Servicing and Stormwater Management Report – 1125 - 1149 Cyrville Road

Stantec Project No. 160401672

November 24, 2021

Prepared for:

Westrich Pacific Corp.

Prepared by:

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Table of Contents

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 INTRODUCTION

Stantec Consulting Ltd. has been commissioned by Westrich Pacific Corp. to prepare the following site servicing and stormwater management (SWM) report in support of a site plan control and zoning amendment application for a proposed multi-family residential development located at 1125 - 1149 Cyrville Road. The site is situated on Cyrville Road, between Ogilvie and Cummings avenue intersections in the City of Ottawa. (see key plan in **Figure 1**).

The proposed development area (0.84 ha) consists of two residential high-rise buildings A and B to be developed in two phases i.e. Phase I and Phase II respectively. Building A is a 6-storey building with a rooftop amenity area on level 6 and has two levels of underground parking with a total of 250 parking stalls. Building A is to contain 208 units in total consisting of 60 one-bedroom units, 143 two-bedroom units, 5 three-bedroom units, and 2399m^2 of communal amenity areas.

Building B is a 12-storey building with roof top amenity area on level 6, it has two levels of underground parking with a total of 104 parking stalls. Building B would house 146 total units, including 55 one-bedroom units, 85 two-bedroom units, six three-bedroom units, and 1967m^2 of communal amenity areas. Parking is to be provided via two underground parking levels (P1 and P2) in each building, totaling 354 parking stalls overall. The site plan has been provided in **Appendix B**.

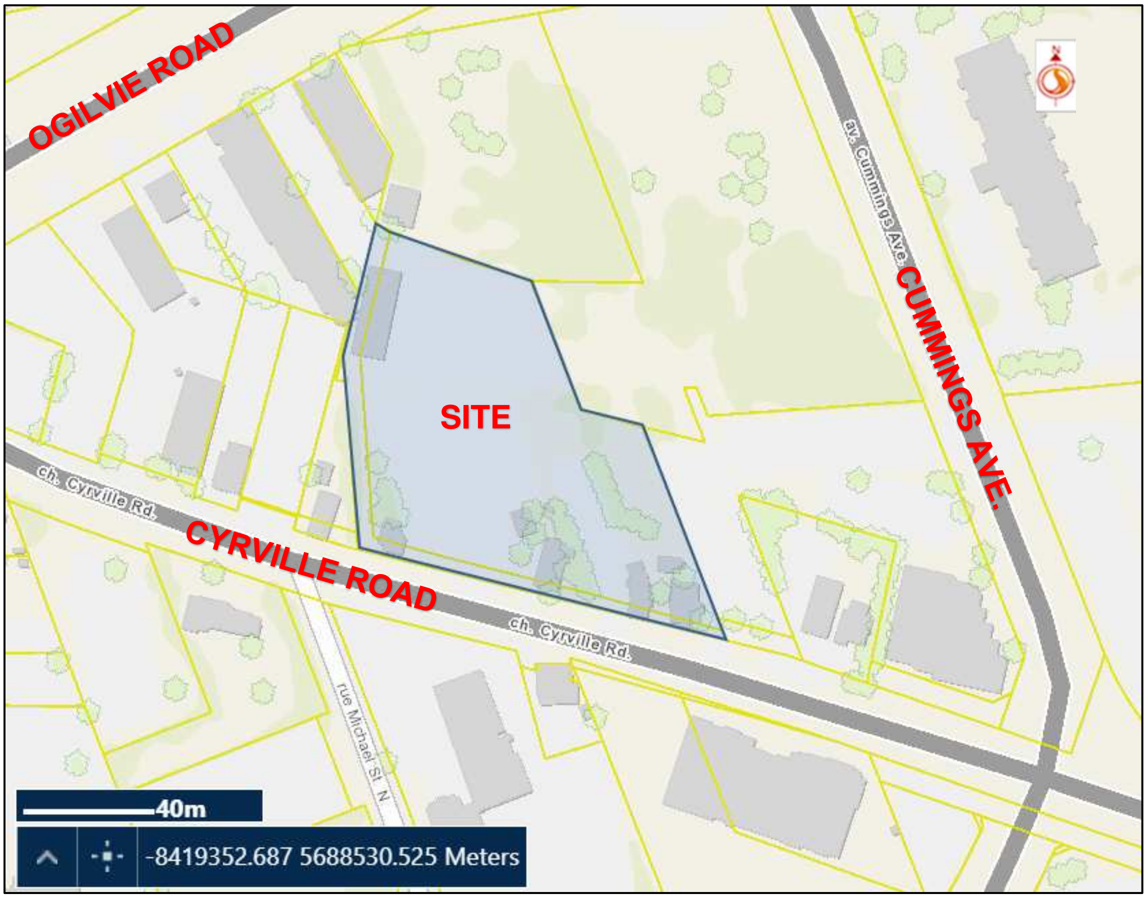

Figure 1: Key Plan (1125 - 1149 Cyrville Road Site highlighted in Blue)

1.1 OBJECTIVE

This Site Servicing and Stormwater Management Brief has been prepared to present a servicing scheme that is free of conflicts and presents the most suitable servicing approach that complies with the relevant City design guidelines. Details of the existing infrastructure were obtained from available as-built drawings and consultation with J+S Architect Inc., Westrich Pacific Corp, City of Ottawa staff, and the adjoining property owners. Infrastructure requirements for water supply, sanitary sewer, and storm sewer services are presented in this report.

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

- Potable Water Servicing
  - Estimate water demands to characterize the feed for the proposed development which will be serviced by an existing 250mm diameter cast iron watermain fronting the site along Cyrville Road.
  - Watermain servicing for the development is to be able to provide average day and maximum day and peak hour demands (i.e., non-emergency conditions) at pressures within the normal operating range of 50 to 80 psi (345 to 552 kPa) under maximum day condition and not less than 40psi under peak hour demand conditions.
  - Under fire flow (emergency) conditions with maximum day demands, the water distribution system is to maintain a minimum pressure greater than 20 psi (140 kPa).
- Prepare a grading plan in accordance with the proposed site plan and existing grades.
- Stormwater Management and Servicing
  - Define major and minor conveyance systems in conjunction with the proposed grading plan.
  - Post development peak 100-year flows controlled to the predevelopment peak 5-year release rate with a runoff coefficient of C=0.5 and a time of concentration of 10 minutes as estimated based on the existing storm sewer infrastructure servicing the existing site.
  - Excess stormwater to be detained on-site to meet the 5-year pre-development target release rate.
  - Connect to the proposed 600mm diameter concrete storm sewer within the Cyrville Road right-of-way.
  - Meet RVCA stormwater quality control requirements for the site.
  - Define and size the proposed storm sewer system.
- Wastewater Servicing
  - Estimate wastewater flows generated by the development and size sanitary sewers which will outlet to the existing 375mm diameter PVC sanitary sewer located on Cyrville Road.
  - Define and size the proposed sanitary sewer system / building services.

