2655 Queensview Drive

Site Servicing and Stormwater Management Report

Stantec

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Westrich Pacific Corp.

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Prepared by:
Stantec Consulting Ltd.

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Stantec Consulting Ltd.

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Prepared by:

Signature

Michael Wu, P.Eng.

Printed Name

Reviewed by:

Signature

Peter Moroz, P.Eng., MBA

Printed Name

Approved by:

Signature

Alyssa Gladish, P.Eng.

Printed Name

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1 Introduction

Stantec Consulting Ltd. (Stantec) has been commissioned by Westrich Pacific Corporation (Westrich) to prepare the following Site Servicing and Stormwater Management Report in support of a Minor Zoning By-Law Amendment (ZBLA) and a Site Plan Control (SPC) application for the proposed development located at 2655 Queensview Drive in the City of Ottawa.

The 0.9 ha site is situated along the north side of Queensview Drive. The site is currently zoned MC20 [3014] S507 and comprises of an existing commercial building, asphalt parking lot, and some vegetated areas. The site is bound by Queensview Drive to the south, existing commercial developments to the east and west, and existing residential development fronting Moncton Road to the north, as shown in the figure.

Figure 1 - Site Location Plan

The proposed redevelopment of the site includes two six-storey medium-rise mixed-use buildings with a total of 338 apartment units, ground floor commercial space, and two levels of underground parking. J+S Architect has provided a Site Plan dated February 2, 2026, which defines the proposed development, while the unit type breakdown is presented in **Table 1.1**.

Table 1.1: Unit Type Breakdown

Unit Type	Building A	Building B	Overall
Studio	-	1	1
1 Bedroom	47	77	124
1 Bedroom + Den	15	18	33
2 Bedroom	82	87	169
3 Bedroom	5	6	11
Total	**149**	**189**	**338**

1.1 Objectives

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 Queensview Drive right of way (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 detailed servicing design of the proposed development. Specific and potential development constraints to be addressed are as follows:

- Potable Water Servicing
    - Estimate water demands to characterize the proposed feed(s) for the development which will be serviced from the existing 300 mm diameter watermain within the Queensview Drive ROW.
    - 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
    - Estimate peak sanitary wastewater discharge for the development, which is to determine the size of the sanitary service lateral to connect to the existing 900 mm diameter sanitary sewer within the Queensview Drive 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.
  - Estimate peak stormwater discharge for the development, which is to determine the size of the proposed storm service lateral which will be connected to the existing 1800 mm x1100 mm storm trunk sewer within the Queensview Drive ROW.

- Prepare a grading plan in accordance with the proposed Site Plan and existing grades. Identify key drainage patterns and grading features.
- Incorporate the relevant geotechnical information, constraints, and recommendations.
- Identify utility site constraints (existing infrastructure removals/accommodations, easements, offsets, etc.)
- Assess the availability of utility plant in proximity of the site
- Identify other approval requirements

Drawing SSP-1 illustrates the proposed internal servicing scheme for the site.

2 Background

Documents referenced in preparing of this stormwater and servicing report for the 2655 Queensview Drive development include:

- *City of Ottawa Water Distribution Design Guidelines* (WDG002) 2nd Edition, City of Ottawa, December 2025.
- *Design Guidelines for Drinking Water Systems*, Ministry of the Environment, Conservation, and Parks (MECP), 2008
- *City of Ottawa Sewer Design Guidelines* (SDG 004) 4th Edition, City of Ottawa, December 2025.
- *Low Impact Development Technical Guidance Report (Draft) – City of Ottawa*, prepared by Dillon Consulting and Aquafor Beech Ltd., November 2019.
- *Low Impact Development Stormwater Management Guidance Manual – Draft for Consultation,* Ministry of the Environment, Conservation and Parks (MECP), January 2022.
- *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
- *Pre-consultation Meeting Minutes – 2655 Queensview Drive (File No.: PC2025-0063),* City of Ottawa, April 2025.
- *Stormwater Management Design Criteria for the Pinecrest Creek/Westboro Area*, City of Ottawa, May 2020.
- *Geotechnical Investigation – Proposed Residential Development, 2655 Queensview Drive,* Paterson Group, November 2025.

3 Water Servicing

3.1 Background

The proposed building is in Pressure Zone 1W of the City of Ottawa's Water Distribution System and an existing 305 mm diameter municipal watermain is available at the site frontage in Queensview Drive. See **Drawing EX-1** for details. The existing 150mm water service to the existing commercial building is to be blanked at the main.

3.2 Design Criteria

3.2.1 Potable Water Demands and Pressures

The water demands are estimated using the City of Ottawa Water Distribution Guidelines (2025) (see detailed calculations in **Appendix A.1**).

Residential Apartment Population Rate

Studio and One Bedroom	1.4 persons / unit
One Bedroom w/ Study and Two Bedroom	2.1 persons / unit
Three Bedroom	3.1 persons / unit

Residential Apartment Demand

Average Daily (AVDY)	280 L/cap/day
Maximum Daily (MXDY)	2.5 × AVDY
Peak Hour (PKHR)	2.2 × MXDY

Commercial Area Demand

Average Daily (AVDY)	28,000 L/gross ha/day
Maximum Daily (MXDY)	1.5 × AVDY
Peak Hour (PKHR)	1.8 × MXDY

Allowable Water Pressure

MXDY Flow	345 kPa (50 psi) to 552 kPa (80 psi)
PKHR Flow Minimum	276 kPa (40 psi)
MXDY + Fire Flow	140 kPa (20 psi)
Maximum Allowable for Occupied Area	552 kPa (80 psi)

3.2.2 Fire Flow and Hydrant Capacity

Detailed fire flow requirements are assessed using the Fire Underwriters Survey (FUS) methodology (2020).

The minimum number of fire hydrants required to meet the fire flow is assessed using Section 18.5 of the National Fire Protection Agency (NFPA) 1 Fire Code document.

- The aggregate fire flow capacity of all fire hydrants within 305m of the building cannot be less than the required fire flow (RFF).
- Table 18.5.4.3 provides the maximum fire flow capacity for which an individual fire hydrant can be credited based on the unobstructed distance from the hydrant to the building face.

