lot grading requirements. Amended topsoil, or a depth of topsoil up to 300 mm, provides runoff volume reduction benefits and is encouraged (but not mandatory) as a best practice over all soft landscaped surfaces.	Not applicable	Not applicable	Not applicable
Draining to the Ottawa River					
Development subject to Plan of Subdivision or Site Plan Control approval(s) - discharging directly to the Ottawa River					
2	all soil infiltration rates	A minimum on-site retention of the 10 mm design storm; refer to LID references[(i)] for guidance on prudent approach to planning infiltration-based LID best management practices. Assumptions re: non-viability of infiltration measures must be substantiated. A green roof, rain harvesting measures and/or a combination of detention/retention measures[(ii)] could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS; some of which ma be achieved by on-site retention of first 10 mm of rainfall.	As per City of Ottawa Sewer Design Guideline	Not applicable
Draining to Pinecrest Creek					
Development subject to Plan of Subdivision or Site Plan Control approval(s) - discharging upstream of the Ottawa River Parkway pipe (ORPP) inlet					
3	all soil infiltration rates	A minimum on-site retention of the 10 mm design storm; refer to LID references[(i)] for guidance on prudent approach to planning infiltration-based LID best management practices. Assumptions re: non-viability of infiltration measures must be substantiated. A green roof, rain harvesting measures and/or a combination of detention/retention measures[(ii)] could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS; some of which may be achieved by on-site retention of first 10 mm of rainfall and detention of the 25 mm design storm[(iii)].	The more stringent of the following criteria will govern: i) 1:100 year discharge from site not to exceed 33.5 L/s/ha) or; ii) Requirements of City of Ottawa Sewer Design Guideline.	Control (detain) the runoff from the 25 mm design storm[(iii)] such that the peak outflow from the site does not exceed 5.8 L/s/ha.
Development subject to Plan of Subdivision or Site Plan Control approval(s) - discharging directly to the Ottawa River Parkway pi					
4	all soil infiltration rates	A minimum on-site retention of the 10 mm design storm; refer to LID references[(i)] for guidance on prudent approach to planning infiltration-based LID best management practices. Assumptions re: non-viability of infiltration measures must be substantiated. A green roof, rain harvesting measures and/or a combination of detention/retention measures[(ii)] could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS; some of which may be achieved by on-site retention of first 10 mm of rainfall.	The more stringent of the following criteria will govern: i) 1:100 year discharge from site not to exceed 33.5 L/s/ha) or; ii) Requirements of City of Ottawa Sewer Design	Not applicable

Notes:

(i) Re: Infiltration measures: Beyond the targets specified in this table, the planning, design and use of these systems shall be in accordance with the guidance in the Stormwater Management Planning and Design Manual (MOE, 2003); the Low Impact Development Stormwater Management Planning and Design Guide (CVC and TRCA, 2010); the Low Impact Development Stormwater Management Planning and Design Wiki at: wiki.sustainabletechnologies.ca; and Draft No.2 Low Impact Development (LID) Stormwater Management Guidance Manual (MOECC, November 2017) or the final version of this Manual, when available. As noted in the MOECC LID SWM Guidance Manual, a prudent approach to planning infiltration-based LID best management practices on any site involves delineating catchment areas that contain high risk site activities and isolating them by applying non-infiltration-based practices to these areas.

(ii) Retention is to hold or retain stormwater on a more permanent basis such as for infiltration to the surrounding soils. Detention is the temporary storage or detaining of stormwater for eventual release to the downstream system.

(iii) 25 mm 4-hour Chicago design storm

D.4 Stormceptor Sizing Report and Detail

Stormceptor®

FORTERRA

Stormceptor® EF Sizing Report

STORMCEPTOR®
ESTIMATED NET ANNUAL SEDIMENT (TSS) LOAD REDUCTION

06/01/2023

Province:	Ontario
City:	Ottawa
Nearest Rainfall Station:	OTTAWA CDA RCS
Climate Station Id:	6105978
Years of Rainfall Data:	20

Site Name:	1299 RR Cistern

Drainage Area (ha):	0.32
Runoff Coefficient 'c':	0.90

Particle Size Distribution:	Fine
Target TSS Removal (%):	80.0

Required Water Quality Runoff Volume Capture (%):	90.00
Estimated Water Quality Flow Rate (L/s):	9.30
Oil / Fuel Spill Risk Site?	No
Upstream Flow Control?	Yes
Upstream Orifice Control Flow Rate to Stormceptor (L/s):	12.50
Peak Conveyance (maximum) Flow Rate (L/s):	
Site Sediment Transport Rate (kg/ha/yr):	

Project Name:	1299 Richmond Road - Brigil Towers
Project Number:	160401697
Designer Name:	Peter Mott
Designer Company:	Stantec Consulting Ltd.
Designer Email:	peter.mott05@gmail.com
Designer Phone:	613-897-0445
EOR Name:	Peter Moroz
EOR Company:	Stantec Consulting Ltd.
EOR Email:	peter.moroz@stantec.com
EOR Phone:	613-294-2851

Net Annual Sediment (TSS) Load Reduction Sizing Summary

Stormceptor Model	TSS Removal Provided (%)
EF4	90
EF6	96
EF8	99
EF10	100
EF12	100

Recommended Stormceptor EF Model: EF4

Estimated Net Annual Sediment (TSS) Load Reduction (%): 90

Water Quality Runoff Volume Capture (%): > 90

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THIRD-PARTY TESTING AND VERIFICATION

►**Stormceptor® EF and Stormceptor® EFO** are the latest evolutions in the Stormceptor® oil-grit separator (OGS) technology series, and are designed to remove a wide variety of pollutants from stormwater and snowmelt runoff. These technologies have been third-party tested in accordance with the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators** and performance has been third-party verified in accordance with the **ISO 14034 Environmental Technology Verification (ETV)** protocol.

PERFORMANCE

►**Stormceptor® EF and EFO** remove stormwater pollutants through gravity separation and floatation, and feature a patent-pending design that generates positive removal of total suspended solids (TSS) throughout each storm event, including high-intensity storms. Captured pollutants include sediment, free oils, and sediment-bound pollutants such as nutrients, heavy metals, and petroleum hydrocarbons. Stormceptor is sized to remove a high level of TSS from the frequent rainfall events that contribute the vast majority of annual runoff volume and pollutant load. The technology incorporates an internal bypass to convey excessive stormwater flows from high-intensity storms through the device without resuspension and washout (scour) of previously captured pollutants. Proper routine maintenance ensures high pollutant removal performance and protection of downstream waterwavs.

PARTICLE SIZE DISTRIBUTION (PSD)

►The **Canadian ETV PSD** shown in the table below was used, or in part, for this sizing. This is the identical PSD that is referenced in the Canadian ETV ***Procedure for Laboratory Testing of Oil-Grit Separators*** for both sediment removal testing and scour testing. The Canadian ETV PSD contains a wide range of particle sizes in the sand and silt fractions, and is considered reasonably representative of the particle size fractions found in typical urban stormwater runoff.

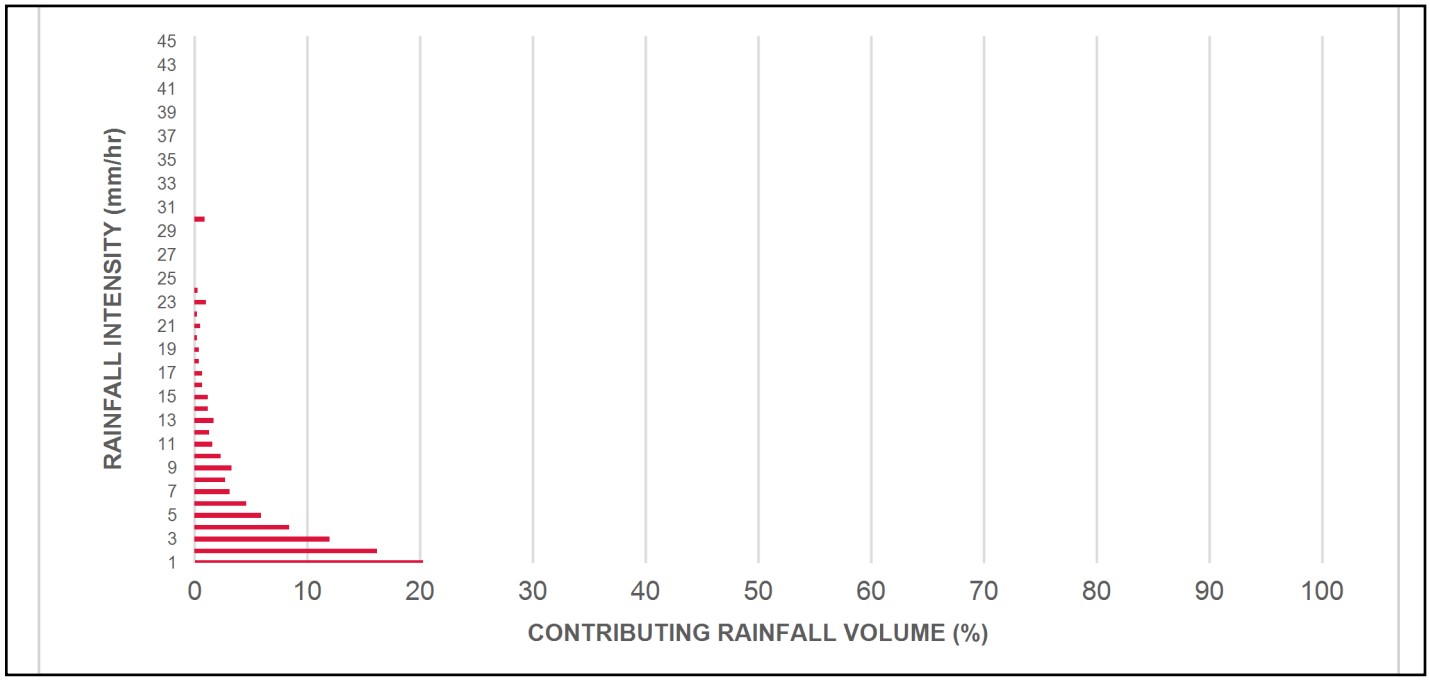
Particle Size (µm)	Percent Less Than	Particle Size Fraction (µm)	Percent
1000	100	500-1000	5
500	95	250-500	5
250	90	150-250	15
150	75	100-150	15
100	60	75-100	10
75	50	50-75	5
50	45	20-50	10
20	35	8-20	15
8	20	5-8	10
5	10	2-5	5
2	5	<2	5

Upstream Flow Controlled Results

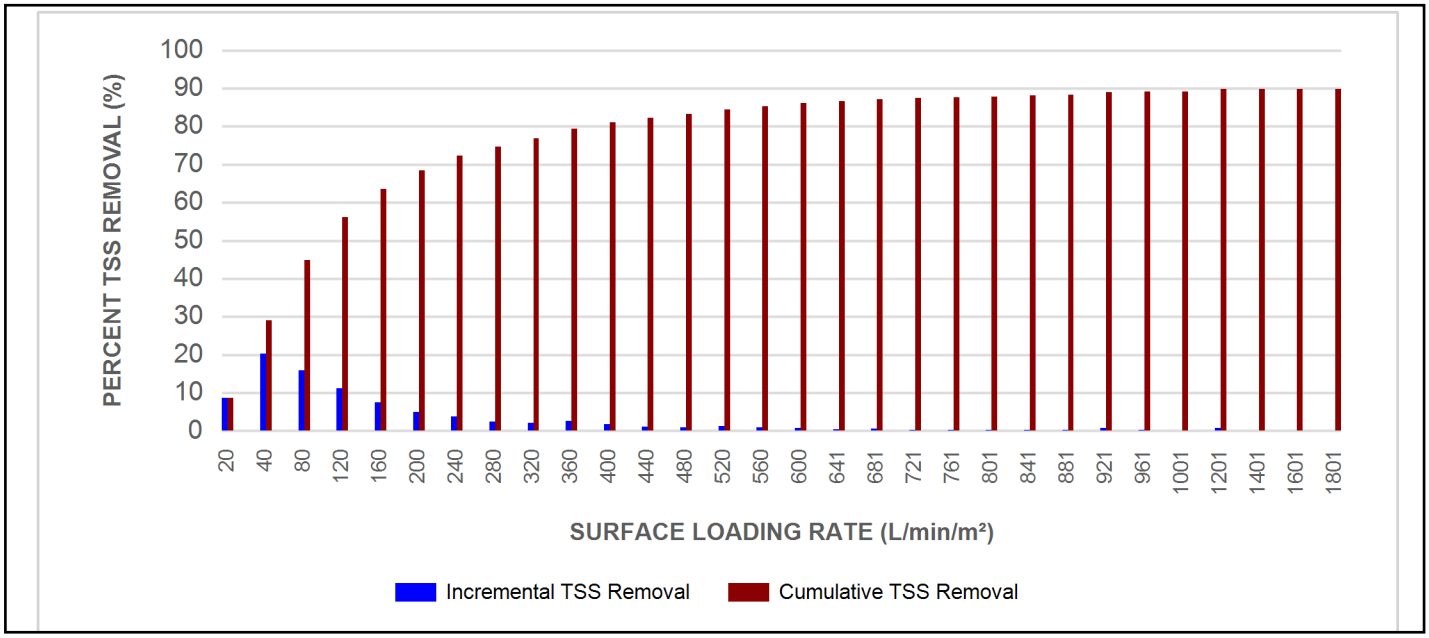
Rainfall Intensity (mm / hr)	Percent Rainfall Volume (%)	Cumulative Rainfall Volume (%)	Flow Rate (L/s)	Flow Rate (L/min)	Surface Loading Rate (L/min/m²)	Removal Efficiency (%)	Incremental Removal (%)	Cumulative Removal (%)
0.5	8.6	8.6	0.40	24.0	20.0	100	8.6	8.6
1	20.3	29.0	0.80	48.0	40.0	100	20.3	29.0
2	16.2	45.2	1.60	96.0	80.0	98	16.0	44.9
3	12.0	57.2	2.40	144.0	120.0	93	11.2	56.1
4	8.4	65.6	3.20	192.0	160.0	88	7.4	63.6
5	5.9	71.6	4.00	240.0	200.0	83	4.9	68.5
6	4.6	76.2	4.80	288.0	240.0	81	3.8	72.3
7	3.1	79.3	5.60	336.0	280.0	79	2.4	74.7
8	2.7	82.0	6.41	384.0	320.0	78	2.1	76.8
9	3.3	85.3	7.21	432.0	360.0	76	2.5	79.4
10	2.3	87.6	8.01	480.0	400.0	74	1.7	81.1
11	1.6	89.2	8.81	528.0	440.0	73	1.1	82.2
12	1.3	90.5	9.61	576.0	480.0	73	1.0	83.2
13	1.7	92.2	10.41	624.0	520.0	72	1.2	84.4
14	1.2	93.5	11.21	673.0	560.0	71	0.9	85.3
15	6.5	100.0	12.00	720.0	600.0	71	4.6	89.9
16	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
17	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
18	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
19	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
20	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
21	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
22	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
23	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
24	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
25	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
30	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
35	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
40	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
45	0.0	100.0	12.00	720.0	600.0	71	0.0	89.9
Estimated Net Annual Sediment (TSS) Load Reduction =								**90 %**

Climate Station ID: 6105978 *Years of Rainfall Data*: 20

RAINFALL DATA FROM OTTAWA CDA RCS RAINFALL STATION



INCREMENTAL AND CUMULATIVE TSS REMOVAL FOR THE RECOMMENDED STORMCEPTOR® MODEL



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Maximum Pipe Diameter / Peak Conveyance

Stormceptor EF / EFO	Model Diameter		Min Angle Inlet / Outlet Pipes	Max Inlet Pipe Diameter		Max Outlet Pipe Diameter		Peak Conveyance Flow Rate	
	(m)	(ft)		(mm)	(in)	(mm)	(in)	(L/s)	(cfs)
EF4 / EFO4	1.2	4	90	609	24	609	24	425	15
EF6 / EFO6	1.8	6	90	914	36	914	36	990	35
EF8 / EFO8	2.4	8	90	1219	48	1219	48	1700	60
EF10 / EFO10	3.0	10	90	1828	72	1828	72	2830	100
EF12 / EFO12	3.6	12	90	1828	72	1828	72	2830	100

SCOUR PREVENTION AND ONLINE CONFIGURATION

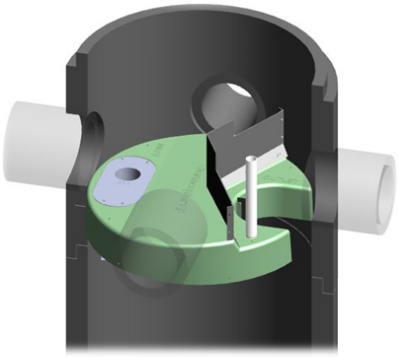
►**Stormceptor® EF and EFO** feature an internal bypass and superior scour prevention technology that have been demonstrated in third-party testing according to the scour testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**, and the exceptional scour test performance has been third-party verified in accordance with the ISO 14034 ETV protocol. As a result, Stormceptor EF and EFO are approved for online installation, eliminating the need for costly additional bypass structures, piping, and installation expense.