The accompanying **Drawing SSP-1** included in **Appendix F** illustrates the proposed internal servicing scheme for the site.

2.0 REFERENCES

Documents referenced in preparation of this Servicing and Stormwater Management Report include:

- *1125 - 1149 Cyrville Road pre-consultation comments,* City of Ottawa, March 2021.
- *City of Ottawa Design Guidelines - Water Distribution*, City of Ottawa, July 2010 (including all subsequent technical bulletins).
- *City of Ottawa Sewer Design Guidelines (SDG)*, City of Ottawa, October 2012 (including all subsequent technical bulletins).
- *Geotechnical Investigation, Proposed Commercial Development 1125 to 1149 Cyrville Road Ottawa Ontario*, Paterson Group Inc., November 2021.
- *Phase I Environmental Site Assessment Update, Proposed Residential Development 1125 to 1149 Cyrville Road Ottawa Ontario,* Paterson Group Inc., November 2021.
- *Phase II Environmental Site Assessment, Proposed Residential Development 1125 to 1149 Cyrville Road Ottawa Ontario,* Paterson Group Inc., November 2021.
- *Phase I-II Environmental Site Assessment, Proposed Commercial Development 1125 to 1149 Cyrville Road Ottawa Ontario,* Paterson Group Inc., March 2020.
- *Environmental Remedial Action Plan, Proposed Site Redevelopment 1125 to 1149 Cyrville Road Ottawa Ontario,* Paterson Group Inc., February 2020.

3.0 POTABLE WATER SERVICING

The proposed site is located within Pressure Zone 1E of the City of Ottawa's water distribution system. The proposed development will be serviced by the existing 250mm diameter watermain on Cyrville Road. In order to create a suitable water service connection for the property, two new service connections to the existing 250mm dia. watermain on Cyrville road separated by a new valve within the Cyrville Road watermain, and a fire hydrant within the private access right-of-way have been proposed as shown on **Drawing SSP-1** in **Appendix F**. Servicing for Building B is proposed to be provided through internal plumbing of Building A within accessible and maintainable space via underground parking areas, and is proposed to cross over the existing storm sewer easement roughly bisecting the site to the underground parking areas of Building B.

The proposed development area (0.84 ha) consists of two residential high-rise buildings (Building A & B) Building A is to contain 208 units in total consisting of 60 one-bedroom units, 143 two-bedroom units, 5 three-bedroom units, and 2,399m^2 of communal amenity areas. Building B is to contain 146 total units, including 55 one-bedroom units, 85 two-bedroom units, six three-bedroom units, and 1967m^2 of communal amenity areas.

Water demands were calculated using the City of Ottawa Water Distribution Guidelines (2010) and revised with Technical bulletin ISTB-2021-03 to determine the typical operating pressures to be expected at the building (see detailed calculations in **Appendix A.1**). A demand rate of 280 L/cap/day was applied for the residential population of the proposed site. The average daily (AVDY) residential demand was estimated with population densities as per City of Ottawa Guidelines; 1.4 persons per one-bedroom apartments, 2.1 persons per two-bedroom apartments, and 3.1 persons per two-bedroom apartments with den and three-bedroom apartments.

An estimated demand of 28,000 L/ha/day was applied to the 2,399m^2 and 1,967m^2 of common areas respectively for Buildings A and B. Maximum Day (MXDY) demands were determined by multiplying the AVDY demands by a factor of 2.5 for residential areas and by a factor of 1.5 for common areas. Peak hourly (PKHR) demands were determined by multiplying the MXDY demands by a factor of 2.2 for residential areas and by a factor of 1.8 for common areas. Residential water demands are detailed in **Table 3-1**.

Table 3-1: Residential Population and Water Demands

Building	Total Units	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
A	208	400	1.37	3.36	7.34
B	145	275	0.95	2.32	5.07
TOTAL SITE	354	675	2.32	5.68	12.41

The proposed development has an average day demand of 2.32 (200.5 m^3/day). Since this value exceeds 50 m^3/day, two service laterals will need to be provided for the development per the City's Water

Distribution Guidelines. Two new 150mm diameter water services will be connected to the existing 250 mm diameter watermain on Cyrville Road separated by a new isolation valve on the main.

The fire flow requirement was calculated in accordance with Fire Underwriters Survey (FUS) and determined to be approximately 12,000 L/min (200 L/s) for Building A, and approximately 8,000 L/min (133.3 L/s) for Building B. Fire flow demand for both buildings were estimated using a non-combustible construction type with two-hour fire separations provided between each floor. Additionally, it is anticipated that both buildings will be sprinklered, with final sprinkler design to conform to the NFPA 13 standard.

As a result, the gross floor area of the ground floor (floor with the largest footprint) + 25% of the gross floor area of the two immediately adjoining floors (the second floor and third floor) were used in the FUS calculation for both buildings, as per Page 17 of the *Fire Underwriters Survey's Water Supply for Public Fire Protection* (1999). Detailed fire flow calculations per the FUS methodology are provided in **Appendix A.2**.

Table 3-2 shows the hydraulic boundary conditions provided by the City of Ottawa on July 20, 2021 based on domestic and fire flow demands estimated with a 350L/cap/day residential demand rate prior to the release of Technical Bulletin ISTB-2021-03. These boundary conditions are used as conservative values when analyzing the level of service for water demands presented in the table above. The boundary conditions are also included in **Appendix A.2**.