Table 3.1: Table 18.5.4.3 of the NFPA 1 - Maximum Fire Hydrant Fire Flow Capacity

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

3.3 Water Demands

3.3.1 Domestic Water Demands

Based on the unit type breakdown in **Table 1.1** and the population densities attributed to each unit type, the proposed building is estimated to have a total population of 635 persons. The calculated domestic water demands for the proposed residential building is summarized in **Table 3.2** and detailed in **Appendix A.1.** Note that decimal places in the table below may appear to be incorrectly summed due to rounding.

Table 3.2: Estimated Water Demands

Type	Population	Area (ha)	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Residential	635	-	2.1	5.1	11.3
Commercial	-	0.9	0.3	0.4	0.8
Total	**635**	**0.9**	**2.4**	**5.6**	**12.1**

3.3.2 Fire Flow Demands

When determined using the Office of the Fire Marshal (OFM) Fire Protection Water Supply Guidelines under the Ontario Building Code, the estimated fire flow for the site equals or exceeds 9,000 L/min (150.0 L/s). Consequently, the Required Fire Flow (RFF) for the Site is estimated using the Fire Underwriters Survey (FUS) methodology.

As per Section 3.2.2.45 of the Ontario Building Code, the FUS estimate is based on the following characteristics of the proposed building, as confirmed through correspondence with the architect attached in **Appendix A.2**.

1. The buildings are to be of Type IV-C – Mass Timber Construction. As a result, for the purpose of the FUS calculation, the 'effective floor area' was determined to be the sum of all floor areas as per page 22 of the FUS 2020.
2. The buildings will be equipped with automatic sprinkler systems, with 100% floor area coverage, that are fully supervised and conform to the NFPA 13 standard.
3. A 2-hour rated firewall is proposed to run through the middle of the buildings (Gridline E for Building A and Gridline F for Building B in the Site Plan), effectively dividing each building in two portions (north and south).

The required fire flow demand was determined to range between 167 L/s (10,000 L/mi) and 233 L/s (14,000 L/min) for the proposed buildings. The worst-case portion of the building was found to be Building A – North with a required fire flow of 233 L/s (14,000 L/min). The proposed 2-hour firewalls are shown in **Drawing SSP-1** and **GP-1**, while detailed fire flow calculations per the FUS methodology are included in **Appendix A.3**.

3.4 Level of Servicing

The estimated domestic water and fire flow demands were submitted to City of Ottawa staff to assess the level of servicing available from the municipal watermain and hydrants within Queensview Drive. **Table 3.3** outlines the boundary conditions received from the City of Ottawa's modelling group on November 24th, 2025 (see **Appendix A.4**).

Table 3.3: Queensview Drive Boundary Conditions

Connection	Queensview Drive
Minimum HGL (m)	107.8
Maximum HGL (m)	115.1
Max. Day + FF (233 L/s) (m)	94.6

3.4.1 Allowable Domestic Pressures

The proposed finished floor elevation of the first floor of both buildings, 67.45 m, will serve as the ground floor elevation for the calculation of the residual pressures at ground level. As calculated from the boundary conditions, the on-site pressures are expected to range from 396 to 467 kPa (57.4 to 67.7 psi) under normal operating conditions, satisfying the design criteria for domestic flows. The mechanical consultant will determine the domestic booster pump requirements to service the upper floors of the building at a later phase of design.

3.4.2 Allowable Fire Flow Pressures

The RFF of 233 L/s (14,000 L/min) combined with the maximum daily domestic demand (MXDY = 5.6 L/s) can be provided by the 305 mm diameter municipal watermain in Queensview Drive and nearby hydrants while maintaining at least the minimum residual pressure of 20 psi. The mechanical consultant will determine fire pump requirements for the proposed sprinkler system at a later phase of design.

3.4.3 Fire Hydrant Coverage

There are three existing fire hydrants in the proximity of the proposed development site, as shown in **Drawing SSP-1.**

According to the NFPA 1 Table 18.5.4.3, the site's RFF can't be met for building A – north, for the three existing fire hydrants in combination. Consequently, a new hydrant, P HYD-01, is proposed within the grassed boulevard on the south side of the Queensview ROW to the south of Building A, as shown in **Drawing SSP-1**. With the additional hydrant, the RFF for each building portion (A-north, A-south, B-north and B-south) can be met by the three nearest fire hydrants in combination. See **Appendix A.5** for fire hydrant coverage table calculations.

The existing fire hydrants have light blue bonnets. Per Table 4.9 of the City of Ottawa Water Design Guidelines these are classified as Class AA fire hydrants and have available flow rates of 1500 GPM (568 L/min) and above.

Ex HYD-01 is located within a distance of 45 m, unobstructed, of each buildings' Siamese (Fire Department) connection per Section 3.2.5.16 of the Ontario Building Code.

3.5 Proposed Water Servicing

The proposed water servicing is shown on **Drawing SSP-1**. The development will be serviced from the 305 mm diameter watermain on Queensview Drive via two 150 mm water service connections to the main, separated by a new isolation valve to facilitate operations and maintenance of the municipal watermain while sustaining adequate/redundant service to the new development. The proposed water services include a valve on the east service on the property line and a district monitoring (DM) Chamber (as per City Standard W3) just inside the property line on the west service. The two water services combine within the private portion of the site, and a single 150mm water service will penetrate the building foundation. One new municipal fire hydrant is proposed to be installed in the south boulevard of Queensview Drive across from the Building A principal entrance. The mechanical consultant will confirm the building internal service size and pumping requirements at a later phase of design.

4 Wastewater Servicing

4.1 Background

The existing sanitary sewer adjacent to the site includes the 900 mm diameter concrete Pinecrest Sanitary Trunk Sewer within the Queensview Drive ROW which drains eastward to the southeast corner of the site, then bends to the north and runs in an easement along the eastern property line of the subject site to the north. See **Drawing EX-1** for details.