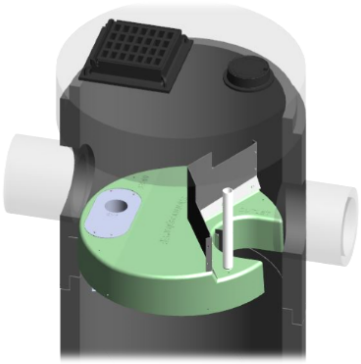
DESIGN FLEXIBILITY

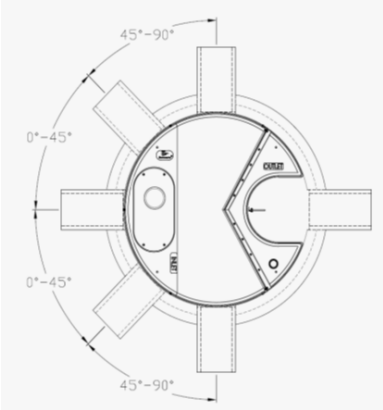
►**Stormceptor® EF** and **EFO** offers design flexibility in one simplified platform, accepting stormwater flow from a single inlet pipe or multiple inlet pipes, and/or surface runoff through an inlet grate. The device can also serve as a junction structure, accommodate a 90-degree inlet-to-outlet bend angle, and can be modified to ensure performance in submerged conditions.

OIL CAPTURE AND RETENTION

►While Stormceptor® EF will capture and retain oil from dry weather spills and low intensity runoff, **Stormceptor® EFO** has demonstrated superior oil capture and greater than 99% oil retention in third-party testing according to the light liquid re-entrainment testing provisions of the Canadian ETV **Procedure for Laboratory Testing of Oil-Grit Separators**. Stormceptor EFO is recommended for sites where oil capture and retention is a requirement.







INLET-TO-OUTLET DROP

Elevation differential between inlet and outlet pipe inverts is dictated by the angle at which the inlet pipe(s) enters the unit.

0° - 45° : The inlet pipe is 1-inch (25mm) higher than the outlet pipe.

45° - 90° : The inlet pipe is 2-inches (50mm) higher than the outlet pipe.

HEAD LOSS

The head loss through Stormceptor EF is similar to that of a 60-degree bend structure. The applicable K value for calculating minor losses through the unit is 1.1. For submerged conditions the applicable K value is 3.0.

Pollutant Capacity

Stormceptor EF / EFO	Model Diameter		Depth (Outlet Pipe Invert to Sump Floor)		Oil Volume		Recommended Sediment Maintenance Depth *		Maximum Sediment Volume *		Maximum Sediment Mass **	
	(m)	(ft)	(m)	(ft)	(L)	(Gal)	(mm)	(in)	(L)	(ft^3)	(kg)	(lb)
EF4 / EFO4	1.2	4	1.52	5.0	265	70	203	8	1190	42	1904	5250
EF6 / EFO6	1.8	6	1.93	6.3	610	160	305	12	3470	123	5552	15375
EF8 / EFO8	2.4	8	2.59	8.5	1070	280	610	24	8780	310	14048	38750
EF10 / EFO10	3.0	10	3.25	10.7	1670	440	610	24	17790	628	28464	78500
EF12 / EFO12	3.6	12	3.89	12.8	2475	655	610	24	31220	1103	49952	137875

*Increased sump depth may be added to increase sediment storage capacity

** Average density of wet packed sediment in sump = 1.6 kg/L (100 lb/ft^3)

Feature	Benefit	Feature Appeals To
Patent-pending enhanced flow treatment and scour prevention technology	Superior, verified third-party performance	Regulator, Specifying & Design Engineer
Third-party verified light liquid capture and retention for EFO version	Proven performance for fuel/oil hotspot locations	Regulator, Specifying & Design Engineer, Site Owner
Functions as bend, junction or inlet structure	Design flexibility	Specifying & Design Engineer
Minimal drop between inlet and outlet	Site installation ease	Contractor
Large diameter outlet riser for inspection and maintenance	Easy maintenance access from grade	Maintenance Contractor & Site Owner

STANDARD STORMCEPTOR EF/EFO DRAWINGS

For standard details, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

STANDARD STORMCEPTOR EF/EFO SPECIFICATION

For specifications, please visit http://www.imbriumsystems.com/stormwater-treatment-solutions/stormceptor-ef

STANDARD PERFORMANCE SPECIFICATION FOR "OIL GRIT SEPARATOR" (OGS) STORMWATER QUALITY TREATMENT DEVICE

PART 1 – GENERAL

1.1 WORK INCLUDED

This section specifies requirements for selecting, sizing, and designing an underground Oil Grit Separator (OGS) device for stormwater quality treatment, with third-party testing results and a Statement of Verification in accordance with ISO 14034 Environmental Management – Environmental Technology Verification (ETV).

1.2 REFERENCE STANDARDS & PROCEDURES

ISO 14034:2016 Environmental management – Environmental technology verification (ETV)

Canadian Environmental Technology Verification (ETV) Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**

1.3 SUBMITTALS

1.3.1 All submittals, including sizing reports & shop drawings, shall be submitted upon request with each order to the contractor then forwarded to the Engineer of Record for review and acceptance. Shop drawings shall detail all OGS components, elevations, and sequence of construction.

1.3.2 Alternative devices shall have features identical to or greater than the specified device, including: treatment chamber diameter, treatment chamber wet volume, sediment storage volume, and oil storage volume.

1.3.3 Unless directed otherwise by the Engineer of Record, OGS stormwater quality treatment product substitutions or alternatives submitted within ten days prior to project bid shall not be accepted. All alternatives or substitutions submitted shall be signed and sealed by a local registered Professional Engineer, based on the exact same criteria detailed in Section 3, in entirety, subject to review and approval by the Engineer of Record.

PART 2 – PRODUCTS

2.1 OGS POLLUTANT STORAGE

The OGS device shall include a sump for sediment storage, and a protected volume for the capture and storage of petroleum hydrocarbons and buoyant gross pollutants. The **minimum** sediment & petroleum hydrocarbon storage capacity shall be as follows:

2.1.1

4 ft (1219 mm) Diameter OGS Units:	1.19 m^3 sediment / 265 L oil
6 ft (1829 mm) Diameter OGS Units:	3.48 m^3 sediment / 609 L oil
8 ft (2438 mm) Diameter OGS Units:	8.78 m^3 sediment / 1,071 L oil
10 ft (3048 mm) Diameter OGS Units:	17.78 m^3 sediment / 1,673 L oil
12 ft (3657 mm) Diameter OGS Units:	31.23 m^3 sediment / 2,476 L oil

PART 3 – PERFORMANCE & DESIGN

3.1 GENERAL

The OGS stormwater quality treatment device shall be verified in accordance with ISO 14034:2016 Environmental management – Environmental technology verification (ETV). The OGS stormwater quality treatment device shall remove oil, sediment and gross pollutants from stormwater runoff during frequent wet weather events, and retain these pollutants during less frequent high flow wet weather events below the insert within the OGS for later removal during maintenance. The Manufacturer shall have at least ten (10) years of local experience, history and success in engineering design, manufacturing and production and supply of OGS stormwater quality treatment device systems, acceptable to the Engineer of Record.

3.2 SIZING METHODOLOGY

The OGS device shall be engineered, designed and sized to provide stormwater quality treatment based on treating a minimum of 90 percent of the average annual runoff volume and a minimum removal of an annual average 60% of the sediment (TSS) load based on the Particle Size Distribution (PSD) specified in the sizing report for the specified device. Sizing of the OGS shall be determined by use of a minimum ten (10) years of local historical rainfall data provided by Environment Canada. Sizing shall also be determined by use of the sediment removal performance data derived from the ISO 14034 ETV third-party verified laboratory testing data from testing conducted in accordance with the Canadian ETV protocol Procedure for Laboratory Testing of Oil-Grit Separators, as follows:

3.2.1 Sediment removal efficiency for a given surface loading rate and its associated flow rate shall be based on sediment removal efficiency demonstrated at the seven (7) tested surface loading rates specified in the protocol, ranging 40 L/min/m^2 to 1400 L/min/m^2, and as stated in the ISO 14034 ETV Verification Statement for the OGS device.

3.2.2 Sediment removal efficiency for surface loading rates between 40 L/min/m^2 and 1400 L/min/m^2 shall be based on linear interpolation of data between consecutive tested surface loading rates.

3.2.3 Sediment removal efficiency for surface loading rates less than the lowest tested surface loading rate of 40 L/min/m^2 shall be assumed to be identical to the sediment removal efficiency at 40 L/min/m^2. No extrapolation shall be allowed that results in a sediment removal efficiency that is greater than that demonstrated at 40 L/min/m^2.

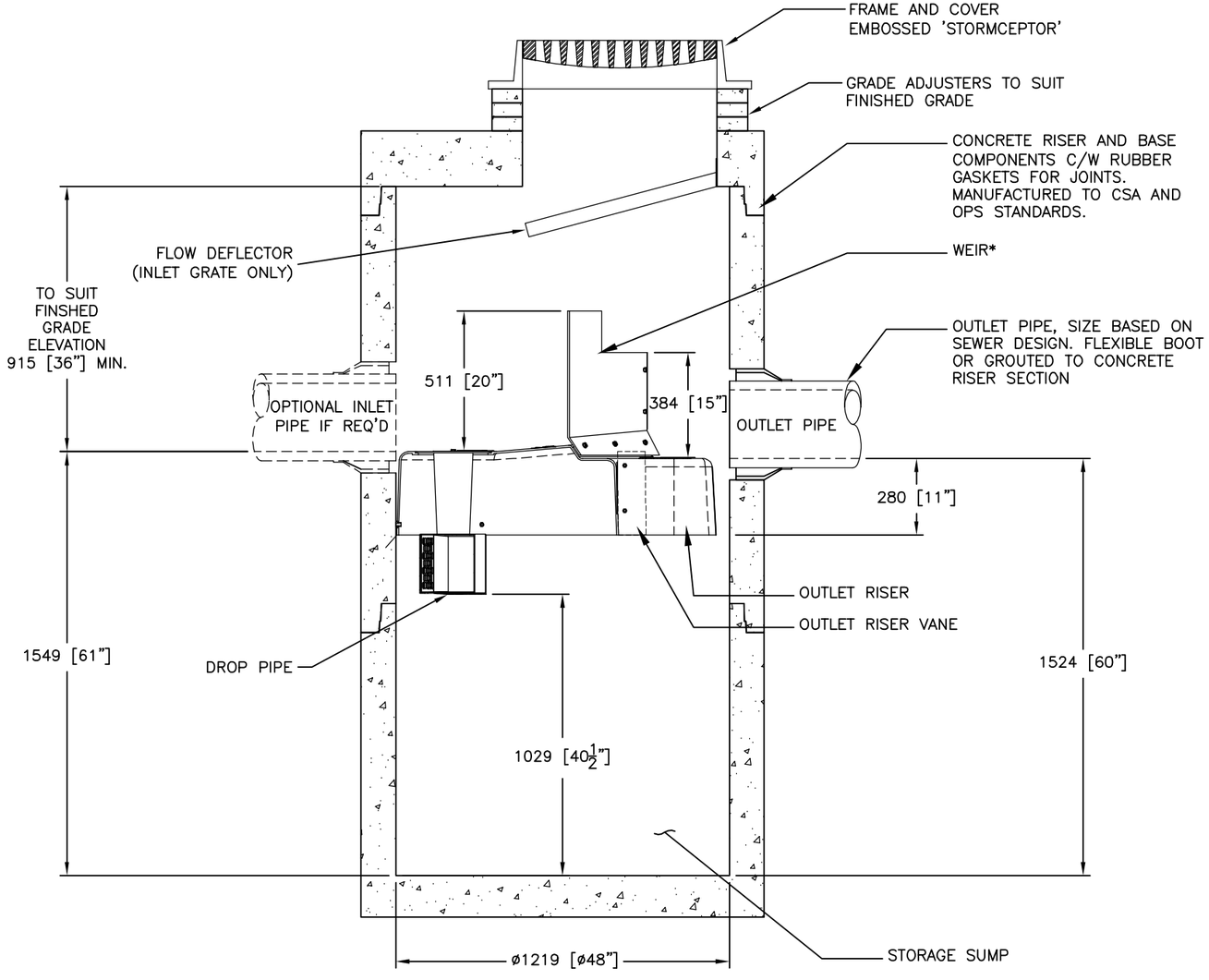
3.2.4 Sediment removal efficiency for surface loading rates greater than the highest tested surface loading rate of 1400 L/min/m^2 shall assume zero sediment removal for the portion of flow that exceeds 1400 L/min/m^2, and shall be calculated using a simple proportioning formula, with 1400 L/min/m^2 in the numerator and the higher surface loading rate in the denominator, and multiplying the resulting fraction times the sediment removal efficiency at 1400 L/min/m^2.

The OGS device shall also have sufficient annual sediment storage capacity as specified and calculated in Section 2.1.