Table 3-2: Boundary Conditions

	Connection @ 1125 Cyrville Road
Min. HGL (m)	109.5
Max. HGL (m)	118.4
Max. Day + Fire Flow (200 L/s) (m)	105.0
Max. Day + Fire Flow (133.3 L/s) (m)	109.5

The desired normal operating objective pressure range as per the City of Ottawa 2010 Water Distribution Design Guidelines is 345 kPa (50 psi) to 552 kPa (80 psi) and no less than 276 kPa (40 psi) at ground elevation. Furthermore, the maximum pressure at any point in the water distribution should not exceed 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.

Both Building A and B's proposed finished floor elevation of 70.32m will serve as the ground elevation for the calculation of residual pressures at ground level. At the peak hour flow conditions (i.e., minimum HGL), the resulting boundary condition HGL of 109.5 m corresponds to a peak hour pressure of 384.3 kPa (55.7 psi). As both buildings have an average storey height of 3.16 m, an additional 31 kPa (4.5 psi) of head loss is incurred for every additional storey over ground level. This results in a peak hour pressure of 229.3 kPa (33.2 psi) at the top floor of the 6-storey Building A, and 43.3 kPa (6.3 psi) at the top floor of the 12-storey Building B, both of which are insufficient pressures to entirely service both buildings. Therefore, a booster pump inside both buildings will be required to maintain an acceptable level of service on the higher floors. This booster pump is to be sized and designed by the buildings' mechanical engineer.

A maximum pressure check can be conducted using the building's finished floor elevation (70.32 m) and the maximum boundary condition HGL of 118.4 m. This results in a pressure of 471.6 kPa (68.4 psi). Since this value is below 80 psi, pressure reducing valves will not be required.

Boundary conditions provided by the City confirm that a fire flow rate of 12,000 L/min (200 L/s) by Building A would have a residual pressure of 340.2 kPa (49.3 psi) on Cyrville Road. Meanwhile, Building B's 8,000 L/min (133.3 L/s) fire flow would result in a residual pressure of 384.3 kPa (55.7 psi) at the watermain.

Building A is within 75 – 150m of the existing fire hydrants on St. Michael Street and Cyrville street as well as the proposed fire hydrant while building is B is within 75 -150m of the existing fire hydrant on St. Michael Street and the proposed hydrant on the northwest corner of Building A. The aggregate flow capacity of all available fire hydrants are 15,200 L/min and 9,500L/min for building A and B respectively according to ISTB-2018-02 thereby meeting the required fire flow for the site.

The proposed hydrant has been located to ensure a maximum 45m distance to proposed building fire department connections per Ontario Building Code requirements.

In conclusion, based on the boundary conditions available, the 250 mm diameter watermain on Cyrville Road provides adequate fire flow capacity as per the requirements of the Fire Underwriters Survey while respecting City of Ottawa design guidelines. Two 150 mm diameter service laterals connected to the 250 mm diameter watermain on Cyrville Road will be capable of providing the anticipated water demands to the lower storeys. A booster pump, to be designed by the buildings' mechanical engineer, will be required to maintain acceptable pressures for the upper storeys of both buildings.

4.0 WASTEWATER SERVICING

As illustrated on **Drawing SSP-1 and SA-1**, sanitary servicing for the proposed development will be provided through a proposed 200 mm diameter connection along the private access road, connecting to the existing 375 mm diameter PVC sanitary sewer flowing westward on Cyrville Road. Servicing for Building B will be interconnected through internal building plumbing of Building A.

Using the City of Ottawa's recommended population densities, the proposed 6-storey residential high-rise Building A is to contain 208 units in total consisting of 60 one-bedroom units,143 two-bedroom units, 5 three-bedroom units, and 2399m^2 of common areas with a total estimated population of 400 people. The proposed 12-storey residential high-rise Building B is to contain 146 units in total consisting of 55 one-bedroom units, 85 two-bedroom units, 6 three-bedroom units, and 1967m^2 of common areas with a total estimated population of 275 people. The anticipated wastewater peak flow generated from the proposed development is summarized in **Table 4-1** while the sanitary sewer design sheet is included in **Appendix C.1**.

Table 4-1: Estimated Wastewater Peak Flow

Residential/Commercial Peak Flows						Infiltration Flow (L/s)	Total Peak Flow (L/s)	
Building		# of Units	Population	Peak Factor	Peak Flow (L/s)			
A	Residential	208 units	400	3.38	4.91	0.21	5.12	**8.78**
B	Residential	146 units	275	3.79	3.38	0.28	3.66	

1. Average residential sanitary flow = 280 L/p/day per City of Ottawa Sewer Design Guidelines.
2. Peak factor for residential units calculated using Harmon's formula. Used a Harmon correction factor of 0.8.
3. Apartment population estimated based on 1.4 persons/unit for one-bedroom apartments, 2.1 persons/unit for one-bedroom with den & two-bedroom apartments, 3.1 persons/unit for two-bedroom with den & three-bedroom apartments
4. Estimated commercial/amenity area/lobby peak flows = 28,000 L/ha/day.
5. Infiltration flow = 0.33 L/s/ha.

The City has expressed no concerns over the 375 mm diameter sanitary sewer on Cyrville Road providing sufficient capacity to service the proposed development

The drains within covered portions of both buildings' underground parking garages will need to be pumped and ultimately outlet to the proposed sanitary sewer system. The design of these drains, internal plumbing, and associated pumping system 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.

5.0 STORMWATER MANAGEMENT

5.1 OBJECTIVES

The goal of this 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 and Rideau Valley Conservation Authority (RVCA), and to provide sufficient details required for approval and construction.

5.2 EXISTING CONDITIONS AND SWM CRITERIA

The proposed re-development area (0.84 ha) is currently a vacant lot mixed with pavement and vegetation areas. The existing pavement structures on the site will be removed to allow for the proposed development.