4.2 Design Criteria

As outlined in the City of Ottawa SDG 004 and the MECP Design Guidelines for Sewage Works, the following criteria were used to estimate the 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 diameter
- Minimum grade of sanitary sewer service = 1.0% (2.0% preferred)
- Average wastewater generation = 280 L/person/day
- Peak Factor = based on Harmon Equation; maximum of 4.0 (residential)
- Harmon correction factor = 0.8
- Infiltration allowance = 0.33 L/s/ha
- Minimum cover for sewer service connections – 2.0 m
- Population density for studio and one-bedroom apartments – 1.4 persons/apartment
- Population density for two-bedroom apartments – 2.1 persons/apartment

4.3 Wastewater Generation and Servicing Design

Based on the current Site Plan and unit breakdown as shown in Table 1.1, the estimated peak wastewater flow rate generated from the proposed development is summarized in **Table 4.1** below.

Table 4.1: Estimated Peak Wastewater Flow

Residential			Commercial Peak Flow (L/s)	Infiltration (L/s)	Total Peak Flow (L/s)
Population	Peak Factor	Peak Flow (L/s)			
635	3.33	6.9	0.4	0.3	7.6

Detailed sanitary sewage calculations are included in **Appendix B.1**. The anticipated peak wastewater flows for the proposed development were provided to the City of Ottawa staff on February 2, 2026, (see **Appendix B.2** for email correspondence) to evaluate the adequacy of the receiving municipal sanitary sewer system in the vicinity of the site and downstream network. The City noted the 900 mm diameter

Pinecrest Sanitary Trunk Sewer (PSTS) is presently at capacity. The only available capacity that the City can guarantee is the existing sanitary outflow from the site as it currently operates. Furthermore, the City indicates that if the sanitary outflow from the proposed Site Plan application exceeds the existing sanitary outflow from the site, the City may put the development application on hold by imposing development conditions requiring adequate servicing or by adding a holding symbol to the applicable zoning until there is sufficient capacity.

Stantec obtained the Water Utility Bills for the existing site as it currently operates. The average daily water consumption at the facility was calculated to be approximately 17.4 m^3/day. For a fitness gym, the sanitary sewage rates are expected to have high peak flows due to shower usage at peak gym usage times, resembling residential usage trends and resulting in higher peaking factors than standard commercial retail. Considering the peaking factor of 4.0 and the average operation of the facility for 19 hours per day, the peak sanitary sewer flow rate was estimated to be 1.0 L/s.

The City has also identified infiltration flows as a significant source of sanitary sewage in the Pinecrest Sanitary Trunk Sewer catchment area. Consequently, the infiltration flows under the current site conditions are also estimated to be 0.3 L/s, so the overall sanitary peak outflow from the existing site is estimated to be 1.3 L/s. The anticipated peak sanitary outflow from the proposed Site Plan application (7.6 L/s) exceeds the estimated existing sanitary outflow from the site (1.3 L/s). It is acknowledged that the City may add a holding symbol to the applicable zoning until there is sufficient capacity made available in the trunk sewer.

As indicated in the City correspondence, **Appendix B.2,** the Infrastructure Planning Unit noted that a review of the Pinecrest Sanitary Trunk Sewer is needed to identify whether there is any available capacity for redevelopment in the area. The City is currently implementing a flow monitoring program to monitor existing inflow and infiltration (I&I) for the areas draining to the Pinecrest Sanitary Trunk Sewer. Preliminary results from the monitoring are anticipated in the Fall of 2026. Based on the results of the monitoring, further works (projected in 2027) may be required to remove any identified I&I and make additional flows available for developments in the subject area. Westrich has expressed interest in working with the City of Ottawa Staff to facilitate future works that will help to provide adequate sanitary servicing capacity and ultimately lift the holding symbol. On behalf of Westrich, Stantec will liaise with the Infrastructure Planning Unit and City of Ottawa Staff to follow the outcomes of the I&I study and remain apprised of the potential future servicing improvement works.

4.4 Proposed Sanitary Servicing

The proposed sanitary servicing is shown on **Drawing SSP-1**. A 200 mm diameter sanitary service, complete with a monitoring manhole just inside the property line (as per SDG section 4.4.4.7), is required to service the proposed development, and will connect to the PSTS concrete sanitary sewer. The mechanical consultant will confirm the sanitary lateral size at a later phase of design. The lateral is to be gravity-drained, with a sanitary sump pump required for the subsurface levels. The mechanical consultant will confirm and design the sanitary sump pump at a later phase of design.

The sanitary monitoring manhole is provided per City standard S18.1 and the service lateral connection into the existing 900 mm diameter sanitary sewer in Queensview Drive is to follow City Standard S11.

5 Stormwater Management and Servicing

5.1 Background

The existing storm sewer adjacent of the site comprises of the 1800mm×1100 mm (nominal) concrete box storm trunk sewer within the north boulevard of Queensview Drive. The storm trunk sewer drains eastward to the manhole structure located southeast of the site, then drains north through the 2400x1500 mm concrete box trunk sewer, known as the Queensway Terrace Trunk Sewer (QTTS) which runs parallel to the Pinecrest Sanitary Trunk Sewer (PSTS) on the eastern side of the property line before discharging into the storm trunk sewer in Moncton Road to the northwest. Note that this information is based off the May 2024 As-Built Drawings by J. L. Richards (18409p&p12) and does not coincide with what is shown in GeoOttawa. See **Drawing EX-1** for details.

The bulk of the existing site includes storm drainage areas routed to the minor storm system. The existing flat roof of the building drains to the storm catchbasin manhole in the east asphalt parking lot. The onsite parking lots are curbed and drain to the site's internal storm catch basins and sewer system. The minor system appears to connect to the storm trunk sewer in Queensview Drive. The site's overland flow drains into the Queensview ROW. A small portion at the northwest (rear) property drains north towards the adjacent lot and hydro easement to the north. Most of the rear property drains south into the site, including two external areas, EX CIST 6 and EX CIST7. It is assumed the entire site is currently draining uncontrolled. The existing site has an overall grade change (decrease) of approximately 2.2 m from the northwest corner down to the southeast corner.