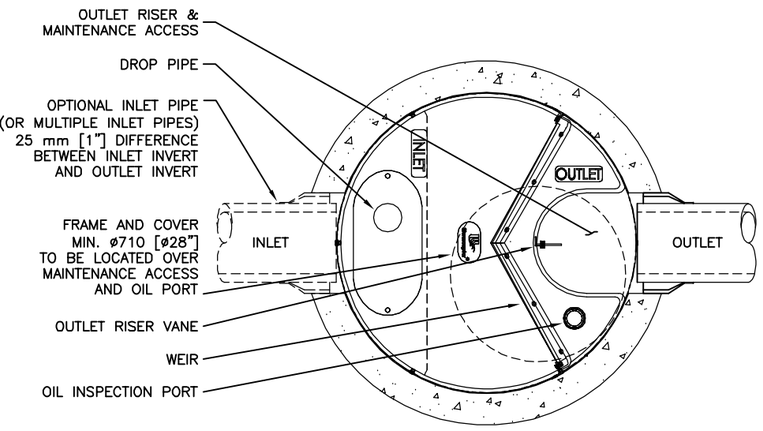
3.3 CANADIAN ETV or ISO 14034 ETV VERIFICATION OF SCOUR TESTING

The OGS device shall have Canadian ETV or ISO 14034 ETV Verification of third-party scour testing conducted in accordance with the Canadian ETV Program's **Procedure for Laboratory Testing of Oil-Grit Separators.**

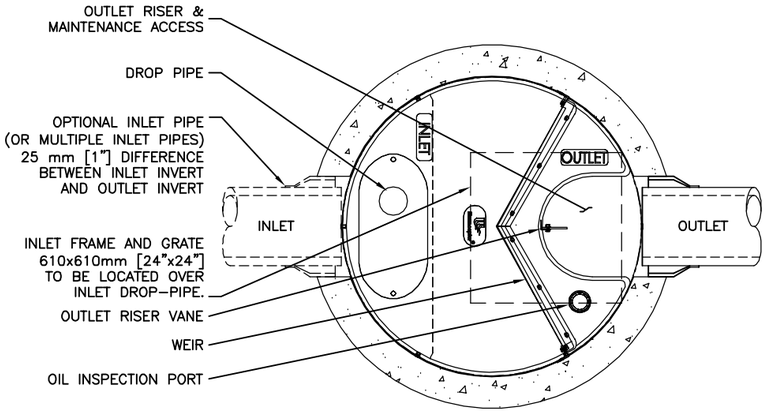
3.3.1 To be acceptable for on-line installation, the OGS device must demonstrate an average scour test effluent concentration less than 10 mg/L at each surface loading rate tested, up to and including 2600 L/min/m^2.



SECTION VIEW



PLAN VIEW (STANDARD)



PLAN VIEW (INLET TOP)

FOR SITE SPECIFIC DRAWINGS PLEASE CONTACT YOUR LOCAL STORMCEPTOR REPRESENTATIVE. SITE SPECIFIC DRAWINGS ARE BASED ON THE BEST AVAILABLE INFORMATION AT THE TIME. SOME FIELD REVISIONS TO THE SYSTEM LOCATION OR CONNECTION PIPING MAY BE NECESSARY BASED ON AVAILABLE SPACE OR SITE CONFIGURATION REVISIONS. ELEVATIONS SHOULD BE MAINTAINED EXCEPT WHERE NOTED ON BYPASS STRUCTURE (IF REQUIRED).

GENERAL NOTES:

* MAXIMUM SURFACE LOADING RATE (SLR) INTO LOWER CHAMBER THROUGH DROP PIPE IS 1135 L/min/m^2 (27.9 gpm/ft^2) FOR STORMCEPTOR EF4 AND 535 L/min/m^2 (13.1 gpm/ft^2) FOR STORMCEPTOR EF04 (OIL CAPTURE CONFIGURATION). WEIR HEIGHT IS 150 mm (6 INCH) FOR EF04.

1. ALL DIMENSIONS INDICATED ARE IN MILLIMETERS (INCHES) UNLESS OTHERWISE SPECIFIED.
2. STORMCEPTOR STRUCTURE INLET AND OUTLET PIPE SIZE AND ORIENTATION SHOWN FOR INFORMATIONAL PURPOSES ONLY.
3. UNLESS OTHERWISE NOTED, BYPASS INFRASTRUCTURE, SUCH AS ALL UPSTREAM DIVERSION STRUCTURES, CONNECTING STRUCTURES, OR PIPE CONDUITS CONNECTING TO COMPLETE THE STORMCEPTOR SYSTEM SHALL BE PROVIDED AND ADDRESSED SEPARATELY.
4. DRAWING FOR INFORMATION PURPOSES ONLY. REFER TO ENGINEER'S SITE/UTILITY PLAN FOR STRUCTURE ORIENTATION.
5. NO PRODUCT SUBSTITUTIONS SHALL BE ACCEPTED UNLESS SUBMITTED 10 DAYS PRIOR TO PROJECT BID DATE, OR AS DIRECTED BY THE ENGINEER OF RECORD.

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 STRUCTURE (LIFTING CLUTCHES PROVIDED)
C. CONTRACTOR WILL INSTALL AND LEVEL THE STRUCTURE, SEALING THE JOINTS, LINE ENTRY AND EXIT POINTS (NON-SHRINK GROUT WITH APPROVED WATERSTOP OR FLEXIBLE BOOT)
D. CONTRACTOR TO TAKE APPROPRIATE MEASURES TO PROTECT THE DEVICE FROM CONSTRUCTION-RELATED EROSION RUNOFF.
E. DEVICE ACTIVATION, BY CONTRACTOR, SHALL OCCUR ONLY AFTER SITE HAS BEEN STABILIZED AND THE STORMCEPTOR UNIT IS CLEAN AND FREE OF DEBRIS.

STANDARD DETAIL
NOT FOR CONSTRUCTION

SITE SPECIFIC DATA REQUIREMENTS

STORMCEPTOR MODEL	EF4
STRUCTURE ID	*
WATER QUALITY FLOW RATE (L/s)	*
PEAK FLOW RATE (L/s)	*
RETURN PERIOD OF PEAK FLOW (yrs)	*
DRAINAGE AREA (HA)	*
DRAINAGE AREA IMPERVIOUSNESS (%)	*

PIPE DATA:	I.E.	MAT'L	DIA	SLOPE %	HGL
INLET #1	*	*	*	*	*
INLET #2	*	*	*	*	*
OUTLET	*	*	*	*	*

* PER ENGINEER OF RECORD

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MARK	DATE	REVISION DESCRIPTION	BY
1	6/8/18	UPDATES	JSK
0	5/26/17	INITIAL RELEASE	JSK

Stormceptor® EF

SCALE = NTS

imbrium®
7007 RIDGE ROAD, SUITE 550, HANOVER, MD 21076
USA: 888-279-9828 CA: 800-388-4601 INTL: +1-416-960-9900

DATE: 5/26/2017

DESIGNED:	DRAWN:
JSK	JSK
CHECKED:	APPROVED:
BSF	SP
PROJECT No.:	SEQUENCE No.:
EF4	*

SHEET: 1 OF 1

Appendix E Background Studies

PATERSON GROUP

Geotechnical Investigation

Proposed Multi-Storey Building

1299 Richmond Road
Ottawa, Ontario

Prepared for Brigil Construction

Report PG6598 – 1 dated April 25, 2023

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Appendix 1 Soil Profile and Test Data Sheets
Symbols and Terms
Analytical Testing Results

Appendix 2 Figure 1 – Key Plan
Figure 2 – Foundation Drainage System
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Drawing PG6598-1 – Test Hole Location Plan

Appendix 3 Typical Foundation Sleeve Installation

1.0 Introduction

Paterson Group (Paterson) was commissioned by Brigil Construction to carry out a geotechnical investigation for the proposed multi-storey residential buildings to be located at 1299 Richmond Road, in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objective of the current this geotechnical investigation was to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of test holes.

- ❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. It contains our findings and includes geotechnical recommendations pertaining to the design and construction of the subject development as they are understood at the time of writing this report.

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation.

2.0 Proposed Development

Based on available information, the proposed development will consist of two multi-storey residential and mixed-use buildings, with three to four underground levels. The development will also include associated asphaltic parking areas, access lanes and landscaped areas. It is further anticipated that the site will be fully municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current investigation was carried out between March 13 to March 15, 2023 and consisted of a total of five (5) boreholes, of which two (2) boreholes were advance to a maximum depths of 13.2 m, below the existing grade, where practical refusal to auguring was encountered; and three (3) of the boreholes were advanced to a maximum depths of 16.5 m, 15.1 m and 15.0 m, below the existing grade, cored and sampled approximately 2.0 to 3.0 m into the bedrock.

The boreholes were put down using a low clearance track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of personnel from Paterson's geotechnical division under the direction of a senior engineer. The testing procedure for boreholes consisted of augering to the required depths and at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Borehole samples were recovered from a 50 mm diameter split-spoon (SS) or the auger flights (AU). All soil samples were visually inspected and initially classified on site. The split-spoon and auger samples were placed in sealed plastic bags. All samples were transported to our laboratory for further examination and classification. The depths at which the split-spoon and auger samples were recovered from the test holes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

A Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split spoon sampler 300 mm into the soil after a 150 mm initial penetration using a 63.5 kg hammer falling from a height of 760 mm.

Rock samples were recovered from boreholes BH 1-23 and BH 3-23 using a core barrel and diamond drilling techniques. The bedrock samples were classified on site, placed in hard cardboard core boxes, and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

The recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are presented on the borehole logs. The recovery value is the length of the bedrock sample recovered over the length of the drilled section. The RQD value is the total length of intact rock pieces longer than 100 mm over the length of the core run. The values indicate the bedrock quality.

The subsurface conditions observed in the boreholes were recorded in detail in the field. The soil profiles are logged on the Soil Profile and Test Data Sheets in Appendix 1 of this report.

Groundwater

Groundwater monitoring wells were installed in all the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. All groundwater observations are noted on the Soil Profile and Test Data sheets presented in Appendix 1.

3.2 Field Survey

The test hole locations were selected by Paterson to provide general coverage of the proposed development taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson with respect to a geodetic datum. The location of the test holes and ground surface elevation at each test hole location are presented on Drawing PG6598-1 – Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were collected from the subject site during the investigation and were visually examined in our laboratory to review the results of the field logging. Moisture content testing was performed on all the recovered field samples. The test results are included in Appendix 1.

All samples will be stored in the laboratory for a period of one month after issuance of this report. The samples will then be discarded unless otherwise directed.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity and the pH of the sample. If available, the results are presented in Appendix 1 and are discussed further in Subsection 6.8.

4.0 Observations

4.1 Surface Conditions

The site is currently occupied by a one-storey slab on grade commercial building. The site is separated from the residential development to the west, high rise building to the north and west by Starflower Lane, and Richmond Road. Assaly Road separates further commercial development to the south along the west side and Richmond Road along the north side.

The site is almost fully covered by the building and associated asphalt parking lot. Richmond road is slightly elevated from the parking area while the surrounding roads slopes down to match the current elevation on site.

4.2 Subsurface Profile

Overburden

Fill

Generally, the subsurface profile was comprised of 50 to 130 mm asphaltic concrete, underlain by crushed stone with silty sand fill up to depths varying from 0.4 to 1 m at BH 1-23 to BH 4-23. Brown silty clay with some sand and gravel was observed underlying the crushed stone at BH 2-23 and BH 4-23 up to 1.6 m below the existing ground surface.

At BH 5-23, the subsurface profile comprised of 100 mm of topsoil, underlain by brown sandy silt with some clay, traces of gravel and organics.

Sandy Silt

Loose brown sandy silt with trace to some clay was generally observed underlying the fill at all the boreholes. The depth of the sandy silt layer varied from 2.2 to 3.1 m, with increasing clay content advancing deeper.

Silty Clay

A stiff to firm brown silty clay layer was encountered under the fill and silty. Seams of silty sand and layers with some sand seams were encountered. The brown silty clay was generally observed to be very stiff to stiff in consistency. The brown silty clay transitioned into a grey silty clay of stiff to firm consistency at depths of 4.7 to 5.5 m at BHs 2-23, 4-23 & 5-23.

Glacial Till

A compact to loose grey glacial till composed of sand in gravel in a silty clay soil matrix was found underlaying the silty clay deposit at depth of 9.3 to 12.0 m. The layer was noted to be highly saturated.

The glacial till became dense to very dense with depth at depths of 12.5 to 14.0 m. The silty clay matrix was noted to change to a silty sand matrix including gravel and cobbles. The layer was noted to be water bearing and highly permeable on site. No further testing was conducted to evaluate the hydraulic conductivity of the layer.

Bedrock

Based on the recovered core samples the bedrock was generally comprised of excellent quality grey quartz sandstone was encountered at the boreholes. Based on available geological mapping, the subject site is located in an area where the bedrock consists of the Ottawa Formation. The bedrock layer is expected to vary in depth from 12.0 m to 14.0 m below the existing grade.

4.3 Groundwater

Groundwater level readings were recorded on March 23, 2023, and are presented in Table 1 and on the Soil Profile and Test Data sheets in Appendix 1. It should be noted that surface water can become trapped within a backfilled borehole that can lead to higher than typical groundwater level observations. Additionally, groundwater levels are subject to seasonal fluctuations, therefore the groundwater levels could vary at the time of construction.

Long-term groundwater level can be estimated based on the observed color, moisture levels and consistency of the recovered soil samples. Based on these observations, the long-term groundwater is between 4.5 to 6.0 m.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Surface Elevation (m)	Groundwater Depth (m)	Groundwater Elevation (m)	Recording Date
BH1-23	69.32	3.79	65.53	March 23, 2023
BH2-23	69.06	5.41	63.65	March 23, 2023
BH3-23	69.30	5.84	63.46	March 23, 2023
BH4-23	69.18	5.63	63.55	March 23, 2023
BH5-23	69.55	3.08	66.47	March 23, 2023

Note:
- The ground surface elevations are referenced to a geodetic datum.
- * Borehole with groundwater monitoring well

5.0 Discussion

5.1 Geotechnical Assessment

Foundation Design Considerations

From a geotechnical perspective, the subject site is suitable for the proposed multi-storey buildings. It is expected that the proposed building will be founded on the quartz sandstone bedrock to a dense glacial till comprised of grey silty sand, some gravel, cobbles and boulders.

Alternately, to avoid excavating the entire building footprint to the bedrock level, footings could be placed over lean concrete infilled trenches. Near vertical, zero entry trench extending at least 300 mm beyond the footing face should be excavated to a clean bedrock surface approved by the geotechnical consultant. The trenches should be infilled by a minimum of 15 MPa lean concrete to the underside of the footing.

The above and other considerations are discussed in the following paragraphs.

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing significant organic materials, should be stripped from under any buildings and other settlement sensitive structures. The existing fill material, where free of organic materials, should be reviewed by the geotechnical consultant at the time of construction to determine if the existing fill can be left in place below paved areas and below the slab granular fill layers.

Fill Placement

Fill placed for grading beneath the building area should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Site-excavated soil can be placed as general landscaping fill where settlement is a minor concern of the ground surface. These materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm thick lifts and to a minimum density of 95% of the respective SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls due to the frost heave potential of the site excavated soils below settlement sensitive areas, such as concrete sidewalks and exterior concrete entrance areas.

The fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or hoe ramming.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity to the blasting operations should be completed prior to commencing site activities. The extent of the survey should be determined by the blasting consultant and should be sufficient to respond to any inquiries/claims related to the blasting operations.

As a general guideline, peak particle velocities (measured at the adjacent structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures. The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is also an experienced blasting consultant.

Excavation side slopes in sound bedrock can be carried out using almost vertical sidewalls. A minimum 1 m horizontal ledge should be left between the bottom of the overburden excavation and the top of the bedrock surface to provide an area to allow for potential sloughing or to provide a stable base for the overburden shoring system.

Horizontal rock anchors may be required at specific locations to prevent pop-outs of the bedrock, especially in areas where fractures in the bedrock are conducive to the failure of the bedrock surface.

The requirements for horizontal rock anchors and bedrock stabilization measures will be evaluated during the excavation program and determined by Paterson at the time of construction.

Vibration Considerations

Construction operations are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels should be incorporated in the construction operations to maintain, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed building.

Bearing Surface Preparation

The excavation is expected to be completed below the groundwater table. Where the bearing surface will consist of glacial till, measures to protect against heaving and ground disturbance should be put in place. Accordingly, it is recommended that the entirety of each building footprint be excavated to the underside of footing elevation, and then covered with a 150 mm thick mud slab to protect the glacial till from disturbance.

Furthermore, groundwater pumping using dry wells with sump pumps which are located centrally within the excavation will be required to control the influx of water during construction. Details can be provided once the groundwater influx is better assessed during the excavation process.

Lean Concrete In-Filled Trenches

Where footings are designed to be supported on bedrock, and the bedrock is not encountered at the design underside of footing elevation, consideration should be given to excavating zero-entry vertical trenches to expose the underlying bedrock surface and then backfilling with lean concrete (15 MPa 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The trench excavation should be at least 150 mm wider than all sides of the footing at the base of the excavation.

The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock. The excavation bottom should be relatively clean using the hydraulic shovel only (no worker entry). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

Footings placed on lean concrete filled trenches extending to the bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**. This is discussed further below.

5.3 Foundation Design

Foundation Option 1: Conventional Footings

Footings placed on an undisturbed, dense glacial till of silty sand matrix bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **400 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **700 kPa**. The provided bearing assumes a minimum depth of 10 m below existing grade. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS. Footings designed using the above-noted bearing resistance value at SLS will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, have been removed prior to the placement of concrete for footings.

Footings placed on the upper levels of the fractured bedrock a clean, surface sounded sandstone bedrock bearing surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,000 kPa**, incorporating a geotechnical resistance factor of 0.5. Alternately, footings can be placed over concrete in-filled (minimum 15 MPa) zero entry, near vertical trenches extended to a surface sounded bedrock bearing surface using the same bearing resistance values. The concrete in-filled trenches should extend a minimum 300 mm beyond the footing faces in all directions.

A factored bearing resistance value at ULS of **7,000 kPa,** incorporating a geotechnical resistance factor of 0.5, can be used for footings founded on clean, surface sounded bedrock and the bedrock is free of seams, fractures and voids within 1.5 m below the founding level. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level within the footing footprint(s). One drill hole should be completed per footing. The drill hole inspection should be completed by the geotechnical consultant.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and have no near surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Footings bearing on an acceptable bedrock bearing surface and designed using the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

Foundation Option 2: Raft Foundation

If the bearing resistance values are not sufficient for shallow foundation, raft foundation can be considered. The following parameters may be used for raft design and will apply for an undisturbed soil bearing surface. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings and approved by the geotechnical consultant.

Based on the following assumptions for a raft foundation, the proposed building can be designed with total and differential settlements of 25 and 15 mm, respectively.

For design purposes, it was assumed that the base of a raft foundation for a multi-storey building would be located at a depth of 9 to 11 m below existing ground surface and founded on glacial till for three (3) underground parking levels.

The amount of settlement of the raft slab will be dependent on the sustained raft contact pressure. The bearing resistance value at SLS (contact pressure) of **500 kPa** will be considered acceptable. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The contact pressure provided considers the stress relief associated with the soil removal required for the proposed building.

The factored bearing resistance value at ULS can be taken as **750 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be **20 MPa/m** for a contact pressure of **750 kPa**. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium.

Bedrock/Soil Transition

If the raft slab is constructed in the areas underlain by bedrock, it is recommended that a minimum 500 mm thick layer (native soil and or crushed stone layer) be present between the raft slab and the bedrock surface to reduce the risks of bending stresses developing in the concrete slab. The rock should be broken down a minimum of 500 mm and backfill using Granular B Type II crushed stone compacted to 98% of the material's SPMDD. The bending stress could lead to cracking of the concrete slab. This requirement could be waived in areas where the bedrock surface is relatively flat within the footprint of the building. This recommendation does not refer to potential concrete shrinkage cracking which should be controlled in the usual manner.

Foundation Option 3: End Bearing Piled Foundation

It is anticipated that the structure might require to be constructed over concrete filled steel pipe piles driven to refusal on the bedrock surface where the depth of the bedrock is located well below the proposed underside of footing for the development.

The bedrock surface is estimated to be located at a depth ranging from 8.2 to 16.7 m in depth throughout the site while the foundation for the development is anticipated at a depth of 9 to 11 below the existing ground surface. The piles will need to be driven through a dense layer of glacial till below 9 to 11 m below existing ground surface.

For deep foundations, concrete-filled steel pipe piles are generally utilized in the Ottawa area. Applicable pile resistance at SLS values and factored pile resistance at ULS values are given in Table 2. A resistance factor of 0.4 has been incorporated into the factored ULS values. Note that these are all geotechnical axial resistance values.

The geotechnical pile resistance values were estimated using the Hiley dynamic formula, to be confirmed during pile installation with a program of dynamic monitoring. For this project, the dynamic monitoring of two (2) to four (4) piles would be recommended. This is considered to be the minimum monitoring program, as the piles under shear walls may be required to be driven using the maximum recommended driving energy to achieve the greatest factored resistance at ULS values. Re-striking of all piles at least once will also be required after at least 48 hours have elapsed since initial driving.

Table 2 - Pile Foundation Design Data

Pile Outside Diameter (mm)	Pile Wall Thickness (mm)	Geotechnical Axial Resistance		Final Set (blows/ 12 mm)	Transferred Hammer Energy (kJ)
		SLS (kN)	Factored at ULS (kN)		
245	9	1,000	1,250	6	27
245	11	1,150	1,450	6	31
245	13	1,300	1,600	6	35

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the driving of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously driven piles, are checked as part of the field review of the pile driving operations.

Prior to the commencement of production pile driving, a limited number of indicator piles should be installed across the site. It is recommended that each indicator pile be dynamically load tested to evaluate pile stresses, hammer efficiency, pile load transfer, and end-of-driving criteria for end-bearing in the bedrock.

Foundation Option 4: Drilled Shafts and Caissons

End bearing cast-in-place caissons can be used where supplemental axial resistance is required for structural design for the proposed building. The caisson should be installed by driving a temporary steel casing and excavating the soil through the casing. A minimum of 35 MPa concrete should be used to in fill the caissons. The caissons are to be structurally reinforced over their entire length.

Two conditions for drilled shafts are applicable for this site. The first alternative is a caisson installed on the sound bedrock augering through the weathered bedrock (end bearing).

The compressive resistance for such piles is directly related to the compressive strength of the bedrock. It is recommended that the entire capacity be derived from the end bearing capacity.

The second alternative is a concrete caisson socketed into bedrock. The axial capacity is increased by the shear capacity of the concrete/rock interface. Furthermore, the tensile resistance of the caisson is increased by the rock capacity. It should be noted that the rock socket should be reinforced.

Table 3 below presents the estimated capacity for different typical caisson sizes for a rock bearing caisson and rock socketed caisson extending 3 m into sound bedrock.

Table 3 - Caisson Pile Capacities

Caisson Diameter		Axial Capacity (kN)	
inch	mm	End Rock Bearing	Rock Socket
36	900	8,000	11,000
42	1,000	10,000	13,000
48	1,200	14,500	17,500
54	1,375	18,500	21,500
60	1,500	22,500	25,500

Notes:
- 3 m rock socket in bedrock
- Reinforced caisson and rock socket when applicable
- 0.4 geotechnical factor applied to the shaft capacity

The minimum recommended centre-to-centre pile spacing is 3 times the pile diameter. The closer the piles are spaced, however, the more potential that the installation of subsequent piles in a group could have influence on piles in the group that have already been driven. These effects, primarily consisting of uplift of previously installed piles.

Option 5: Diaphragm/ Secant Wall Foundation

The multi-storey building will be founded on reinforced diaphragm walls or secant piles that are socketed within the bedrock along the perimeter of the parking garage. Furthermore, the interior portion of the building (shear walls, elevator shafts, stairwells and other portions of the structure selected by the structural engineer) will be founded on steel reinforced barrette walls socketed in the bedrock.

Table 4 – Bearing Resistance Values for Diaphragm Walls Socketed in Bedrock

Socketed Length (m)	Width of Diaphragm Wall (mm)	Bearing Resistance Value at SLS (kPa)	Factored Bearing Resistance Value at ULS (kPa)
1	600	2,000	2,500
	750	2,000	2,500
2	600	2,500	3,250
	750	2,500	3,250
3	600	3,000	3,500
	750	3,000	3,500

The diaphragm wall can also be designed using the bedrock shear strength if drilled and socketed within the bedrock. This shear strength value will be **500** to **800 kPa** and is reduced due to the fractured nature of the upper levels of the grey limestone layer.

Settlement

The total and differential settlement will be dependent on characteristics of the proposed buildings. For design purposes, the total and differential settlements are estimated to be 25 to 20 mm, respectively.

A building founded on deep foundations or shallow foundations bearing on an acceptable bedrock bearing surface and designed for the bearing resistance value provided herein will be subjected to negligible potential postconstruction total and differential settlements.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels.

Above the groundwater level, adequate lateral support is provided to the in-situ bearing medium soils when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil.

Adequate lateral support is provided to bedrock bearing medium when a plane extending down and out from the bottom edges of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

A heavily fractured, weathered bedrock and/or overburden bearing medium will require a lateral support zone of 1H:1V (or flatter).

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class C for the foundations anticipated at the subject site. The soils underlying the subject site are not susceptible to liquefaction.

A higher seismic site class, such as Class A or B, is available for design provided footings are extended within 3 from the bedrock surface and a site-specific seismic shear wave velocity test is conducted by the geotechnical consultant. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements. The soils underlying the subject site are not considered to be susceptible to liquefaction.

5.5 Basement Slab

The basement areas for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction.

All backfill material within the footprint of the proposed building(s) should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building(s) should be placed in a maximum of 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions encountered at the time of the current and previous fieldwork, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structures. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3. The applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3, where applicable. A hydrostatic pressure should be added to the total static earth pressure when using the effective unit weight.

Lateral Earth Pressures

The static horizontal earth pressure (p_o) can be calculated using a triangular earth pressure distribution equal to $K_o \cdot \gamma \cdot H$ where:

K_o = at-rest earth pressure coefficient of the applicable retained soil, 0.5
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

An additional pressure having a magnitude equal to $K_o \cdot q$ and acting on the entire height of the wall should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be used in conjunction with the seismic loading case.

Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}). The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

a_c = $(1.45-a_{max}/g)a_{max}$
γ = unit weight of fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)
g = gravity, 9.81 m/s^2

The peak ground acceleration, (a_{max}), for the Ottawa area is 0.32g according to OBC 2012. Note that the vertical seismic coefficient is assumed to be zero.

The earth force component (P_o) under seismic conditions can be calculated using $P_o = 0.5\ K_o\ \gamma\ H^2$, where $K_o = 0.5$ for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of the wall, where:

$$h = \{P_o \cdot (H/3) + \Delta P_{AE} \cdot (0.6 \cdot H)\}/P_{AE}$$

The earth forces calculated are unfactored. For the ULS case, the earth loads should be factored as live loads, as per OBC 2012.

5.7 Pavement Structure

For design purposes, it is recommended that the rigid pavement structure for the underground parking level consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 5 on the next page. The flexible pavement structure presented in Table 6 and Table 7 should be used for driveways and car only parking areas and at grade access lanes and heavy loading parking areas.

Table 5 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
150	**Exposure Class C2 - 32 MPa Concrete** (5 to 8% Air Entrainment)
300	**BASE** – OPSS Granular A Crushed Stone
SUBGRADE – Hard to firm silty clay, or OPSS Granular A or OPSS Granular B Type II granular fill material placed over in situ soil.	

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example; a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m).

The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 6 – Recommended Pavement Structure – Light Vehicle Parking	
Thickness (mm)	**Material Description**
40	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
50	**Binder Course –** Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
300	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

Table 7 – Recommended Pavement Structure –Local Roadways, Access Lanes and Heavy Vehicle Parking	
Thickness (mm)	**Material Description**
40	**Wear Course –** Superpave 12.5-FC2 Asphaltic Concrete
50	**Upper Binder Course –** Superpave 19.0 Asphaltic Concrete
150	**BASE –** OPSS Granular A Crushed Stone
400	**SUBBASE –** OPSS Granular B Type II Crushed Stone
SUBGRADE - Either fill, in situ silty clay or sand/crushed stone material placed over in situ soil	

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type I or Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable compaction equipment.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

Where silty clay is encountered at subgrade level, consideration should be given to installing subdrains during the pavement construction. These drains should be constructed according to City of Ottawa specifications. The drains should be connected to a positive outlet. The subgrade surface should be crowned to promote water flow to the drainage lines. The subdrains will help drain the pavement structure, especially in early Spring when the subgrade is saturated and weaker and, therefore, more susceptible to permanent deformation.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

It is understood that the proposed building will accommodate four underground parking levels and will be extended to a bedrock. It is recommended that a dual-shoring (shoring/foundation) system such as a diaphragm wall, slurry wall or secant pile wall be considered for the building. As such, the shoring system is expected to reduce the potential for groundwater infiltration into the underground parking level structures. Based on Paterson's experience in the Ottawa area with slurry wall shoring systems, minor water infiltration may be observed long-term within the underground parking level structures. Therefore, to mitigate long-term water ingress into the underground parking levels, it is recommended to install an adequate foundation drainage system and negative side waterproofing. Refer to Figure 2 – Foundation Drainage System, for specific details of the foundation drainage recommendations in Appendix 1.

Foundation Drainage System

A foundation drainage system is recommended for the underground parking levels to prevent water from seeping through the slurry walls.

Furthermore, to manage and control the groundwater infiltration to the building's storm sump pump(s) over the long-term, the following foundation drainage system is recommended to be installed on the interior side of the slurry walls or secant piles using the below methodology:

- Any discontinuities, leaks or imperfection in the foundation wall should be repaired and covered with a negative side waterproofing. While total application might not be required provision for installing a membrane such as Hygrothane by Elastochem or CN2000 series by Kelso should be considered.