The Stormwater Management (SWM) criteria for the subject site is based on pre-application consultation comments in **Appendix D.1** as provided by the City of Ottawa in March 2021 as follows:

i. Post-development peak flows up to 100-year event are to be controlled to the pre-development peak 5-year release rate. Excess stormwater is to be detained on-site.
ii. The 5-yr 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.
iii. Maximum Pre-development runoff coefficient of C=0.50.
iv. Pre-development time of concentration of tc=20, and post development tc=10.
v. Permissible surface ponding (including dynamic flow depth) of 350mm for paved areas during the 100-year storm event. No major system spillage to adjacent properties is to occur for events up to and including the 100-year storm event.
vi. Emergency major overland flows are to be directed to the adjacent municipal ROW.
vii. 100-year major system spill elevations must be 300mm lower than adjacent building openings.
viii. Permanent storm sewer infrastructure (apart from a single building service connection location for Building B) will not be permitted within the existing storm sewer easement on-site.

Other criteria considered in the SWM design are described in Section 5 of the Ottawa Sewer Design Guidelines (October 2012) including all subsequent technical bulletins.

Pre-development (i.e., current) site conditions have been classified into impervious (hard) and pervious (soft) areas, with impervious areas accounting for 77.14% (0.648 ha) while pervious areas cover up to 22.86% (0.192 ha) of the site. Based on these statistics, the overall pre-development runoff coefficient (C) for the site was calculated as C=0.73. As a result, a C value of 0.50 will be used to estimate the allowable release rate from the site.

The allowable peak stormwater flow rate for the site was calculated as follows using the Modified Rational Method:

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

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

$$5 - year\ Intensity\ \left(\frac{mm}{hr}\right) = \frac{998.071}{(20 + 6.053)^{0.814}} = 70.25\ mm/hr$$

$$Q = 2.78(0.50)(70.25mm/hr)(0.84\ ha) = \mathbf{82.0}\boldsymbol{L/s}$$

Therefore, the post-development peak flows up to the 100-year storm event must be controlled to **82.0 L/s**. The pre-development time of concentration was assumed to be 20 minutes, as directed by the City during pre-consultation.

5.3 STORMWATER MANAGEMENT DESIGN

The proposed 0.84 ha re-development area will be serviced by a proposed 600mm diameter storm sewer running east to west on Cyrville Road, as shown on **Drawing SD-1** in **Appendix F**.

Stormwater cisterns located in the underground parking area will attenuate peak flows from the roofs of both buildings, rooftop amenity areas, outdoor amenity areas and landscaped areas within the site to ensure that the overall site release rate meets the allowable release rate. The proposed stormwater cisterns will be fed by the internal plumbing of the buildings. As shown on **Drawing SD-1** in **Appendix F**, peak flows from the proposed Cistern B will outlet to a 300 mm diameter storm service lateral crossing the easement and drained to a storm service stub within the proposed site private access via Building A. The internal plumbing in Building A should accommodate an independent connection of Building B to the storm service stub at Building A.

Catch basins and landscape drains for the areas tributary to the stormwater cistern will connect to the cistern via internal plumbing (designed by the building's mechanical engineer). The stormwater cisterns will be pumped at a controlled rate and ultimately outleting to the proposed 600 mm diameter concrete storm sewer on Cyrville Road (see **Drawing SD-1** in **Appendix F)**. The stormwater cistern's controlled release rate will be set via pump to be designed by a mechanical engineer based on calculations provided in **Appendix D**.

The stormwater cistern location(s) will be coordinated with the building's architect and structural engineer. Peak flows have been identified to the building's mechanical engineer to size the internal plumbing system appropriately.

Surface storage is also proposed on the private access way to retain storm run-off exceeding a 2-year storm event.

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

5.3.1 Water Quantity Control

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** delineates the appropriate sub-catchment areas. The MRM spreadsheet is included in **Appendix D.2.**

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

1) Rooftop storage is available on three of five roof catchment areas.
2) On-site stormwater runoff (including green areas) will be collected using a combination of area drains, catch-basins, and internal building plumbing for detention in two stormwater cisterns A and B each for Building A and B respectively.
3) Some pedestrian access and landscaping areas on the south and eastern perimeters of the site will sheet drain uncontrolled to Cyrville Road (UNC-1).
4) The storm runoff within the access road area of the site will be captured into the site storm sewer system and directed to a proposed oil/grit separator unit.

In order to meet the target release rate for stormwater of 82.0 L/s up to the 100-year storm, on-site storage is required.

5.3.1.1 Rooftop Storage

Rooftop storage is proposed on the site on all rooftop areas excluding areas intended for outdoor amenity use such as: ROOF 1B and ROOF 2B. These rooftop amenity areas will drain uncontrolled to the cistern via the internal plumbing of the respective buildings for storage and controlled release (see Drawing SD-1).

Rooftop storage will be achieved by installing restricted flow roof drains. The following calculations assume the roof will be equipped with standard Watts Model R1100 Accuflow Roof Drains or approved equivalent, see **Appendix D.2** for Modified Rational Method design sheet.

Watts Drainage "Accutrol" roof drain weir data has been used to calculate a practical roof release rate and detention storage volume for the rooftops. It should be noted that the "Accutrol" weir has been used as an example only, and that other products may be specified for use, provided that the total roof drain release rate is restricted to match the maximum rate of release indicated in **Table 5-1**, and that sufficient roof storage is provided to meet (or exceed) the resulting volume of detained stormwater.

Table 5-1: Roof Control Area

Roof ID	Accutrol weir setting	# of Drains	5-yr release rate (L/s)	100-yr release rate (L/s)	100-yr ponding depth (m)	100-yr storage required (cu.m)	Storage provided (cu.m)
Roof 1A	50% open	3	3.68	4.70	0.15	39.24	40.00
Roof 2A	50% open	3	3.07	3.76	0.15	31.38	32.00
Roof 1C	75% open	5	6.15	7.85	0.15	67.12	68.00

5.3.1.2 Access Road Areas

The private access road consists of a flexible pavement providing access and exits to the underground parking lots of both Building B. SWM in the proposed access road areas will be achieved using a proposed catchbasin equipped with an inlet control device (ICD) to restrict minor system peak flows (2-year) to the 100-year storm. **Table 5-2** below shows the characteristics of the proposed ICDs (see **Appendix D.2** for detailed calculations).