5.2 Stormwater Management (SWM) Criteria

The current design practices outlined in the SDG (2025), project pre-consultation notes, the Stormwater Management Design Criteria for the Pinecrest Creek/Westboro Area (PWDC), and consultation with City of Ottawa (Pre-con) staff are combined to establish the Stormwater Management (SWM) criteria for the subject site. The following summarizes the criteria, with the source of each criterion indicated in brackets:

General

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

Storm Sewer & Inlet Controls

- Intensity-Duration-Frequency (IDF) information is to be derived from the Meteorological Services of Canada rainfall data. Historical data must be taken from the MacDonald Cartier Airport, collected between 1966 to 1997 (Pre-con).

- Discharge for the site for each storm event up to the 100-year to be restricted to the more stringent of the following:
  - the pre-development 2-year release rate per the SDG; or,
  - the release rate of 33.5 L/s/ha (PWDC).
- The preferred stormwater system outlet for this site is the 1800 × 1100 mm concrete box storm trunk sewer within Queensview Drive (Pre-con).
- Time of Concentration (Tc) should not be less than 10 minutes (SDG).

This site is subject to Low Impact Development (LID) requirements as indicated in the PWDC document. Referring to *Table 3-1: SWM Design Criteria for the Pinecrest Creek/Westboro Study Area,* this site falls under category 7 for Residential Development Requiring Site Plan Control Approval that is Discharging directly to the Ottawa River Parkway (ORP) Pipe. Category 7 has the provisions as follows:

a) Sites with soil infiltration rates ≥ 1mm/hour:

- Runoff volume reduction:
  Minimum on-site retention of the 10mm design storm
- Water Quality (TSS Removal):
  Inherent TSS removal due to on-site retention of the 10mm design storm
- Water Quantity Flood Flow Management:
  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's Sewer Design Guideline

b) Sites with soil infiltration rates ≤ 1mm/hour:

- Runoff volume reduction:
  If the entire property is underlain by native soils with infiltration rates less than 1 mm/hr, no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. A green roof and/or rain harvesting measures could be implemented to provide further runoff volume reduction.
- Water Quality (TSS Removal):
  Inherent TSS removal from on-site retention in landscaped areas.
- Water Quantity Flood Flow Management:
  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's Sewer Design Guideline

Surface Storage & Overland Flow

- Building openings to be a minimum of 0.30 m above the 100-year water level (SDG).
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.35 m (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 (SDG).

Quality Control

- Quality control measures are not required for this site, as the proposed development has no surficial parking areas or other known sources of contaminants.

5.3 Existing Drainage Conditions

The existing site (0.9 ha) presently comprises of an existing commercial building with paved surface parking area and grassed perimeter. The overall site pre-development runoff coefficient was determined to be C=0.67, more than the maximum runoff coefficient of C=0.50 per the City Design Guidelines. Consequently, the maximum C=0.50 was assigned to the site for the purposes of establishing the predevelopment release rates.

The pre-development release rates for the site have been determined using the rational method, 2-year rainfall event IDF curve, and the drainage characteristics identified above. A time of concentration for the pre-development area was determined using the FAA method and found to be below 10 minutes. Consequently, the minimum 10-minute Tc was assigned. The onsite and external areas contributing to the storm sewer outlet total 0.88ha, which is the area used to determine the peak pre-development flow rate as shown in **Table 5.1.** The pre-development peak flow rate is calculated using the rational method as shown in **Appendix C.1** and 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	2-Year Predevelopment Flow Rate (L/s) C=0.50, A=0.88 ha, t_c=10 min
2-Year	33.8

5.4 Stormwater Management Design

The Modified Rational Method was employed to assess the rate and volume of runoff anticipated during post-development rainfall runoff events. The site was subdivided into sub-catchments (subareas) as defined by the proposed grades, spill elevations, and the location, nature, or presence/absence of drainage infrastructure. Currently, there are no inlet control devices (ICD's) on the proposed inlets (catch basins, area drains, etc.) on this site. Each sub-catchment was assigned a runoff coefficient based on the proposed finished surface(s). A summary of subareas and runoff coefficients is provided in **Table 5.2**. Further details can be found in **Appendix C.1**, while **Drawing SD-1** illustrates the proposed sub-catchments.

Table 5.2: Summary of Subcatchment Areas

Catchment Areas	C	A (ha)	Flow Type	Storage Type	Outlet
CIST 1	0.90	0.013	Controlled	Cistern	Queensview Drive Sewer
CIST 2	0.24	0.022	Controlled	Cistern	Queensview Drive Sewer
CIST 3	0.44	0.007	Controlled	Cistern	Queensview Drive Sewer
CIST 4	0.50	0.011	Controlled	Cistern	Queensview Drive Sewer
CIST 5	0.50	0.011	Controlled	Cistern	Queensview Drive Sewer
CIST 6	0.51	0.010	Controlled	Cistern	Queensview Drive Sewer
CIST 7	0.50	0.011	Controlled	Cistern	Queensview Drive Sewer
CIST 8	0.49	0.012	Controlled	Cistern	Queensview Drive Sewer
CIST 9	0.47	0.018	Controlled	Cistern	Queensview Drive Sewer
CIST 10	0.57	0.020	Controlled	Cistern	Queensview Drive Sewer
CIST 11	0.74	0.047	Controlled	Cistern	Queensview Drive Sewer
CIST 12	0.46	0.020	Controlled	Cistern	Queensview Drive Sewer
CIST 13	0.67	0.037	Controlled	Cistern	Queensview Drive Sewer
CIST 14	0.56	0.064	Controlled	Cistern	Queensview Drive Sewer
CIST 15	0.62	0.030	Controlled	Cistern	Queensview Drive Sewer
CIST 16	0.79	0.022	Controlled	Cistern	Queensview Drive Sewer
CIST 17	0.46	0.063	Controlled	Cistern	Queensview Drive Sewer
EX CIST 6	0.90	0.007	Controlled	Cistern	Queensview Drive Sewer
EX CIST 7	0.90	0.012	Controlled	Cistern	Queensview Drive Sewer
R102A	0.90	0.205	Controlled	Rooftop and Cistern	Queensview Drive Sewer
R102B	0.90	0.239	Controlled	Rooftop and Cistern	Queensview Drive Sewer
UNC-1	0.62	0.055	Uncontrolled	-	Queensview Drive ROW
Total Site	**0.73**	**0.934**	-	-	-