- It is recommended that a composite foundation drainage membrane, such as 6000 series membrane by DeltaDrain, G100N by MiraDrain or equivalent approved other, be placed on the interior slurry wall face. The composite foundation drainage board should extend from finished grade to the footing level with the geotextile layer facing the prepped substrate surface (foundation wall). It is highly recommended that the drainage board be installed horizontally, in a shingle-fashion, with a minimum overlapping of 150 mm between the sheets to minimize seams throughout the system. The drainage should drain down to the lower slab and subfloor drainage system. Sleeves or continuous drainage will be required between the different parking levels.

- It is recommended that a 100 mm thick 35 MPa shotcrete liner be installed overlying the recommended drainage system to further prevent seepage into the underground parking levels. The shotcrete layer will provide an aesthetic finish to the interior underground parking levels and adequately seal the proposed drainage system.

- Furthermore, it is recommended that 150 mm diameter sleeves placed at 3 m centres be cast in the proposed shotcrete liner to allow water to flow to an interior underfloor drainage system. The underfloor drainage system should direct water to the storm sump pit(s) within the lower basement area.

Interior Underfloor Drainage System

The interior underfloor drainage system will be required to control water infiltration below the lowest underground parking level slab and to redirect groundwater from the buildings foundation drainage system to the buildings sump pit(s). The interior underfloor drainage pipes should consist of a 150 mm diameter corrugated perforated PVC pipes surrounded by a minimum of 150 mm of 19 mm clear crushed stone. It is recommended that the interior underfloor drainage system be mechanically connected to the 150 mm drainage sleeves and gravity connected to the underfloor drainage system which in turn is connected to the buildings storm sump pit(s).

The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Subfloor Water Infiltration

Due to the variability in the limestone, it is expected that water might infiltrated through seems and cracks. Paterson should review the water infiltration. It is recommended to carry a minimum 75 mm mubslab and horizontal membrane to act has hydraulic barrier on top of the bedrock.

Elevator Pit Waterproofing

The horizontally applied Colphene BSW H waterproofing membrane (or approved other) should be placed on an adequately prepared mud slab and extend vertically within the inside of the temporary forms of the elevator raft slab. Once the concrete raft slab and elevator shaft sidewalls are poured in place, it is recommended that a waterproofing membrane, such as Colphene Torch'n Stick (or approved other) should be applied to the exterior of the elevator pit sidewalls. The Colphene Torch'n Stick waterproofing membrane should extend over the vertical portion of the previously applied Colphene BSW H waterproofing membrane installed on the concrete raft slab in accordance with the manufacturer's specifications. As a secondary defense, a continuous PVC waterstop such as Southern waterstop 14RCB or equivalent should be installed within the concrete raft slab below the elevator pit sidewalls.

A protection board should be placed over the waterproofing membrane to protect the waterproofing membrane from damage during backfilling operations. The area between the elevator pit and bedrock excavation face should be in-filled with lean concrete, OPSS Granular B Type 2 or Granular A crushed stone. Refer to Figure 3 – Waterproofing System for Elevator, for specific details of the elevator waterproofing in Appendix 2.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free draining non frost susceptible granular materials.

The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless used in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

Sidewalks and Walkways

Backfill material below sidewalk and walkway subgrade areas or other settlement sensitive structures which are not adjacent to the buildings should consist of free-draining, non-frost susceptible material. This material should be placed in maximum 300 mm thick loose lifts and compacted to at least 98% of its SPMDD under dry and above freezing conditions.

Adverse Effects from Dewatering on Adjacent Structures

Since the excavation is expected to extend in a water bearing sandy till, construction dewatering is not recommended at depths greater than 6 to 7 m. The excavation should consider the use of a nearly waterproofed shoring system. It is estimated that groundwater lowering will affect the residential neighborhood to the north if more than 400,000 L/day is pumped during the excavation process. The use of a secant or diaphragm wall socketed a minimum of 1.5 m in bedrock will lower the groundwater infiltration into the excavation to controllable and acceptable levels.

The temporary dewatering of the bedrock during the excavation and construction stage will not be susceptible to significant consolidation since the material.

Implementation of dual use shoring system recommended above is expected to limit the drawdown of the local groundwater table over the long term and in a limited area. Therefore, in our opinion, no adverse effects to nearby structures and infrastructure are expected over the long term if a watertight shoring is used for construction.

6.2 Protection of Footings Against Frost Action

Perimeter footings, of heated structures are required to be insulated against the deleterious effect of frost action. A minimum of 1.5 m thick soil cover (or equivalent) should be provided in this regard. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

However, foundations which are founded directly on clean, surface-sounded bedrock with no cracks or fissures, and which is approved by Paterson at the time of construction, is not considered frost susceptible and does not require soil cover.

6.3 Excavation Side Slopes

Temporary Side Slopes

The temporary excavation side slopes anticipated should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain a safe working distance from the excavation sides.

Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

A trench box is recommended to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain open for extended periods of time.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer.

Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural design prior to implementation.

The temporary system could consist of diaphragm walls or secant pile walls. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 8 - Soil Parameters

Parameters	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible while the at-rest pressure should be calculated if no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component.

For design purposes, the minimum factor of safety of 1.5 should be calculated.

Diaphragm Wall System

For design purposes, the earth pressure acting on a slurry wall shoring system can be estimated using a trapezoidal earth pressure envelope with a maximum pressure of 0.3·y·H for strutted or anchored shoring. The earth pressure will be zero for the top and bottom of the excavation and will increase to the maximum which occurs at 0.25·H from both the bottom and the top of the excavation. The earth pressure distribution can also be estimated using an earth pressure coefficient and a quasi-hydrostatic distribution.

The active earth pressure coefficient should be used where wall movements are permissible while the at-rest pressure coefficient should be used if no movement is permissible.

The total unit weight should be used above the groundwater level while the submerged unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the submerged unit weights are used for earth pressure calculations (i.e. below the groundwater level).

The excavation of a diaphragm wall should be carried out in sections or panels with the excavation filled with a bentonite-rich slurry to provide adequate support for the trench walls. Once the excavation is complete, reinforcing may be installed, if required, and concrete can be poured from the bottom of the excavation using tremie methods.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications & Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa.

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer or water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD.

To reduce long-term lowering of the groundwater level at this site, clay seals should be provided in the service trenches.

The seals should be at least 1.5 m long and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, sub bedding and cover material. The barriers should consist of relatively dry and compatible brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the material's SPMDD. The clay seals should be placed at the site boundaries and at strategic locations at no more than 60 m intervals in the service trenches.

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be low through the sides of the excavation and controllable using open sumps if a watertight shoring system is used. Pumping from open sumps should be sufficient to control the groundwater influx through the sides of shallow excavations. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required for this project if more than 400,000 L/day of ground and/or surface water is to be pumped during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

For typical ground or surface water volumes, being pumped during the construction phase, between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16.

If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project.

The subsurface conditions mostly consist of frost susceptible materials. In the presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters and tarpaulins or other suitable means.

The base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precaution must be taken where excavations are carried out in close proximity of existing structures, which may be adversely affected due to the freezing conditions.

6.7 Corrosion Potential and Sulphate

The results of analytical testing show that the sulphate content is less than 0.1%. This result is indicative that Type 10 Portland cement (normal cement) would be appropriate for this site. The chloride content and the pH of the samples indicate that they are not significant factors in creating a corrosive environment for exposed ferrous metals at this site, whereas the resistivity is indicative of a severe to a very aggressive environment.

7.0 Recommendations

For the foundation design data provided herein to be applicable that a material testing and observation services program is required to be completed. The following aspects be performed by the geotechnical consultant:

- ❑ Review of the site master grading plan, once available.
- ❑ Review of the excavation and shoring plan (can be prepared by Paterson)
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ Observation of the placement of the foundation insulation, if applicable.
- ❑ Observe and review the installation of the drainage and waterproofing system.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling and follow-up field density tests to determine the level of compaction achieved.
- ❑ Field density tests to determine the level of compaction achieved.
- ❑ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming the construction has been conducted in general accordance with the recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request that we be permitted to review the grading plan once available and our recommendations when the drawings and specifications are complete.

A geotechnical investigation of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. The extent of the limited area depends on the soil, bedrock and groundwater conditions, as well the history of the site reflecting natural, construction, and other activities. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The recommendations provided in this report are intended for the use of design professionals associated with this project. Contractors bidding on or undertaking the work should examine the factual information contained in this report and the site conditions, satisfy themselves as to the adequacy of the information provided for construction purposes, supplement the factual information if required, and develop their own interpretation of the factual information based on both their and their subcontractors construction methods, equipment capabilities and schedules.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Brigil Construction or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Pratheep Thirumoolan, M.Eng. Joey R. Villeneuve, M.A.Sc., P.Eng, ing.

Report Distribution:

- ❑ Brigil Construction
- ❑ Paterson Group Inc

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 13, 2023

FILE NO. PG6598

HOLE NO. BH 1-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.32
Asphaltic concrete 0.13		AU	1				
FILL: Grey crushed stone, some sand 0.69		SS	2	100	9	1	68.32
Loose, brown **SANDY SILT,** trace to some clay		SS	3	100	5	2	67.32
2.97		SS	4	100	5	3	66.32
Very stiff to stiff, brown **SILTY CLAY,** trace sand seams		SS	5	100	3	4	65.32
		SS	6	100	5	5	64.32
		SS	7	100	2	6	63.32
6.02		SS	8	100	1		
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	9	25	5	7	62.32
		SS	10	100	3		
		SS	11	17	4	8	61.32
		G	12			9	60.32
		SS	13	33	3		
		SS	14		3	10	59.32
11.13		SS	15	100	4	11	58.32
GLACIAL TILL: Dense to very dense, brown silty sand to sandy silt, some clay, gravel, cobbles and boulders		SS	16	84	40		
		S	17	67	50+	12	57.32
13.26		SS	18	50	50+	13	56.32
End of Borehole							
Practical refusal to augering at 13.26m depth.							
(GWL @ 3.79m - March 23, 2023)							

Pen. Resist. Blows/0.3m
● **50 mm Dia. Cone**
○ **Water Content %**

20 40 60 80

Monitoring Well Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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9 Auriga Drive, Ottawa, Ontario K2E 7T9

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

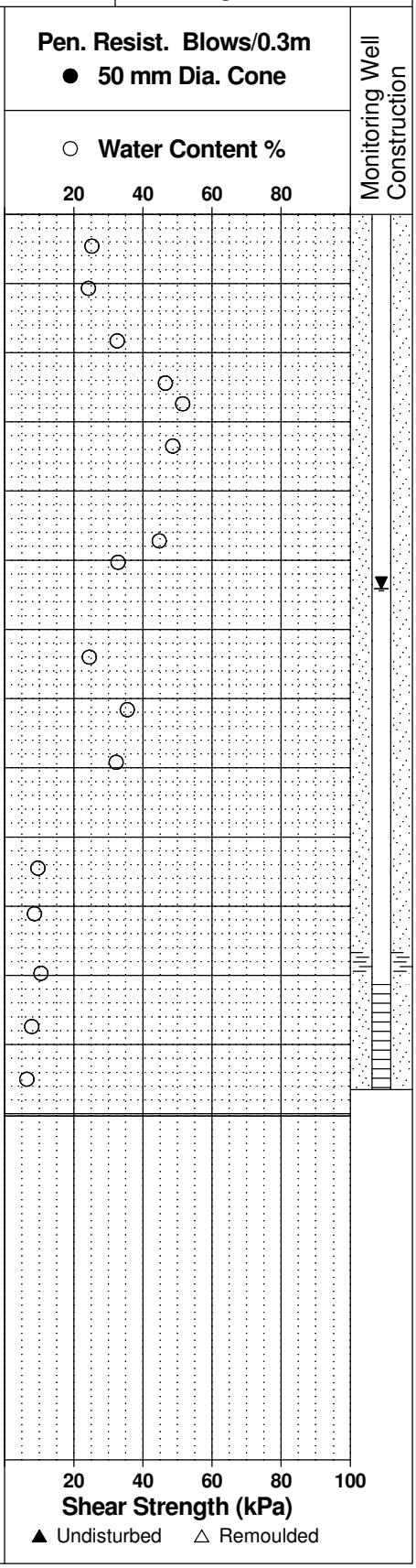
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 13, 2023

FILE NO. PG6598

HOLE NO. BH 2-23

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	69.06
Asphaltic concrete	0.08		AU	1				
FILL: Crushed stone, some silty sand	0.23		SS	2	100	5	1	68.06
FILL: Brown silty clay, some sand	0.76		SS	3	100	6	2	67.06
Loose, brown **SANDY SILT,** trace to some clay	2.21		SS	4	100	3		
Very stiff to stiff, brown **SILTY CLAY,** some sand seams			SS	5	100	2	3	66.06
- stiff to firm and grey by 4.7m depth			SS	6	100	2	4	65.06
			SS	7	100	1	5	64.06
			SS	8	100	1	6	63.06
			SS	9	100	1	7	62.06
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders	8.23		SS	10	0	7	8	61.06
	9.30		SS	11	58	24	9	60.06
GLACIAL TILL: Compact to dense, grey silty sand to sandy silt with gravel, cobbles and boulders			SS	12	88	50+	10	59.06
			SS	13	29	50+	11	58.06
			SS	14		50+	12	57.06
			SS	15	71	50+		
	13.03		SS	16	0	50+	13	56.06
End of Borehole								
Practical refusal to augering at 13.03m depth.								
(GWL @ 5.41m - March 23, 2023)								



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

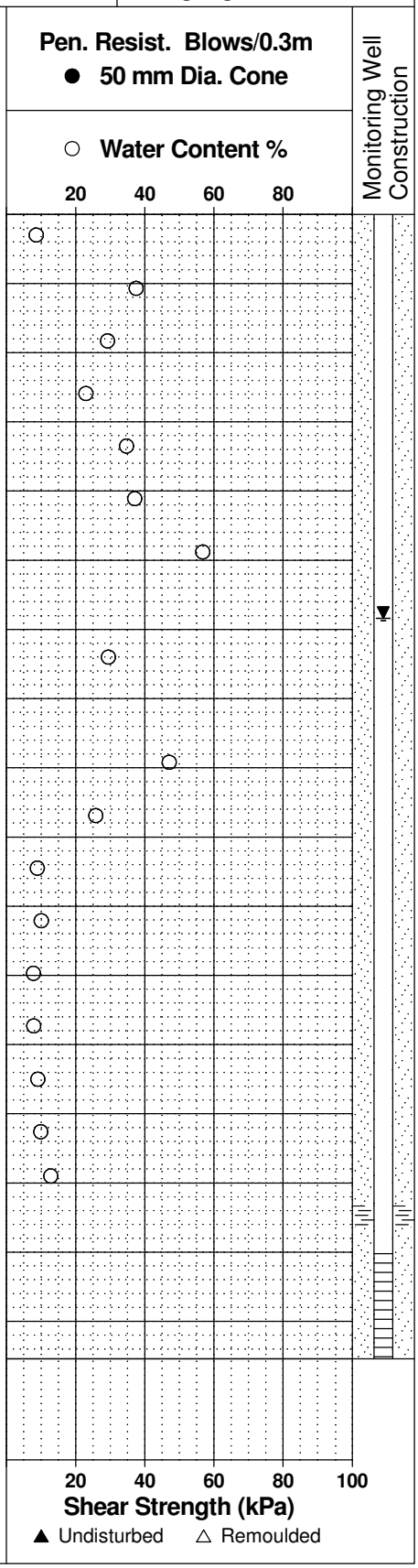
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 14, 2023