Table 5-2: Schedule of Inlet Control Device

Outlet Orifice Name	Catch basin ID	Tributary Area ID	ICD Type	5 - Year Head (m)	100 - Year Head (m)	5 -Yr Flow (L/s)	100 - Yr Flow (L/s)
L100-IC	CB L100A	L100A	LMF 105	1.53	1.54	12.11	12.15

The 5-year storage / flow values are conservative and represent the maximum permissible release rates/storage volumes

5.3.1.3 Uncontrolled Areas

One uncontrolled area (UNC-1) cannot be graded to enter the site storm sewer system and as such, they will sheet drain to Cyrville Road to the south as per existing conditions (see **Drawing EX-1** and **Drawing SD-1**).

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

Area IDs	Area (ha)	Runoff 'C' (5- Year)	5 Year uncontrolled peak flow (L/s)	Runoff 'C' (100 -Year)	100 Year uncontrolled peak flow (L/s)
UNC-1	0.07	0.48	9.73	0.60	20.85

5.3.1.4 Stormwater Cistern

The allowable release rate from the proposed building's underground cistern was determined by subtracting all uncontrolled 100-year peak flows (area UNC-1) as well as Access Road tributary contributions from the allowable 82.0 L/s release rate, which results in approximately 49.67 L/s.

The stormwater cisterns will be designed to provide a combined storage of 61 m^3 of storage with a maximum controlled release rate of 49.00 L/s. The stormwater cisterns are each to discharge at controlled release rates using a pump. **Table 5-5** and **Table 5-5** summarizes the respective flow rates and volume of stormwater in Cisterns A and B in the 5-year and 100-year storm events.

Table 5-4: Proposed Cistern-A 5 and 100-Year Release Rate

Storm Return Period	Area IDs	Area (ha)	$Q_{release}$ (L/s)	$V_{required}$(m^3)	$V_{available}$ (m^3)
5-year	ROOF1A, ROOF1B, ROOF1C, TANK 1A, TANK1B, TANK 1C	0.12	34.00	7.86	39.00
100-year	ROOF1A, ROOF1B, ROOF1C, TANK 1A, TANK1B, TANK 1C	0.12	34.00	38.70	

Table 5-5: Proposed Cistern-B 5 and 100-Year Release Rate

Storm Return Period	Area IDs	Area (ha)	$Q_{release}$ (L/s)	$V_{required}$ (m^3)	$V_{available}$ (m^3)
5-year	ROOF 2A, ROOF 2B, TANK 2B, TANK 2A	0.20	15.00	5.40	21.00
100-year	ROOF 2A, ROOF 2B, TANK 2B, TANK 2A	0.20	15.00	20.77	

5.3.2 Results

Table 5-6 and

Table 5-7 demonstrate that the proposed stormwater management plan provides adequate attenuation storage to meet the target peak outflow for the site.

Table 5-6: Estimated Post-Development Discharge (5-Year)

Area Type	$Q_{release}$ (L/s)	Target (L/s)
Controlled Cistern Discharge (Tank 1A and Tank 2B)	49.00	82.0
Access road areas (L100)	12.11	
Uncontrolled (UNC-1)	9.73	
Total	**71**	

Table 5-7: Estimated Post-Development Discharge (100-Year)

Area Type	$Q_{release}$ (L/s)	Target (L/s)
Controlled Cistern Discharge (Tank 1A and Tank 2B)	49.00	82.0
Access road areas (L100)	12.15	
Uncontrolled (UNC-1)	20.85	
Total	**82**	

5.3.3 Water Quality Control

The RVCA confirmed that enhanced water quality protection (80% TSS removal) is required for the site as distance from the downstream outlet to a watercourse is less than 500m. The water quality objective noted is for 'enhanced' (80% TSS removal, refer to correspondence with RVCA in **Appendix D.3**.)

To achieve this end, storm runoff within the access road area of the site will be captured into the site storm sewer system and directed to a proposed oil/grit separator unit. The Stormceptor sizing software has been used to size the required unit to provide 80% long-term TSS removal based on proposed drainage areas (i.e., 0.84ha) as shown in the Stormceptor sizing design sheet included in **Appendix D.5.**

An Imbrium Stormceptor EF06 designed to provide 80% TSS removal has been proposed to collect and treat storm runoff from the site before outleting to the proposed 600mm diameter storm sewer on Cyrville Road as shown in **Drawing SSP-1** in **Appendix F.** The EF06 unit has been used as an example only and other approved equivalent products may be specified for use so long as an equivalent treatment rate and unit oil/sediment storage capacity may be achieved.

6.0 GRADING AND DRAINAGE

The proposed re-development site measures approximately 0.84 ha in area. A detailed grading plan (see **Drawing GP-1**) has been prepared to satisfy the stormwater management requirements described in **Section 5.0** and to allow for positive drainage away from the face of the building.

The site grading along the access road is designed to effectively drain stormwater runoff in the area into proposed catch basins. Grading for the access ramp to the underground parking levels have been coordinated with the architect. The subject site in its majority maintains overland flow routes towards Cyrville Road to the south.

7.0 UTILITIES

Hydro Ottawa, Bell, Rogers, and Enbridge all have existing utility plant in the area, which will be used to service Building A of the site. As directed by the City of Ottawa, no private utilities are to cross the central storm sewer easement between Building A and B. As such, electrical, gas, and other utilities will be supplied to Building B from the adjacent property to the north, through Ogilvie Road. The detailed design of the required utility services will be further investigated as part of the composite utility planning process, which will follow design circulation for the servicing plans.

Municipal water and sewer services are supplied to both buildings by underground service trenches along the private access road, connecting to Cyrville Road. Electricity and the phone line are to service Building A from underground service trenches south of the buildings

November 24, 2021

8.0 EROSION CONTROL DURING CONSTRUCTION

In order 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. Provide sediment traps and basins during dewatering works.
7. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
8. 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 EC/DS-1** for the proposed location of silt fences, straw bales, and other erosion control measures.

9.0 GEOTECHNICAL INVESTIGATION AND PHASE I ESA

9.1 GEOTECHNICAL INVESTIGATION

The geotechnical report for the site was prepared by Paterson Group Inc. on November 16, 2021 (**see Appendix E.1**). This report was commissioned for a proposed residential development by Westrich Pacific Corp, based on field investigations completed in August 16 and 17, 2011.