5.4.1 Controlled Drainage Areas

As shown in **Table 5.2** and **Drawing SD-1**, the intent of the SWM design is for the majority of the site's stormwater runoff to be directed to the underground parking level(s) of the building to a stormwater cistern with a mechanically pumped (controlled) outlet. Thus, most of the site's stormwater outflow can be controlled, through the cistern storage alone, or through a combination of rooftop and cistern storage. Two external areas EX CIST 6 and EX CIST 7 are tributary to the on-site stormwater management system and

are included in the site's controlled drainage areas. Per existing conditions, UNC-1 continues to drain uncontrolled towards the Queensview Drive ROW and is therefore excluded from the site's total controlled areas attributable to the allowable release rate.

5.4.2 Allowable Release Rate

Per Figure 3 of the Stormwater Management Design Criteria for the Pinecrest Creek/Westboro Area (City, 2020), the site area is tributary to the Ottawa River Parkway storm sewer pipe, where for quantity control, the more stringent of the following is applied:

- Control post-development discharge for all storm events, up to and including the 100-year storm event, to the pre-development 2-year release rate per the Ottawa Sewer Design Guidelines; or,
- Restriction of discharge from the site to 33.5 L/s/ha

Recall from **Table 5.1**, the peak pre-development release rate from the controlled onsite area is determined to be 33.1 L/s. For the 0.86 ha area considered, this equates to 38.4 L/s/ha. The PWDC target of 33.5 L/s/ha is found to be the more stringent of the two criteria. Consequently, the target release rate for the 2655 Queensview Drive site is to be based on the PWDC target.

The two external drainage areas are conveyed to the building cistern to be routed to the storm sewer outlet on Queensview Drive. As external drainage areas, the runoff from these areas does not need to be controlled on the subject site. Consequently, the anticipated peak runoff from these external drainage areas should be added to the allowable release rate for the site. In **Appendix C.1** the PWDC allowable discharge for the controlled onsite area is found to be 28.8 L/s. The 2-year peak runoff rate from the external areas is 3.6 L/s and when added to the PWDC allowable discharge for the site, the target release rate becomes **32.4 L/s** for all events up to and including the 100-year event.

Runoff coefficient values have been increased by 25% for the post-development 100-year storm event based on the SDG.

5.4.3 Quantity Control

The site requires quantity control measures to meet the restrictive stormwater release criteria. It is proposed to be achieved with two methods of storage:

1. The upper roof areas are to provide rooftop storage to a maximum depth of 150 mm via restricted release roof drains. The roof drains will outlet to the building cistern through the building internal plumbing.
2. The stormwater cistern will be located below the underground parking level. The release rate from the cistern will be restricted by mechanically pumping to a maximum release rate. The cistern will outlet through the building storm service to the existing

A spreadsheet using the Modified Rational Method (MRM) was used to design the roof drain release rates and cistern as shown in **Appendix C.1**.

5.4.3.1 Rooftop Storage

It is proposed to retain stormwater on the building rooftops by installing restricted flow roof drains. The MRM calculations assume the roof areas for Building A and B will be equipped with 8 and 9 roof drains, respectively, as per the roof plans provided by the architect. Discharge from the controlled roof drains will be routed by the mechanical consultant through the building's internal plumbing to the proposed building cistern located below the underground parking level.

The roof drain weir data for standard Watts model roof drains complete with Watts Drainage Adjustable Accutrol Weirs have been used in the MRM calculations. The weir data and the roof plan have been used to calculate a practical roof release rate and detention storage volume for the rooftop areas, with 80 % of the roof area assumed to be available for storage. It should be noted that the Watts model roof drains and Accutrol weir have been used as examples only, and that other products may be specified for use, provided that:

- the peak roof release rate is restricted to match the maximum rate of release indicated in **Table 5.3**,
- sufficient roof storage is provided to meet (or exceed) the required volume of detained stormwater indicated in **Table 5.3**, and
- the maximum ponding depth of 150 mm is not exceeded during a design storm event.

The proposed drain release rates and storage volumes have been calculated based on the roof drain weirs at 50% opened setting. Rooftop storage volumes and controlled release rates are calculated in **Appendix C.1** and summarized in **Table 5.3**.

Table 5.3: Roof Subcatchment Areas Stormwater Management

Design Storm	Storage Depth (mm)	Peak Discharge (L/s)	Volume Stored (m^3)
2-Year (R102A)	99	7.5	23.5
100-Year (R102A)	148	10.0	79.3
2-Year (R102B)	99	8.5	27.9
100-Year (R102B)	149	11.3	93.7

5.4.3.1 Stormwater Cistern

The volume of storage required to attenuate rainfall runoff within the building footprint can't be achieved with the rooftop storage areas alone. Additional stormwater detention storage capacity is to be achieved with a stormwater cistern located below the underground parking level. Outflow from the roof drains and runoff from the rear yards, side yards, and central courtyard are to be directed by the building internal mechanical systems to the cistern. The release rate from the cistern will be restricted by mechanically pumping to a maximum release rate. The cistern will outlet to the building storm service.

The stormwater cistern is to provide a minimum active storage volume of 140 m^3 with a maximum controlled release rate of 32.4 L/s. The stormwater cistern is to discharge at the specified controlled release rate using a mechanical pump. Cistern storage volumes and controlled release rates are calculated using the MRM

spreadsheet in **Appendix C.1** and summarized in **Table 5.4** for the 2-year and 100-year return period post-development storm events.