FILE NO. PG6598

HOLE NO. BH 3-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.30
Asphaltic concrete 0.05		AU	1				
FILL: Crushed stone 0.56		SS	2	100	6	1	68.30
Loose, brown **SANDY SILT,** trace to some clay		SS	3	92	4	2	67.30
- some sand seams by 2.3m depth 2.74		SS	4	100	7		
		SS	5	100	4	3	66.30
Very stiff, brown **SILTY CLAY** some sand seams		SS	6	100	2	4	65.30
- silty sand layer from 5 to 5.2m depth		SS	7	100	5	5	64.30
		SS	8	100	1	6	63.30
- trace gravel by 8.1m depth		SS	9	100	1	7	62.30
8.99		SS	10	8	2	8	61.30
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders		SS	11	8	3	9	60.30
10.21		SS	12	21	16	10	59.30
		SS	13	100	50+	11	58.30
GLACIAL TILL: Compact to very dense, grey silty sand, some gravel, cobbles and boulders, trace clay		SS	14	67	50+	12	57.30
		SS	15	83	50+	13	56.30
		SS	16	100	50+		
14.05		SS	17	86	50+	14	55.30
		RC	1	100	100	15	54.30
BEDROCK: Excellent quality, grey quartz sandstone		RC	2	100	98	16	53.30
16.54							
End of Borehole							
(GWL @ 5.84m - March 23, 2023)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 15, 2023

FILE NO. PG6598

HOLE NO. BH 4-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.18
Asphaltic concrete 0.08		AU	1				
FILL: Crushed stone with silty sand 0.38		SS	2	100	6	1	68.18
FILL: Brown silty clay, trace sand and gravel 0.69		SS	3	100	6	2	67.18
Loose, brown **SANDY SILT,** trace to some clay 3.05		SS	4	100	4	3	66.18
Very stiff, brown **SILTY CLAY,** trace to some sand seams		SS	5	100	6	4	65.18
5.49						5	64.18
Stiff to firm, grey **SILTY CLAY**		SS	6	100	1	6	63.18
		SS	7	100	1	7	62.18
8.53						8	61.18
GLACIAL TILL: Grey silty clay with sand and gravel 9.45		SS	8	100	50+	9	60.18
GLACIAL TILL: Very dense, grey silty sand to sandy silt with gravel, cobbles and boulders		SS	9	58	50+	10	59.18
		SS	10	58	50+	11	58.18
12.44		SS	11	90	50+	12	57.18
		RC	1	100	100		
BEDROCK: Excellent quality, grey quartz sandstone		RC	2	100	95	13	56.18
		RC	3	100	95	14	55.18
15.11						15	54.18
End of Borehole							
(GWL @ 5.63m - March 23, 2023)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %

20 40 60 80

Monitoring Well Construction

20 40 60 80 100
Shear Strength (kPa)
▲ Undisturbed △ Remoulded

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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Mixed-Use High-Rise Development
1299 Richmond Road, Ottawa, Ontario

DATUM Geodetic

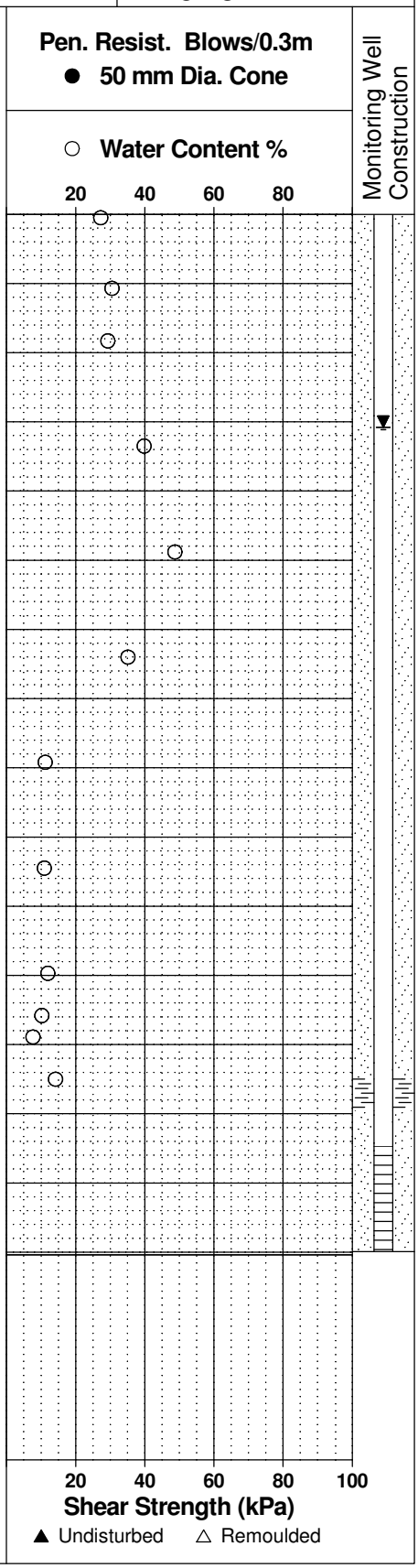
REMARKS

BORINGS BY CME-55 Low Clearance Drill **DATE** March 15, 2023

FILE NO. PG6598

HOLE NO. BH 5-23

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	69.55
TOPSOIL 0.10		AU	1				
FILL: Brown sandy silt, some clay, trace gravel and organics 0.60		SS	2	83	11	1	68.55
Compact to loose, brown **SANDY SILT,** trace to some clay		SS	3	92	7	2	67.55
- increasing clay content with depth 2.59							
Hard to very stiff, brown **SILTY CLAY,** some sand		SS	4	100	5	3	66.55
		SS	5	100	3	4	65.55
- stiff and grey by 5.5m depth 6.10						5	64.55
		SS	6	75	3	6	63.55
GLACIAL TILL: Grey silty clay with sand and gravel, occasional cobbles		SS	7	58	12	7	62.55
		SS	8	83	4	8	61.55
		SS	9	58	2	9	60.55
11.99		SS	10	100	1	10	59.55
GLACIAL TILL: Dense, grey silty sand to sandy silt with gravel, cobbles and boulders, some to trace clay 12.78		SS	11	83	45	11	58.55
		RC	1	100	100	12	57.55
BEDROCK: Excellent quality, grey quartz sandstone		RC	2	100	98	13	56.55
15.04						14	55.55
End of Borehole						15	54.55
(GWL @ 3.08m - March 23, 2023)							



SYMBOLS AND TERMS

SOIL DESCRIPTION

Behavioural properties, such as structure and strength, take precedence over particle gradation in describing soils. Terminology describing soil structure are as follows:

Desiccated	-	having visible signs of weathering by oxidation of clay minerals, shrinkage cracks, etc.
Fissured	-	having cracks, and hence a blocky structure.
Varved	-	composed of regular alternating layers of silt and clay.
Stratified	-	composed of alternating layers of different soil types, e.g. silt and sand or silt and clay.
Well-Graded	-	Having wide range in grain sizes and substantial amounts of all intermediate particle sizes (see Grain Size Distribution).
Uniformly-Graded	-	Predominantly of one grain size (see Grain Size Distribution).

The standard terminology to describe the strength of cohesionless soils is the relative density, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm.

Relative Density	'N' Value	Relative Density %
Very Loose	<4	<15
Loose	4-10	15-35
Compact	10-30	35-65
Dense	30-50	65-85
Very Dense	>50	>85

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

Consistency	Undrained Shear Strength (kPa)	'N' Value
Very Soft	<12	<2
Soft	12-25	2-4
Firm	25-50	4-8
Stiff	50-100	8-15
Very Stiff	100-200	15-30
Hard	>200	>30

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

ROCK DESCRIPTION

The structural description of the bedrock mass is based on the Rock Quality Designation (RQD).

The RQD classification is based on a modified core recovery percentage in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be a result of closely-spaced discontinuities (resulting from shearing, jointing, faulting, or weathering) in the rock mass and are not counted. RQD is ideally determined from NXL size core. However, it can be used on smaller core sizes, such as BX, if the bulk of the fractures caused by drilling stresses (called “mechanical breaks”) are easily distinguishable from the normal in situ fractures.

RQD %	ROCK QUALITY
90-100	Excellent, intact, very sound
75-90	Good, massive, moderately jointed or sound
50-75	Fair, blocky and seamy, fractured
25-50	Poor, shattered and very seamy or blocky, severely fractured
0-25	Very poor, crushed, very severely fractured

SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D30)^2$ / (D10 x D60)
Cu	-	Uniformity coefficient = D60 / D10

Cc and Cu are used to assess the grading of sands and gravels:
Well-graded gravels have: $1 < Cc < 3$ and $Cu > 4$
Well-graded sands have: $1 < Cc < 3$ and $Cu > 6$
Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.
Cc and Cu are not applicable for the description of soils with more than 10% silt and clay
(more than 10% finer than 0.075 mm or the #200 sieve)

CONSOLIDATION TEST

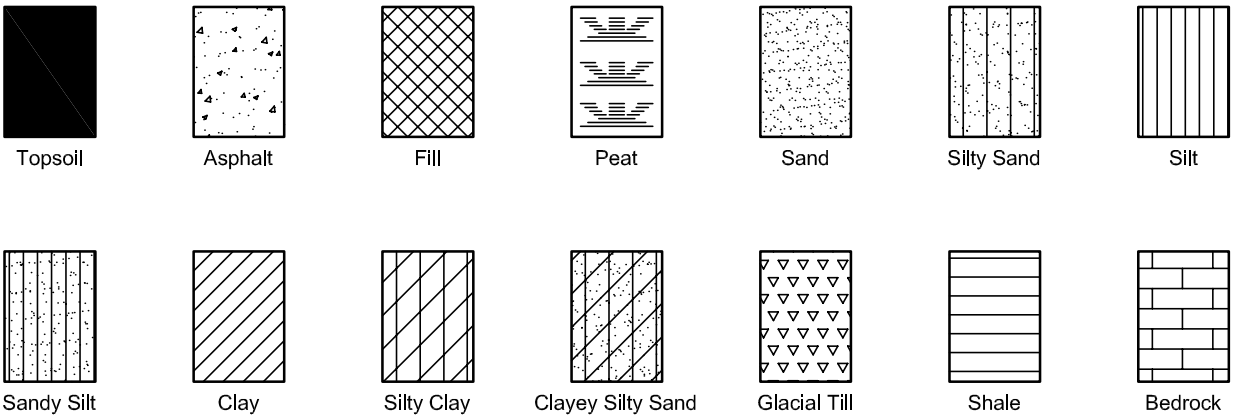
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
Wo	-	Initial water content (at start of consolidation test)

PERMEABILITY TEST

k	-	Coefficient of permeability or hydraulic conductivity is a measure of the ability of water to flow through the sample. The value of k is measured at a specified unit weight for (remoulded) cohesionless soil samples, because its value will vary with the unit weight or density of the sample during the test.

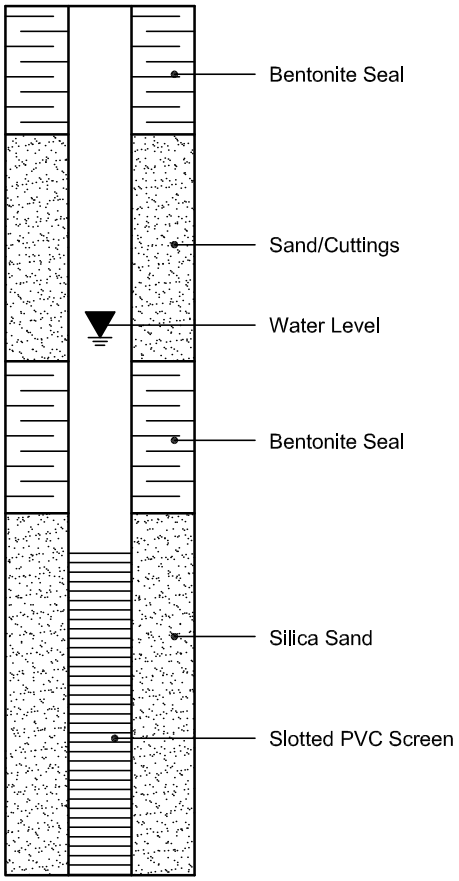
SYMBOLS AND TERMS (continued)