The investigation consisted of twelve boreholes advanced to a maximum depth of 5.7m below existing grade. As stated in the geotechnical report, the subsurface profile encountered at the boreholes consists of pavement structure, topsoil or crushed stone fill at ground surface underlain by brown silty clay with gravel and/or silty sand with gravel. A weathered shale bedrock was encountered below the above-noted layers at all borehole locations. Shale bedrock was cored at BH 1, BH 2 and BH 12 to a maximum depth of 5.7 m below existing ground surface. The bedrock in the immediate area of the subject site consists of potentially expansive shale from the Billings Formation at a 2 to 5 m depth.

Three groundwater monitoring wells were installed at BH1, BH2, and BH12 as part of the geotechnical investigation. Groundwater levels at the borehole locations were measured on August 22, 2011 and were found to be dry upon completion of the field program. It should be noted that groundwater levels are subject to seasonal fluctuations and can thus vary at the time of construction.

Bedrock removal will be required for the proposed building excavations, and can be accomplished by hoe ramming where only small quantity of the bedrock needs to be removed, while line drilling and controlled blasting would be ideal for larger quantities. Sound bedrock may be removed by line drilling and 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 of 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 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 side walls. A minimum 1m 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.

A potential for heaving and rapid deterioration of the shale bedrock exists at this site. Paterson recommends limiting exposure of the bedrock surface to oxygen, through either a double-sided pour or blind pour. The bedrock surface within the proposed building footprint should be protected from excessive dewatering and exposure to ambient air. To accomplish this a minimum 100 mm thick layer of lean concrete (minimum 15 MPa - 28 day compressive strength) should be placed across the base of the excavation. As an alternative, a blind pour of the basement wall against the bedrock excavated face is also acceptable. Excavated vertical sides of the exposed bedrock where groundwater is encountered can be protected using a sprayed elastomeric coating waterproofing layerto seal the bedrock from exposure to air and dewatering.

Footings placed on a clean, surface-sounded bedrock surface can be designed using a bearing resistance value at serviceability limit states (SLS) of 500 kPa and a factored bearing resistance value at ultimate limit states (ULS) of 750 kPa. A geotechnical resistance factor of 0.5 was applied to the reported bearing resistance values at ULS. Footings placed on engineered fill approved by the geotechnical consultant can be designed using a bearing resistance value at SLS of 150 kPa and a factored bearing resistance value at ULS of 300 kPa. Detailed design parameters to be considered in the foundation design are detailed in **Appendix E.1**

The report recommends a rigid pavement structure for the parking garage and a flexible pavement structure for the design of the access lanes.

Table 9-1: Recommended Rigid Pavement Structure - Parking Garage

Material	Thickness
Reinforced Concrete Slab	125 mm
Bedding, OPSS Granular A Crushed Stone	200 mm
Subgrade: In situ soil, or OPSS Granular B Type I or II material placed over in situ soil	

Table 9-2: Recommended Flexible Pavement Structure – Access Lane

Material	Thickness
Wear Course, HL-3 or Superpave 12.5 Asphaltic Concrete	40 mm
Binder Course, HL-8 or SP 19 Asphaltic Concrete	50 mm
Granular Base Course, OPSS Granular A Crushed Stone	150 mm
Granular Subbase Course, OPSS Granular B Type II	450 mm
Subgrade: Either in situ soils, engineered fill or OPSS Granular B Type I or II material placed over in situ soil	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. 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 II material. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

The subject site is considered satisfactory for a proposed residential development in this geotechnical investigation, where it was expected that the proposed residential buildings would be founded by conventional shallow footings placed on the shale bedrock surface.

9.2 2021 PHASE I ENVIRONMENTAL SITE ASSESSMENT (ESA)

An update to the Phase I ESA (originally done in 2013) was completed by Paterson Group on November 10, 2021 (see **Appendix E.2**) to meet the requirements of MECP O.Reg. 153/04, as amended. References are provided to findings from the the 2013 Phase I ESA (Subsection **9.2.1**), as well as the latest 2020 Phase I-II ESA (Subsection **9.2.2** and **Appendix E.4**).

A site visit conducted on October 29, 2021, confirming that the site exists as vacant land with a temporary MOD space/trailer on the central west portion of the site as well as a small sea container situated on the northern end of the lot. Three (3) hydro poles are also present on the central portion of the site. The majority of the land is covered in gravel with some low brush and three (3) asphaltic concrete paved laneways situated where the former buildings were once present, fronting Cyrville Road. Site drainage consists primarily of infiltration. The site topography is relatively flat and slightly below the grade of Cyrville Road, while the regional topography slopes down in a southwesterly direction.

No signs of staining or discolouration were observed at the time of the site visit. No obvious signs of fill material were noted on the subject land at the time of the site assessment. No evidence of any above ground or underground storage tanks was noted at the time of the site visit. No areas of ponded water exist. No evidence of current or former railway or spur lines was observed on the Phase I ESA Property at the time of the site visit. No areas of unidentified substances were observed on-site at this time. No PCAs were identified during the site visit.

Paterson Group's recommendation was that a Phase II ESA (see Section **9.3** and **Appendix E.3**) is required for the property.

9.2.1 2013 Phase I Environmental Site Assessment (ESA)

Discussed in the 2021 Phase I ESA, the Phase I ESA conducted in 2013 identified six (6) potentially contaminating activities (PCAs) on- and off-site. These PCAs were considered to result in areas of potential environmental concern (APEC) on the Phase I Property. This includes: the former on-site automotive service garage (APEC 1), the former underground storage tank located east of the garage building (APEC 2); fill material impacted with heavy metals identified during the previous Phase II-ESA (APEC 3); the drycleaning establishment at 1060 Ogilvie Road (APEC 4), the former retail fuel outlet at 1150 Cyrville Road (APEC 5) and the former retail fuel outlet at 1098 Ogilvie Road (APEC 6).

Additionally, a previous subsurface environmental investigation conducted on the subject site by Paterson in September of 2011 identified soil and groundwater impacted with PHCs, as well as heavy metal impacted fill material. A concentration of chloroform marginally exceeding the MECP Table 7 Commercial Standards, was also identified in the groundwater in the vicinity of the former tank nest. The chloroform is considered to have been the result of municipal water introduced to the borehole during coring operations and is not considered to pose a concern to the subject property.