Table 5.4: Proposed Cistern 2- and 100-Year Storage Requirements

Design Storm Return Period	Area ID	Drainage Area (ha)	$Q_{release}$ (L/s)	$V_{required}$ (m^3)	$V_{available}$ (m^3)
2-year	CIST 1 – CIST 17, EX CIT 6, EX CIST 7, R102A, R102B	0.86	32.4	24.1	140.0
100-year				135.4	

The final design of the cistern and outlet pump within the proposed building is to be completed by the mechanical and structural engineers and coordinated with the architect. In the event of a power outage or breakdown of the pump, the cistern will be designed with an emergency overflow spill outlet at or above the 100-year storage level of the cistern. The emergency overflow will spill into a gravity drain designed by the mechanical consultant to convey the 100-year flow rates. The drain inlet/invert will be set to allow flow by gravity through the building internal plumbing system to be wye-teed into the building storm service lateral downstream of the backwater flow prevention valve. The cistern's 100-year storage capacity requirements, allowable release rates, emergency overflow, and gravity drain outlet will need to be coordinated with the structural and mechanical engineering consultants at a later phase of design.

The storm sewer design sheet is attached in **Appendix C.2**.

5.4.3.2 Uncontrolled Areas

There is an uncontrolled subcatchment area, consisting of UNC-1, which drains by overland (surface) flows to the Queensview Drive ROW. The peak post-development release rates from the uncontrolled areas are calculated in **Appendix C.1** and are summarized in **Table 5.5**. Note that the totals may not sum exactly due to rounding.

Table 5.5: Peak Post-Development Uncontrolled Surface Flow Rates

Design Storm	Peak Flow (L/s)
2-Year	7.2
100-Year	21.0

Under existing conditions, approximately 0.18 ha of the site drains uncontrolled towards the Queensview Drive ROW with an imperviousness of C=0.36, as shown in **Drawing EX-1.** This translates to a pre-development peak runoff flow rate of 31.7 L/s to the Queensview Drive ROW during the 100-year storm event. As shown by the peak post-development uncontrolled flow rates in **Table 5.5**, developing the site will reduce uncontrolled discharge to Queensview Drive during the 100-year event by around 34%.

5.4.3.3 Results

The proposed stormwater management plan meets the requirements identified during pre-consultation **Table 5.6** provides a summary of the peak design discharge rates calculated from the MRM analysis, shown in **Appendix C.1**.

Table 5.6: Summary Site 2-Year and 100-Year Event Peak Release Rates

Drainage Areas	2-Year Peak Discharge (L/s)	100-Year Peak Discharge (L/s)
Controlled Cistern to Sewer	32.4	32.4
Uncontrolled Surface Flow to ROW	7.2	21.0

5.4.4 Quality Control

The current Site Plan does not include any onsite uses that would be a significant source of contaminants. Under the PWDC Table 3-1, Condition 7 Water Quality (TSS Removal) requirements, the site will achieve inherent TSS removal from on-site retention in landscaped areas. No additional quality control measures are required.

5.4.5 Low Impact Development

This site is subject to Low Impact Development (LID) requirements as indicated in the PWDC document and as described in **Section 5.2.** The geotechnical investigation on this site (Paterson, Nov. 2025) found that bedrock underlies the site between 1.5m and 4.5m below the existing ground surface. The underground parking structure limits the onsite areas that could be feasible for infiltration on this site. The rear yard was contemplated for an infiltration trench. The boreholes near this area (BH 1-25 and BH 2-25) were found to have fair quality bedrock at depths of 1.60m (66.45m) and 1.47m (65.66) below the existing ground surface, underlain by good to excellent quality bedrock. It is assumed that the soil infiltration rate of the bedrock is less than 1mm/hour. As per the SDG 004, 2025 and the Low Impact Development Technical Guidance Report (2022), these geotechnical conditions are not suitable for an LID infiltration system, since 1.0m clearance between the invert of the infiltration bed and the bedrock cannot be achieved.

Consequently, this site is subject to the runoff volume reduction flow management PWDC Table 3-1, Condition 7(b), sites with soil infiltration rates ≤ 1mm/hour, where no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. On this site, we believe there will be benefits to include amended soil below all landscaped areas, not just the front yard. The extents and specifications for the amended soil will be coordinated with the landscape architect for the next submission. The PWDC Table also recommends the implementation of a green roof and/or rain harvesting measures to provide further runoff volume reduction. A green roof is not

advisable on this site, since the building is a wood frame structure. Rainwater harvesting systems are not currently supported in the site design concept.

5.5 Proposed Stormwater Servicing

The internal stormwater drainage network collecting discharge from the trench drain, area drains throughout the site, the roof drains, and the side yard catch basins will outlet to the stormwater cistern, where stormwater runoff is stored, then mechanically pumped at a controlled release rate to a proposed single 300 mm diameter stormwater building service, complete with full port backwater valve as per City Standard S14.1 and an onsite stormwater monitoring manhole (1200 mm diameter) as per the City of Ottawa SDG. The proposed foundation drain will also outlet to the proposed 300 mm building service, downstream of the full port backwater valve. Stormwater sump(s) and pump(s) are required for the building foundation drain.

The existing grades along the east side of the site and a portion of the adjacent property previously drained west into the subject site, with an existing low point along the property line. To provide positive drainage from the proposed building face, tie-in to the existing grades along the property line, and provide an emergency overland flow route toward the ROW, and a swale is proposed along the east side of the site. The catchbasin located in the east side yard (CB CIST 17) was required to capture and convey the runoff from the proposed drainage swale to the building cistern for quantity control to meet the release rate targets. We understand the construction and development limitations within the City's sanitary easement along the east side of the site and are seeking an exception for the installation of this shallow CB within the easement. The 200mm diameter storm pipe invert is noted as 65.46 m elevation at 1.0m from the building face near the crossing of the sanitary sewer. From the as-built drawings, the top of the sanitary sewer is located at approximately 62.93m. The proposed storm connection will have over two meters of clearance above the top of the sanitary trunk sewer.

Through the stormwater monitor manhole, discharge from the site outlets through a proposed 300 mm diameter storm sewer to the existing 1800 mm × 1100 mm (nominal) box concrete storm trunk sewer in Queensview Drive via a proposed 1829 mm × 2438 mm box manhole. Sizing and design of the service, backwater valves, and the internal drainage system (including the roof drains, roof scuppers, cistern pump, and cistern emergency overflow drain), and confirmation of requirements for the foundation drain sump pump and cistern pump, are to be completed by the mechanical consultant. Design of the stormwater cistern is to be completed by the structural engineer. The proposed stormwater servicing is shown on **Drawings SSP-1** and **SD-1**.