STRATA PLOT

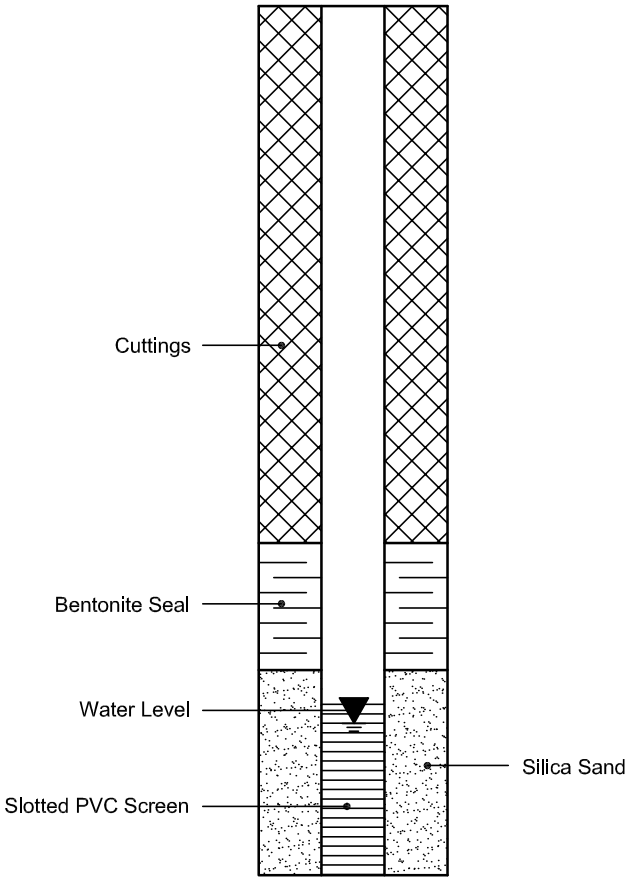


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 – KEY PLAN

FIGURE 2 – FOUNDATION DRAINAGE SYSTEM

FIGURE 3 – ELEVATOR PIT WATERPROOFING

DRAWING PG6598-1 - TEST HOLE LOCATION PLAN

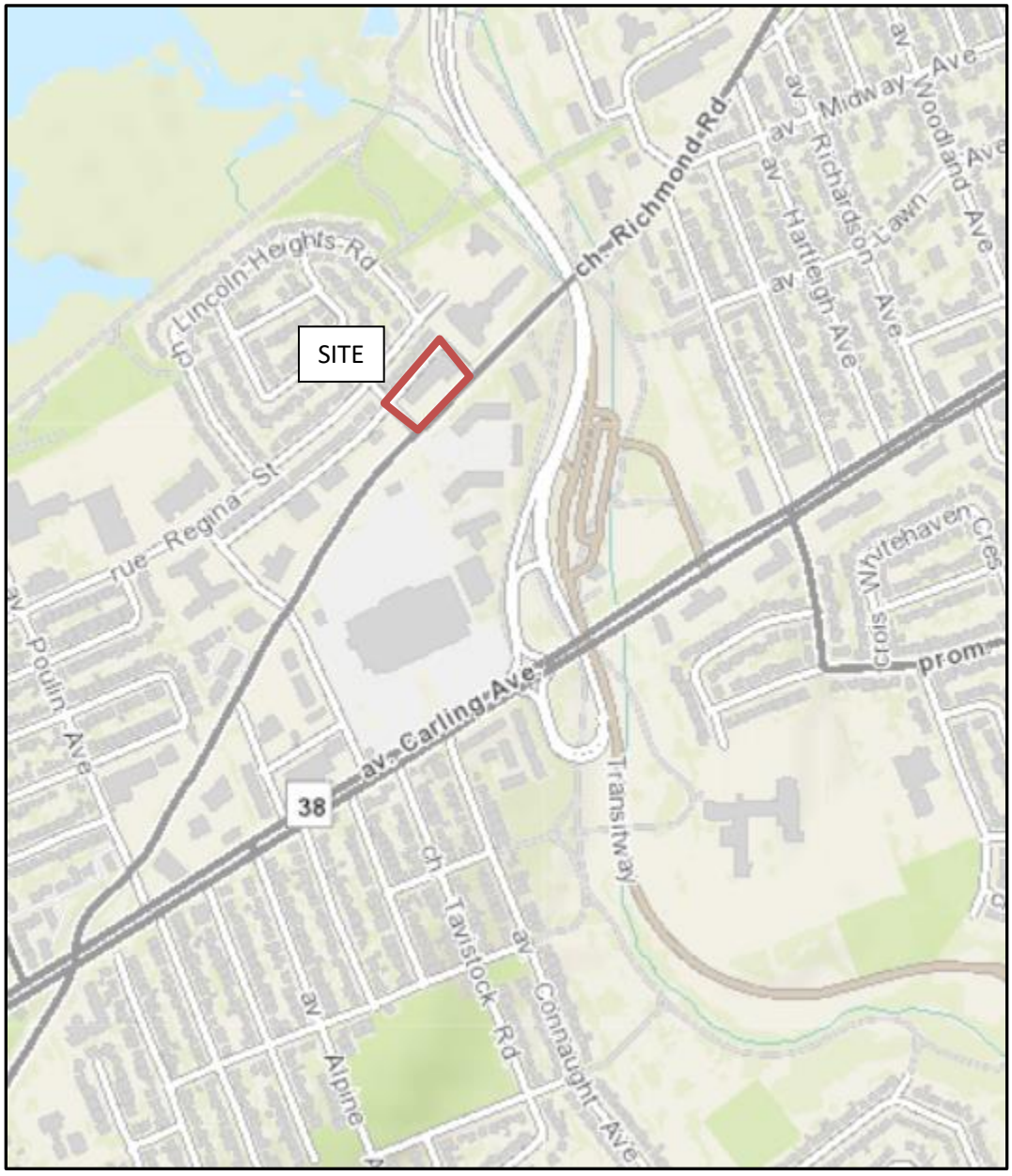
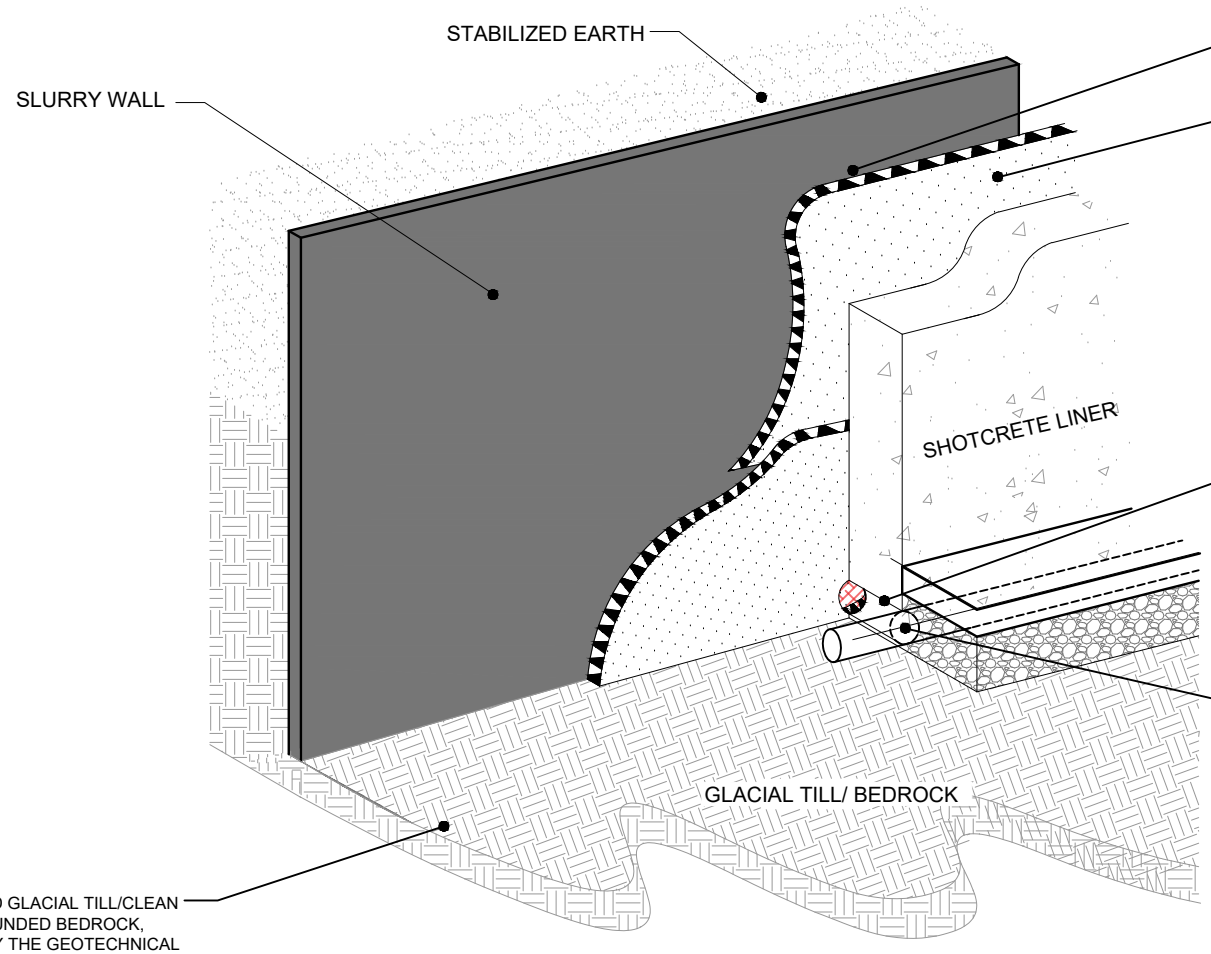


FIGURE 1

KEY PLAN

PATERSON GROUP



GEOTEXTILE LINED SIDE OF THE DRAINAGE BOARD FACING THE INTERIOR SLURRY WALL SURFACE

COMPOSITE FOUNDATION DRAINAGE MEMBRANE 6000 SERIES MEMBRANE BY DELTA DRAIN, G100N BY MIRADRAIN OR EQUIVALENT. INSTALL IN HORIZONTAL LIFTS WITH MINIMUM 150mm HORIZONTAL AND VERTICAL OVERLAP IN SHINGLE FASHION WITH GEOTEXTILE FACING AWAY FROM THE APPLICATOR. MECHANICALLY SECURE VERTICAL AND HORIZONTAL JOINTS WITH 3M TAPE OR APPROVED EQUIVALENT.

150mm DIAMETER SOLID PVC SLEEVE PLACED THROUGH THE BASE OF THE EXTERIOR FOUNDATION WALL AT APPROXIMATELY 3.0m SPACING AROUND THE EXTERIOR PERIMETER OF THE STRUCTURE.

THE SLEEVES SHOULD BE MECHANICALLY CONNECTED TO THE COMPOSITE FOUNDATION DRAINAGE MEMBRANE AND THE 150mm DIAMETER INTERIOR SUBFLOOR PERIMETER DRAINAGE SYSTEM GRAVITY CONNECTED TO THE SUMP PIT.

IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE FOUNDATION DRAINAGE SYSTEM.

UNDISTURBED GLACIAL TILL/CLEAN SURFACE SOUNDED BEDROCK, APPROVED BY THE GEOTECHNICAL CONSULTANT

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

BIRGIL

GEOTECHNICAL INVESTIGATION

PROPOSED MIX-USE HI-RISE DEVELOPMENT

1299 RICHMOND ROAD

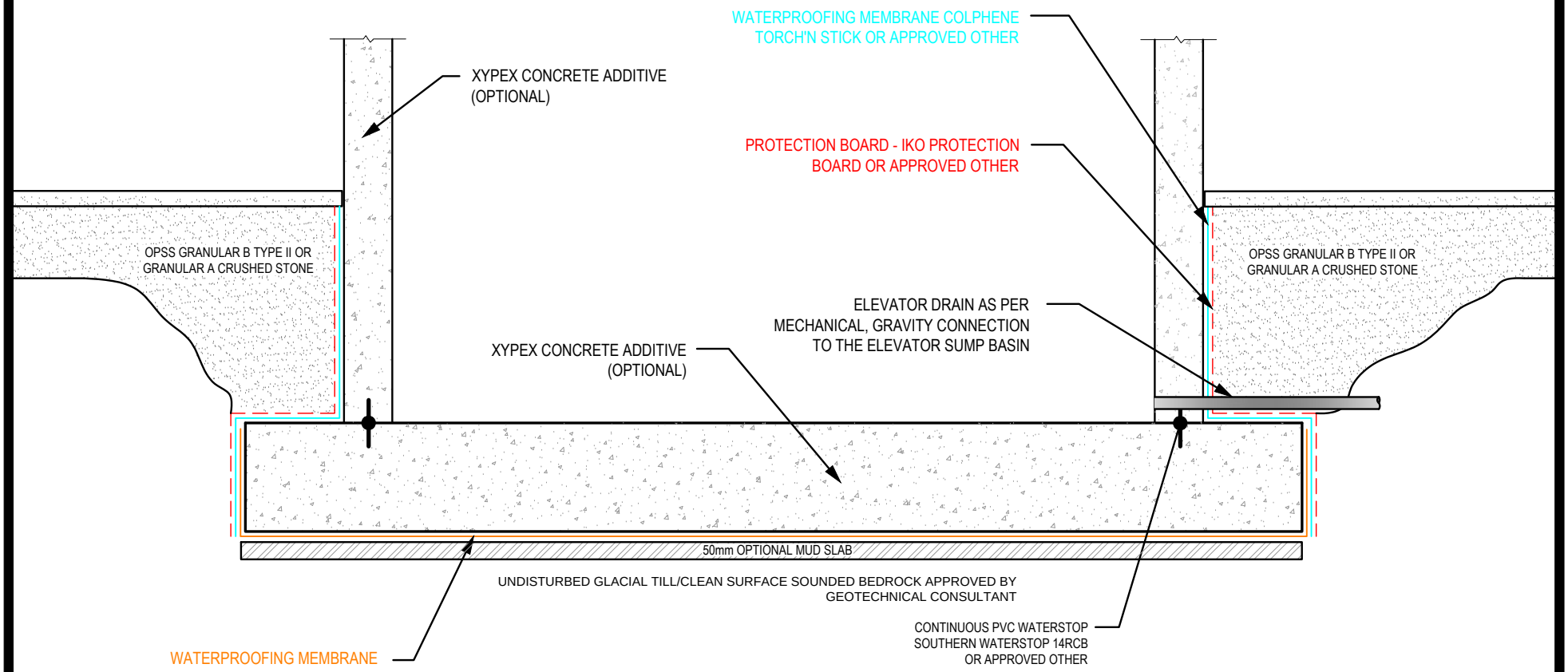
OTTAWA, ONTARIO

Title:

FOUNDATION DRAINAGE SYSTEM

Scale: NTS	**Date:** 04/2023
Drawn by: YA	**Report No.:** PG6598-1
Checked by: PT	**Drawing No.:** FIG. 2
Approved by: JV	**Revision No.:**

p:\autocad drawings\geotechnical\pg65xx\pg6598\pg6598 - figure 2 - water suppression system.dwg



NOTES:

1. IT IS RECOMMENDED THAT PERIODIC INSPECTIONS BE COMPLETED BY THE GEOTECHNICAL CONSULTANT AT THE TIME OF CONSTRUCTION DURING THE INSTALLATION OF THE ELEVATOR WATERPROOFING MEMBRANE(S).

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

BIRGIL

GEOTECHNICAL INVESTIGATION

PROPOSED MIX-USE HI-RISE DEVELOPMENT

1299 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

WATERPROOFING SYSTEM FOR ELEVATOR

Scale: N.T.S.	**Date:** 04/2023
Drawn by: YA	**Report No.:** PG6598-1
Checked by: PT	**Drawing No.:** FIGURE 3
Approved by: JV	**Revision No.:**

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STARFLOWER LANE

MH

ASSALY ROAD

SIDEWALK

BH 4-23
69.18
[56.74]

UNDERGROUND PARKING OUTLINE

1299 RICHMOND ROAD
PROPOSED MIX-USE HI-RISE BUILDING

BH 3-23
69.30
[55.25]

BH 2-23
69.06
(56.03)

CB2
T/G=68.91m

ASPHALTIC CONCRETE
PARKING LOT

CB1
T/G=68.96m

BH 1-23
69.32
(56.06)

BH 5-23
69.55
[56.77]

FH

CB

SIDEWALK

CB3
T/G=69.53m

RICHMOND ROAD

LEGEND:

- BOREHOLE WITH MONITORING WELL LOCATION
- 69.32 GROUND SURFACE ELEVATION (m)
- [56.77] BEDROCK SURFACE ELEVATION (m)
- (56.06) PRACTICAL AUGERING REFUSAL ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY BDP QUADRANGLE

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:400

0 5 10 15 20 25m

PATERSON GROUP
9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

BIRGIL
GEOTECHNICAL INVESTIGATION
PROPOSED MIX-USE HI-RISE DEVELOPMENT
1299 RICHMOND ROAD
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:400	Date: 03/2023
Drawn by: YA	Report No.: PG6598-1
Checked by: NS	Dwg. No.: PG6598-1
Approved by: JV	Revision No.:

11x17

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APPENDIX 3

TYPICAL FOUNDATION SLEEVE INSTALLATION

Typical 150 mm Diameter Sleeve Installation

Photo 1 – Step 1: It is recommended that the upper 1/3 of the 150 mm drainage sleeve be cut at a 45 degree angle to hydraulically connect the composite foundation drainage board to the interior and underfloor drainage system.

Photo 2 – Step 2: It is recommended that the 150 mm diameter drainage sleeve be installed by carefully cutting an ‘X’ shaped incision through the composite foundation drainage and inserting the 150 mm diameter drainage sleeve inside the ‘X’ by pulling the four (4) triangular flaps towards the installer.