9.2.2 2020 Phase I-II Environmental Site Assessment (ESA)

A Phase I and II Environmental Site Assessment (ESA) for the site was prepared by Paterson Group Inc.on March 26, 2020, commissioned by Mark Motors, and Main and Main Developments, for the subject property on 1125 to 1149 Cyrville Road. This ESA report is referenced in **Appendix E.4.**

The purpose of the Phase I-II ESA was to follow-up on materials and groundwater information at previously identified areas. Namely, the south-central portion of the site with metal impacted fill material, and the northwestern portion of the site with PHC impacted soil, surrounding the former underground storage tank nest.

Four boreholes (BH3-20 to BH6-20) were placed on the property on February 21, 2020, with depths ranging from 4.3 to 5.8 m below the existing grade. BH3-20 is firstly located at the south-central portion of the site and was equipped as a groundwater monitoring well. The three remaining boreholes (BH4-20, BH5-20, and BH6-20) were drilled at the northwestern location, with BH4-20 and BH6-20 equipped with groundwater monitoring well instrumentation.

For the groundwater samples, no visual or olfactory signs of hydrocarbon contamination were noted in the groundwater. No PHC or VOC parameters were identified above the laboratory method detection limits in any of the groundwater samples analyzed. As such, the test results complied with MECP Table 7 standards for residential land use.

No unusual olfactory observations were noted in the soil samples from the four boreholes. The BTEX and PHC parameter concentrations identified in soil sample analyzed from BH4-20 complied with MECP Table 7 standards for residential land use. The toluene, xylene and PHC (F1-F3) concentrations identified in the sample analyzed from BH5 exceed the MECP Table 7 standards for residential land use. The metal parameter concentrations identified in the soil samples analyzed from BH4-20 and BH6-20 complied with MECP Table 7 standards for residential land use. However, some of the metal parameter concentrations identified in the soil samples analyzed from BH3-20 and BH5-20 are in excess of MECP Table 7 standards for residential land use.

With impacted soils exceeding MECP Table 7 standards at both locations, the source of the impacted soil is suspected to be in relation to the historical use of the subject lands. Based on the very limited groundwater analytical results, a soil remedial program was not deemed critical at the time of the ESA, with undertaking during site redevelopment is acceptable. Once the remedial program commences, it is recommended that the impacted soil be transferred to an approved waste disposal facility under the guidance of Paterson's environmental field personnel. Notably, surplus soil, which contains contaminant concentrations meeting the subject property standards but are in excess of MECP Table 1 (background) standards, will have to be disposed of at an approved waste disposal facility.

9.2.3 2020 Remedial Action Plan

Shown in **Appendix E.5**, Paterson Group Inc. proposed a remedial action plan for the subject site on February 12, 2020, addressing previously identified locations, namely the south-central portion of the site with metal impacted fill material, and the northwestern portion of the site with PHC impacted soil, surrounding the former underground storage tank nest. The proposed remedial program involves a full depth approach, which will excavate all hydrocarbon and/or heavy metal impacted soil and bedrock from within the boundaries of the subject site. Any free product that may be encountered in the groundwater within the excavation would be pumped by an MECP licensed pumping contractor for off-site transfer and disposal.

In the northwest portion of the site, where PHC impacted soils were identified in a previous study, a portable treatment system would be installed to treat on-site accumulated groundwater by means of granular activated carbon. The groundwater treatment system will consist of one unit and will remain in place until the on-site groundwater concentrations are in compliance with both the MECP Table 7 standards and City of Ottawa sewer discharge standards.

Upon completion, there would be a confirmatory sampling program to ensure that the site meets MECP Table 7 standards. Finally, backfills would occur at the excavations, using clean excavated material if deemed geotechnically suitable, or with OPSS Granular B Type II crushed stone as engineered fill up to the underside of the pavement structure.

9.3 2021 PHASE II ENVIRONMENTAL SITE ASSESSMENT (ESA)

Attached in **Appendix E.3**, the Phase II ESA was prepared by Paterson Group on November 12, 2021 to address potentially contaminating activities (PCAs) that were identified during the previous Phase I ESAs and Phase I ESA Update and considered to result in areas of potential environmental concern (APECs) on the subject Property. The subsurface investigations conducted for this Phase II ESA consisted of three (3) field drilling programs that were conducted in 2007, 2011 and 2020. The 2007 field program consisted of drilling eight (8) boreholes (BH1-07 through BH8-07) in the immediate area of the former garage and underground storage tank (UST). No monitoring wells were installed during the 2007 program. The 2011 field program consisted of drilling 12 boreholes (BH1 through BH12), three (3) of which were completed as groundwater monitoring wells (BH1, BH2 and BH12). The 2020 field program consisted of drilling four (4) boreholes, three (3) of which were completed as groundwater monitoring wells (BH3-20, BH4-20 and BH6-20).

The general soil profile encountered during the field programs consisted of an asphaltic concrete pavement structure, topsoil or fill (crushed stones), followed by a fill material consisting of silty sand with some gravel or silty clay with sand, underlain by topsoil or silty clay, followed by silty sand with shale fragments, underlain by shale bedrock.

Nineteen (19) soil samples were submitted for laboratory analysis of benzene, toluene, ethylbenzene and xylenes (BTEX), petroleum hydrocarbons (PHCs, Fractions F1-F4, volatile organic compounds (VOCs) and/or metals (including lead (Pb), mercury (Hg) and hexavalent chromium (CrVI)). BTEX and PHC concentrations above the MECP Table 7 Residential Standards were identified in the soil samples in the immediate area of the former UST nest. Metal concentrations above the selected MECP standards were generally identified in the fill material on the south-central portion of the site. All other soil samples complied with the MECP Table 7 Residential Standards.

Groundwater samples from monitoring wells BH1, BH2, BH12, BH3-20, BH4-20 and BH6-20 were collected during the August 2011, March 2020 and November 2021 sampling events. No free product or petroleum hydrocarbon sheen was noted on the purge water during the groundwater sampling events. All groundwater samples were analyzed for BTEX, PHC and VOCs, with the exception of the groundwater sample BH3-20 from the November 3, 2021 collection.