6 Site Grading

The proposed development measures approximately 0.9 ha in area. The existing topography across the site is split by both the existing building and the parking, with the south half sloped south and directing drainage towards the Queensview Drive ROW, while the north half is sloped towards the adjacent properties to the north.

A detailed grading plan (see **Drawing GP-1**) has been prepared to satisfy the stormwater management requirements, as detailed in **Section 5**, allow for positive drainage away from the face of proposed building, and provide minimum cover requirements for water services, storm and sanitary sewers, and footings, where possible. Site grading has been established to provide emergency overland flow routes required for stormwater management in accordance with City of Ottawa SDG.

Queensview Drive is noted to have previously flooded during a 100+ year storm event. The ramp to the underground parking is located at the side of the property with higher elevations, and the ramp has been graded up toward the building for the first 14m to provide protection from the major overland flow anticipated in large storm events. The elevation of the ramp at the grade break is designed to provide over 100mm clearance above the top of curb of the roadway and above the crown of the roadway.

There is a grade difference between the site and the adjacent properties to the west and north. A structural cast in place retaining wall is required at the northwest corner of the site and along the west property line to accommodate the grade difference, as shown in **Drawing GP-1.** The wall has been offset a minimum of 0.15m from the property line. Existing fencing along the north property line and proposed fencing along the west property line is to meet the rail/barrier safety requirements along the wall.

7 Utilities

Overhead (OH) hydro-wires and a natural gas line run parallel to the south side of Queensview Drive, while Bell, cable, and telecom lines run parallel to the north side of the ROW. A hydro corridor with utility poles and overhead wires runs along the west property line (subject to Easement Instrument CR398311) and rear property line (subject to Easement Instrument NS91437). The building has been setback from each corridor in accordance with its respective easement. At the southwest corner of the site there are two communications pedestals on concrete pads. Existing utilities are shown in **Drawing EX-1**.

All utilities within the work area will require relocation during construction. The existing utility poles within the public right of way are to be protected during construction; however, 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.

As the site is surrounded by existing commercial developments, 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.

A HydroOttawa transformer is proposed at the front of the building (to the west of the underground parking ramp). A minimum 3.0 m clearance has been provided from the transformer to all underground services (water, storm, and sanitary). The transformer bollards and a concrete pad are shown in **Drawings SP-1 and GP-1.**

The Pinecrest Sanitary Trunk Sewer passes through the east side of this site, running south to north, nearly parallel with the property line. There is an existing 9.0m easement in favour of the City measured from the east property line into the site. According to the SDG004, 2025, the existing easement is insufficient in width. As a condition of Site Plan Approval, the City requires the transfer of a new 9.0m wide easement that is to be centered over the PSTS and include the area up to the east property line. Refer to the latest Site Plan for the new limits of the easement.

8 Approvals

The proposed development lies on a private site under singular ownership; drains to an approved separated sewer outlet (under the City of Ottawa CLI-ECA); and is not intended to service industrial land or land uses. Therefore, the onsite storage and stormwater management works are exempt from the Ministry of the Environment, Conservation and Parks (MECP) Environmental Compliance Application (ECA) process under Ontario Regulation 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 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. An Erosion Control Plan Concept is provided in **Drawing ECDS-1**. Development and implementation of a detailed erosion and sedimentation control plan is the responsibility of the Contractor at the time of permitting and construction.

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 and slopes as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with geotextiles, geogrid, plastic, or synthetic mulches.
6. Install catch basin inserts or equivalent in all proposed catch basins and catch basin manholes and in all existing catch basins that will receive run-off from the site.
7. A silt fence shall be installed around the perimeter of all and any stockpiles of material to be used or removed from site. (Location to be determined by the Contractor).

8. A visual inspection shall be done daily on sediment control measures and cleaned of any accumulated silt as required. The deposits will be disposed off-site as per the requirements of the contract.

9. Sediment control barriers may only be removed temporarily with approval of Contract Administrator to accommodate construction operations. All affected barriers must be reinstated at night when construction is completed. No removal will occur if there is a significant rainfall event anticipated (>10 mm) unless a new device has been installed to protect existing storm and sanitary sewer systems or downstream watercourses.

10. No refueling or cleaning of equipment is permitted near any existing waterway.

11. The Contractor shall remove sediment control measures when, in the opinion of the Contract Administrator, the measure(s) are no longer required. No control measures shall be permanently removed without prior written authorization from the Contract Administrator.

12. The Contractor shall periodically, or when requested by the Contract Administrator, clean out accumulated sediments as required.

13. The Contractor shall immediately report to the engineer any accidental discharges of sediment material into the watercourse. Appropriate response measures, including any repairs to existing control measures or the implementation of additional control measures, shall be carried out by the Contractor without delay.

14. Install trackout control mats (mud mats) at the entrance/egress to prevent migration of sediment into the public ROW.

15. Stormwater swales to be covered with hydro-seed and mulch.

16. Provide sediment traps and basins during dewatering works.

17. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.

18. 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, at minimum:

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

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

10 Geotechnical Investigation

A geotechnical investigation report was prepared by Paterson Group on November 7, 2025, to provide an assessment of the subsurface conditions found at the site and provide geotechnical design and construction recommendations for the proposed development. Four (4) boreholes, numbered BH 1-25 to BH 4-25, were advanced to a maximum depth of 9.1 metres below the existing ground surface. Excerpts from the geotechnical investigation report are attached in **Appendix D.1**.

The geotechnical investigation identified the following subsurface characteristics:

- Bedrock underlies the site between 1.5m and 4.5m below the existing ground surface.
- Monitoring wells were installed in three (3) boreholes, and a flexible PVC standpipe was installed in BH 1-25. Groundwater was measured on May 27, 2025, and ranged from depths of 1.44 m to 4.48 m below the existing ground surface, which equates to the range of 63.3 m to 65.69 m elevation. Though as groundwater levels are subject to seasonal fluctuations, they could vary at the time of construction.
- No sensitive marine clays were detected on this Site. Consequently, the geotechnical report does not identify any grade raise restrictions.