Photo 3 – Step 3: Apply a suitable primer prior to the placement of the adhesive tape such as 3M tape, WP200 BlueSkine or equivalent.

Photo 4 – Step 4: An adhesive such as 3M tape, BlueSkin, or equivalent be utilized to seal the 150 mm drainage sleeve to the composite foundation drainage board to act as a barrier in preventing concrete from blocking connection during the placement of the exterior concrete foundation wall.

Photo 5 – Step 5: As an additional precaution, it is also recommended that an adhesive tape be placed on the interior outlet end of the drainage sleeve between the temporary form work to further prevent concrete from entering the drainage sleeve during the placement of concrete. Once the temporary form work has been removed, the adhesive tape can be cut away to allow groundwater to have a positive gravity connection to the interior perimeter and underfloor drainage system.

Appendix F Preconsultation

Kilborn, Kris

From: Philip Thibert <pthibert@brigil.com>
Sent: Friday, April 8, 2022 10:51 AM
To: Kilborn, Kris; Sharp, Mike
Subject: FW: Pre-Consultation Follow-Up: 1299 Richmond Road
Attachments: design_brief_TOR_1299 Richmond.pdf; Plans and Study List.pdf

---

From: McCreight, Laurel <Laurel.McCreight@ottawa.ca>
Sent: April 7, 2022 3:14 PM
To: Tyler Yakichuk <yakichuk@fotenn.com>
Cc: Lisa Dalla Rosa <dallarosa@fotenn.com>
Subject: Pre-Consultation Follow-Up: 1299 Richmond Road

CAUTION: This email is from an external sender. Do not click links or open attachments unless you recognize the sender and know the content is safe.

Hello Tyler,

Please refer to the below regarding the Pre-Application Consultation Meeting held on Thursday March 31st, 2022 for the property located at 1299 Richmond Road for a Major Zoning By-law Amendment and Site Plan Control Application to permit a residential development. I have also attached the required Plans & Study List for application submission.

Below are staff's preliminary comments based on the information available at the time of pre-consultation meeting.

Planning

- A Design Brief is required as part of the submission. The Terms of Reference of the Design Brief is attached for convenience. Please note:
    - Both a wind study and a shadow study is required.
    - The context study should include a broader area.
    - The City is undertaking an Secondary Planning Study for the area around the Lincoln Fields LRT station. The subject property is within the study area. The applicant should seek active engagement in the planning process. In the absence of a vision and policy directions from the Secondary Plan, the applicant is encouraged to study both the existing and potential future context and document the study in the Design Brief to inform the deliberation.
- The site is within a Design Priority Area. The proposed development, if it includes buildings greater than 6 storeys, will be subject to formal review by the City's Urban Design Review Panel. Information on scheduling and submission can be found on the UDRP website.
- A few high-level design comments:
    - In the current Official Plan, the site located within an Arterial Mainstreet corridor that supports mid-rise development with appropriate transition. In the new Official Plan, the site is within the Inner City Transect, on a Mainstreet Corridor and at the edge of a Hub. The new OP promotes urban built form. Policies of the Hub support high-rise development on a site within 300m radius or 400m walking distance from a Rapid Transit Station. Policies of the Mainstreet Corridor also support high-rise development on a lot fronting on a ROW that is greater than 30m, subject to the allowance of transition (on a lot that is of sufficient size). The property is measured at approximately 38m to 45m in depth. The Starflower is approximately 6m. Given the dimension of the site and the context it appears that one of the main design challenges is to establish appropriate transition if high-rise buildings are proposed. The

Secondary Plan may develop specific policies to guide built form transition in this area. In the absence of such area-specific policies, however, the applicant should study high-level City policies:
    - The City's current general guidance on development along Arterial Mainstreet, including built form transition, can be found in the current Official Plan and in the Urban Design Guidelines for Arterial Mainstreet.
    - In the new Official Plan, the City's general approach to transition can be found in the Urban Design section.
    - The applicant should also seek guidance from the Urban Design Guidelines for High-Rise Buildings. It is expected that a 45° angular plane will be used to guide the determination of appropriate transition.
  - In addition to transition, the provision of a park is also detrimental to the overall design.
  - The applicant should pay close attention to analyzing the character of Starflower Lane and ensure that the development will create a positive relationship with the adjacent low-rise residential uses along the Lane.
- Please refer to the Bird Friendly Design Guidelines.
- The application will be subject to public consultation (conducted through the posting of on-site signage, the notification of community groups, and through the City of Ottawa's DevApps website); the statutory public meeting for Zoning By-law Amendments is Planning Committee.
- Please continue to refine the proposed development based on the above comments.
  - A second pre-application consultation meeting is recommended.
- Please reach out to the Ward Councillor, Theresa Kavanagh, once the plans for the proposed development have been further refined and you are ready to submit formal applications.

Parks Planning

- As the site is larger than 4,000 sq.m., parkland dedication is required. The required amount dedication will be in accordance with the by-law.
- The dedication shall be at the southwest corner of the site and will function as an urban plaza.
- Please demonstrate how this urban plaza will link into the pedestrian network in the area and into other important sites or locations.
- If parkland was previously conveyed or CIL was paid, proof of this must be provided.
- Parks and Facilities Planning is currently undertaking a legislated review for the replacement of the Parkland Dedication By-law, with the new by-law to be considered by City Council in early July 2022. To ensure you are aware of parkland dedication requirements for your proposed development, we encourage you to familiarize yourself with the existing Parkland Dedication By-law and to sign up for project notifications on the Engage Ottawa project page or by emailing the project lead at Kersten.Nitsche@ottawa.ca

Please contact Parks Planner Kersten Nitsche for follow-up questions.

Transportation

- Follow Transportation Impact Assessment Guidelines:
  - A TIA is required. Submit Scoping Report at your earliest convenience to josiane.gervais@ottawa.ca.
  - Start this process asap. The application will not be deemed complete until the submission of the draft step 1-4, including the functional draft RMA package (if applicable) and/or monitoring report (if applicable).
  - TOD measures apply.
  - Request base mapping asap if RMA is required. Contact Engineering Services (https://ottawa.ca/en/city-hall/planning-and-development/engineering-services)
  - An update to the *TRANS Trip Generation Manual* has been completed (October 2020). This manual is to be utilized for this TIA. A copy of this document can be provided upon request.
- ROW protection on Richmond Road between Highway 417 and Ottawa River Parkway is 37.5m even. Future ROW line must be shown on the site plan, and all set-backs must be measured from this new property line.

- Strong preference for vehicular access to be provided by the lower-tiered roadway, Assaly Rd or Starflower Lane. Access onto Richmond should be avoided. Ensure site access meets the City's Private Approach Bylaw.
- Corner triangles as per OP Annex 1 - Road Classification and Rights-of-Way at the following locations on the final plan will be required (measure on the property line/ROW protected line; no structure above or below this triangle): Local Road to Arterial Road: 5 m x 5 m
- Clear throat requirements for >200 apartment units on an arterial is 40m. Ensure this length is provided. The clear throat length is measured from the ends of the driveway curb return radii at the roadway and the point of first conflict on-site. Note the minimum throat length provided must be maintained with the future ROW protection.

- Corner clearances should follow minimum distances set out within TAC Figure 8.8.2.

- TMP includes:
    - Site is within 400m of Lincoln Fields LRT Station
    - Transit Priority Corridor along Carling Ave (Isolated/Continuous Measures) (2031 Affordable Network)
- If Residential Only: As the site proposed is residential, AODA legislation applies for all areas accessible to the public (i.e. outdoor pathways, parking, etc.).
- OR
- If Mixed-Use: As the proposed site is mixed-use and for general public use, AODA legislation applies.
    - Ensure all crosswalks located internally on the site provide a TWSI at the depressed curb, per requirements of the Integrated Accessibility Standards Regulation under the AODA.
    - Clearly define accessible parking stalls and ensure they meet AODA standards (include an access aisle next to the parking stall and a pedestrian curb ramp at the end of the access aisle, as required).
    - Please consider using the City's Accessibility Design Standards, which provide a summary of AODA requirements. https://ottawa.ca/en/city-hall/creating-equal-inclusive-and-diverse-city/accessibility-services/accessibility-design-standards-features#accessibility-design-standards
- On site plan:
    - Show all details of the roads abutting the site up to and including the opposite curb; include such items as pavement markings, accesses and/or sidewalks.
    - Turning movement diagrams required for all accesses showing the largest vehicle to access/egress the site.
    - Turning movement diagrams required for internal movements (loading areas, garbage).
    - Show all curb radii measurements; ensure that all curb radii are reduced as much as possible and fall within TAC guidelines (Figure 8.5.1).
    - Show dimensions for site elements (i.e. lane/aisle widths, access width and throat length, parking stalls, sidewalks, pedestrian pathways, etc.)
    - Sidewalk is to be continuous across access as per City Specification 7.1.
    - Show slope of garage ramp on site plan. Note that underground ramps should be limited to a 12% grade and must contain a subsurface melting device when exceeding 6%. Ramp grades greater than 15% can be psychological barriers to some drivers.
    - Parking stalls at the end of dead-end parking aisles require adequate turning around space
    - Grey out any area that will not be impacted by this application.
- Noise Impact Studies required for the following:
    - Road
    - Rail
    - Stationary, due to the proximity to neighboring exposed mechanical equipment and/or if there will be any exposed mechanical equipment due to the proximity to neighboring noise sensitive land uses.

Please contact Transportation Project Manager Josiane Gervais for follow-up questions.

Engineering

- The Servicing Study Guidelines for Development Applications are available at the following address: https://ottawa.ca/en/planning-development-and-construction/developing-property/development-application-review-process/development-application-submission/guide-preparing-studies-and-plans#servicing-study-guidelines-development-applications
- Servicing and site works shall be in accordance with the following documents:
    - Ottawa Sewer Design Guidelines (October 2012)
    - Ottawa Design Guidelines – Water Distribution (2010)
    - Geotechnical Investigation and Reporting Guidelines for Development Applications in the City of Ottawa (2007)
    - City of Ottawa Slope Stability Guidelines for Development Applications (revised 2012)
    - City of Ottawa Environmental Noise Control Guidelines (January 2016)
    - City of Ottawa Park and Pathway Development Manual (2012)
    - City of Ottawa Accessibility Design Standards (2012)
    - Ottawa Standard Tender Documents (latest version)
    - Ontario Provincial Standards for Roads & Public Works (2013)
- Record drawings and utility plans are also available for purchase from the City (Contact the City's Information Centre by email at InformationCentre@ottawa.ca or by phone at (613) 580-2424 x.44455).
- Watermain Infrastructure:
    - As per Section 4.4.7.2 of the Ottawa Design Guidelines – Water Distribution, a DMA (District Metering Area) chamber will be required for private developments serviced by a connection 150mm or larger.
    - There is an available 305mm diameter CI watermain on Richmond Road. A water boundary condition request is needed for the water connection to the City main.
    - As per Section 4.4.7.2 of the Ottawa Design Guidelines – Water Distribution, a DMA (District Metering Area) chamber will be required for private developments serviced by a connection 150mm or larger.
    - Residential areas servicing 50 or more dwellings shall be connected with a minimum of two watermains, separated by an isolation valve to avoid the creation of a vulnerable service area.
    - Water Boundary condition requests must include the location of the service and the expected loads required by the proposed development. Please provide an email to Shika Rathnasooriya with the following information:
        - Location of service
        - Type of development and the amount of fire flow required (as per FUS, 1999 – See technical bulletin ISTB 2021-03).
        - Average daily demand: ___ l/s.
        - Maximum daily demand: ___l/s.
        - Maximum hourly daily demand: ___ l/s.
- Sanitary / Storm Infrastructure:
    - There is an existing 300mm diameter CONC sanitary sewer on Richmond Road.
    - A monitoring maintenance hole will be required for a private sanitary sewer outletting to a public sanitary sewer. The maintenance hole should be located in an accessible location on private property near the property line (ie. Not in a parking area).
    - All services (STM, SAN, WTR) should be grouped in a common trench to minimize the number of road cuts.
    - Sewer connections to be made above the springline of the sewermain as per:
        - Std Dwg S11.1 for flexible main sewers.
        - Std Dwg S11 (For rigid main sewers).
        - Std Dwg S11.2 (for rigid main sewers using bell end insert method).
        - Connections to manholes permitted when the connection is to rigid main sewers where the lateral exceeds 50% the diameter of the sewermain. – Connect obvert to obvert with the outlet pipe unless pipes are a similar size.
- The Stormwater Management Criteria, for the subject site, is to be based on the following:

- The proposed site falls within the Pinecrest Creek Study Area. As the flows will ultimately discharge downstream of the Ottawa River Parkway pipe, the design criteria will be based on flows discharging directly to the Ottawa River.
- The 100-year post development peak flows for the development area discharging to the minor storm sewer system are to be controlled to the corresponding 2-year pre-development peak flows. Onsite storage is to be provided for storm events up to and including the 100-yr storm.
- There should be no stormwater ponding in parking areas or drive aisles during the 2-year storm event.
- Quality control to be provided as specified by the RVCA.
- The design of the storm sewers in the area are based on a 2-yr storm. If discharging to a storm sewer, the SWM criteria is to be based on the following for the development area:
  - The 2-year storm event using the IDF information derived from the Meteorological Services of Canada rainfall data, taken from the MacDonald Cartier Airport, collected 1966 to 1997.
  - The pre-development runoff coefficient or a maximum equivalent 'C' of 0.5, whichever is less.
  - A calculated time of concentration (Cannot be less than 10 minutes).
  - Flows to the storm sewer in excess of the 2-yr storm release rate, up to and including the 100-year storm event, must be detained on site.

- Phase 1 ESAs and Phase 2 ESAs must conform to clause 4.8.4 of the Official Plan that requires that development applications conform to Ontario Regulation 153/04.
- As per the City of Ottawa Slope Stability Guidelines for Development Applications an engineering report is required for any retaining walls proposed 1.0 m or greater in height within the subject site that addresses the global stability of the wall and provides structural details. A Retaining Wall Stability Analysis Report and Retaining Wall Structural Details are required to be provided from a Professional Engineer licensed in the Province of Ontario that demonstrates the proposed retaining wall structure has been assessed for global instability as per City standards. Please ensure the analysis and required documentation are provided as part of the submission to address this comment.

Please contact Project Manager Shika Rathnasooriya for follow-up questions.

Other

Please refer to the links to the guide to preparing studies and plans and development application fees for general information. Additional information is available related to building permits, development charges, and the Accessibility Design Standards. Be aware that other fees and permits may be required, outside of the development review process. You may obtain background drawings by contacting informationcentre@ottawa.ca.

These pre-consultation comments are valid for one year. If you submit a development application(s) after this time, you may be required to meet for another pre-consultation meeting and/or the submission requirements may change. You are as well encouraged to contact us for a follow-up meeting if the plan/concept will be further refined.

Please do not hesitate to contact me if you have any questions.

Regards,
Laurel

Laurel McCreight MCIP, RPP
Planner II
Development Review West
Urbaniste
Examen des demandes d'aménagement ouest

City of Ottawa | Ville d'Ottawa
613.580.2424 ext./poste 16587

ottawa.ca/planning / ottawa.ca/urbanisme

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Appendix G Drawings