Concentrations of BTEX, PHCs and 1-4 dichlorobenzene exceeding the MECP Table 7 Standards were identified in the immediate area of the former UST nest. All other groundwater samples complied with the selected MECP Standards. Benzene tested from the BH3-20 sample was marginally in excess of the standard, while the duplicate sample concentration indicated compliance with the standard due to sediment. Paterson recommends the groundwater at BH3-20 should be retested for confirmatory purposes.

Based on the findings of the Phase II ESA, Paterson recommended that a soil and groundwater remediation program be carried out at the Phase II Property. The remediation should be completed in conjunction with the construction excavation. It is anticipated that the impacted groundwater will be removed in conjunction with the excavation and removal of the impacted soil and upper levels of the underlying bedrock.

Prior to remedial activities, it is recommended that a representative sample of impacted soil be submitted for a leachate analysis in accordance with O.Reg. 347/558, as required for disposal at an approved landfill site. It is recommended that Paterson personnel be on-site at the time of the remedial activities to direct excavation and segregation of impacted soil, and to collect additional delineation and confirmatory soil samples as required in accordance with O.Reg. 153/04 to support the filing of a Record of Site Condition. Excess soil requiring off-site disposal during construction must be managed in accordance with Ontario Regulation 406/19: On-site and Excess Soil Management.

Paterson also recommends that the groundwater monitoring wells be maintained for future sampling purposes. The monitoring wells are registered with the MECP under Ontario Regulation 903 (Ontario Water Resources Act). However, the wells would require decommission according to this regulation, if they are determined to not be of use in the future, or will be destroyed during future construction activities.

10.0 APPROVALS/PERMITS

A Ministry of the Environment, Conservation and Parks (MECP) Environmental Compliance Approval (ECA) is not expected to be required for the remainder of the subject site as the site is under singular, private ownership, is not within industrial lands, and does not discharge to a combined sewer.

If the ground or surface water volumes being pumped during the construction phase are between 50,000 and 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 preparation of the Water Taking and Discharge Plan by a Qualified Person as stipulated under O.Reg. 63/16. A Permit to Take Water (PTTW) through the MECP would be required for dewatering in excess of 400,000 L/day, which is unlikely for this site. However, if a PTTW is required, at least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP. If blasting is used to remove the bedrock as part of the excavation for the building foundation, prior approval is required from the owners/operators of any water storage reservoir, pumping station, and water works transformer station within 200 m of the site.

11.0 CONCLUSIONS

11.1 POTABLE WATER SERVICING

The proposed 6 and 12-storey residential high-rise buildings will be serviced by the existing 250mm diameter watermain on Cyrville Road. Water demand calculation was based on a demand rate of 280 L/cap/day for residential units and 28,000 L/ha/day for commercial and amenity space. As such, total demands for the development are 2.32L/s, 5.68L/s, and 12.41L/s, respectively for average day, maximum day, and peak hour flow. The fire flow requirement was calculated in accordance with Fire Underwriters Survey (FUS) and determined to be approximately 12,000 L/min (200 L/s) for Building A and 8,000 L/min (133.3 L/s) for Building B. It is anticipated that the building will be sprinklered, with final sprinkler design to conform to the NFPA 13 standard. A booster pump, to be designed by the buildings' mechanical engineer, will be required to maintain minimum required pressures for the upper storeys.

11.2 WASTEWATER SERVICING

The estimated sanitary peak flow for the site is 8.78L/s for a projected population of 675 people, this site will be serviced by an existing 375 mm diameter PVC sanitary sewer flowing westward on Cyrville Road. A 200 mm diameter service pipe will firstly direct sanitary flows from the 12-storey Building B to the 6-storey Building A. The development's combined sanitary flows will be routed through a single 200 mm service lateral southward along the private access road and connect to the existing Cyrville Road sanitary sewer.

The proposed sanitary service lateral is sufficiently sized to provide gravity drainage for the site. The floor drains in the underground parking will be connected to the building plumbing system and discharged to the sanitary service lateral through a sump pump. A backflow preventer will be required for the proposed building in accordance with the Ottawa Sewer Design Guide and will be coordinated with the buildings' mechanical engineer.

11.3 STORMWATER MANAGEMENT AND SERVICING

The proposed 0.84 ha re-development area will be serviced by a proposed 600mm diameter concrete storm sewer running east to west on Cyrville Road. A stormwater cistern will attenuate peak flows from the building's roof and outdoor amenity areas. In order to meet the site target release rate for stormwater of 82.0 L/s, on-site storm detention facilities will need to be provided in both buildings. The proposed stormwater cisterns will be serviced by the internal plumbing of the building.

The use of 39.0m^3 and 21m^3 stormwater cisterns within the respective underground parking levels of Buildings A and B are proposed to achieve this end. The stormwater cisterns will be pumped at a controlled rate of no more than 34.00 and 15.00 L/s respectively in Buildings A and B. The stormwater cisterns controlled release rate will be set by a pump to be designed by a mechanical engineer.

A proposed oil/grit separator unit will treat storm runoff from the site to achieve 80% TSS removal. An Imbrium stormceptor EF06 or approved equivalent is recommended for this purpose.

11.4 SITE GRADING AND DRAINAGE

Grading for the site is designed as per City of Ottawa requirements and provides for outlet of emergency overland flow under extreme flood conditions. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

11.5 UTILITIES

Hydro Ottawa, Bell, Rogers, and Enbridge all have existing utility plants in the area, which will be used to service the 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 moved/reconfigured to allow sufficient clearance to the proposed building. The relocation of existing utilities will be coordinated with the individual utility providers upon design circulation.

11.6 APPROVALS/PERMITS

A Ministry of the Environment, Conservation and Parks (MECP) Environmental Compliance Approval (ECA) is not expected to be required for the subject site as the site is under singular, private ownership, is not within industrial lands, and does not discharge to a combined sewer.

A Permit to Take Water (PTTW) may be required if the dewatering during the construction of the underground parking level is expected to exceed 400,000 L/day. No other approval requirements from other regulatory agencies are anticipated. For dewatering activities between 50,000 and 400,000 L/day, registration on the Environmental Activity and Sector Registry (EASR) will be required. If blasting is used to remove the bedrock as part of the excavation for the building foundation, prior approval is required from the owners/operators of any water storage reservoir, pumping station, and water works transformer station within 200 m of the site.