Based on Paterson's recommendations, the site is suitable for the proposed development. The following recommendations were made for the design and construction:

- Conventional spread footings placed on an undisturbed, compact to very dense glacial till deposit or clean surface sounded bedrock.
- Removal of bedrock and/or large boulders are anticipated to be required for construction and servicing installation.
- Various asphalt pavement structures are recommended for the site, including:
    - Car-Only Parking Areas and Fire-Truck Routes (Table 4)
    - Heavy-Truck Traffic and Loading Areas (Table 5)
    - Car-Only Parking Areas (Podium Deck and Raft Slab), (Table 6)
    - Access Lane, Fire Truck Lane, Ramp, and Heavy Truck Parking Areas (Podium Deck and Raft Slab), (Table 7)

  The asphalt pavement structures are included for reference in **Appendix D.1**.
- A 1.5m thick soil cover or a combination of soil cover in conjunction insulation should be provided over all footings to protect against frost action.
- Due to the varying groundwater elevations measured onsite, foundation drains and waterproofing systems will be required for the proposed structures.
- Due to the proximity of the critical storm and sanitary trunk sewers to this site and the anticipated rock breaking requirements for the underground levels, vibrational monitoring and mitigation measures are recommended, and pre-blast/pre-construction survey requirements are identified.
- Excavation, backfill, and engineered fill specifications are provided.
- Pipe bedding and backfill requirements are provided.
- Site specific sub-excavation requirements, including rock removal provisions are identified.

For further information, please refer to the complete geotechnical report.

11 Closing

This Site Servicing and Stormwater Management report assessed the servicing conditions within and immediately adjacent to the project site and evaluated the feasibility of new stormwater management infrastructure to meet the quantity control objectives for the site. Other SPC design requirements appeared in this report including:

- potable water and fire flow servicing,
- sanitary servicing,
- stormwater servicing,
- grading and drainage,
- utilities considerations,
- approvals,
- best management practices for erosion and sediment control during construction, and
- geotechnical considerations.

This report has demonstrated the proposed site servicing meets the design criteria as outlined during the pre-consultation process and in accordance with the 2025 Design Guidelines. The adjacent municipal infrastructure has been shown to provide adequate servicing for potable water, fire flow servicing, and stormwater servicing; however, it is acknowledged that the City may add a holding symbol to the applicable zoning until there is sufficient capacity made available in the sanitary trunk sewer to receive the anticipated wastewater discharge from this proposed development.

We trust that the information, servicing strategies, and proposed and the proposed stormwater management systems presented in this report are adequate to support the Site Plan Control Application for this infill development. Please contact the author or engineer of record of this report if you have any questions or concerns.

Appendix A Water Servicing

A.1 Water Demand Sheet

A.2 Correspondence with the Architect Confirming Building Construction

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Stantec Consulting Ltd.
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4
stantec.com

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1 Introduction

Stantec Consulting Ltd. (Stantec) has been commissioned by Westrich Pacific Corporation (Westrich) to prepare the following Site Servicing and Stormwater Management Report in support of a Minor Zoning By-Law Amendment (ZBLA) and a Site Plan Control (SPC) application for the proposed development located at 2655 Queensview Drive in the City of Ottawa.

The 0.9 ha site is situated along the north side of Queensview Drive. The site is currently zoned MC20 [3014] S507 and comprises of an existing commercial building, asphalt parking lot, and some vegetated areas. The site is bound by Queensview Drive to the south, existing commercial developments to the east and west, and existing residential development fronting Moncton Road to the north, as shown in the figure.

Figure 1 - Site Location Plan

The proposed redevelopment of the site includes two six-storey medium-rise mixed-use buildings with a total of 338 apartment units, ground floor commercial space, and two levels of underground parking. J+S Architect has provided a Site Plan dated February 2, 2026, which defines the proposed development, while the unit type breakdown is presented in **Table 1.1**.

Table 1.1: Unit Type Breakdown

Unit Type	Building A	Building B	Overall
Studio	-	1	1
1 Bedroom	47	77	124
1 Bedroom + Den	15	18	33
2 Bedroom	82	87	169
3 Bedroom	5	6	11
Total	**149**	**189**	**338**

1.1 Objectives

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 Queensview Drive right of way (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 detailed servicing design of the proposed development. Specific and potential development constraints to be addressed are as follows:

- Potable Water Servicing
    - Estimate water demands to characterize the proposed feed(s) for the development which will be serviced from the existing 300 mm diameter watermain within the Queensview Drive ROW.
    - 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
    - Estimate peak sanitary wastewater discharge for the development, which is to determine the size of the sanitary service lateral to connect to the existing 900 mm diameter sanitary sewer within the Queensview Drive 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.
  - Estimate peak stormwater discharge for the development, which is to determine the size of the proposed storm service lateral which will be connected to the existing 1800 mm x1100 mm storm trunk sewer within the Queensview Drive ROW.

- Prepare a grading plan in accordance with the proposed Site Plan and existing grades. Identify key drainage patterns and grading features.
- Incorporate the relevant geotechnical information, constraints, and recommendations.
- Identify utility site constraints (existing infrastructure removals/accommodations, easements, offsets, etc.)
- Assess the availability of utility plant in proximity of the site
- Identify other approval requirements

Drawing SSP-1 illustrates the proposed internal servicing scheme for the site.

2 Background

Documents referenced in preparing of this stormwater and servicing report for the 2655 Queensview Drive development include:

- *City of Ottawa Water Distribution Design Guidelines* (WDG002) 2nd Edition, City of Ottawa, December 2025.
- *Design Guidelines for Drinking Water Systems*, Ministry of the Environment, Conservation, and Parks (MECP), 2008
- *City of Ottawa Sewer Design Guidelines* (SDG 004) 4th Edition, City of Ottawa, December 2025.
- *Low Impact Development Technical Guidance Report (Draft) – City of Ottawa*, prepared by Dillon Consulting and Aquafor Beech Ltd., November 2019.
- *Low Impact Development Stormwater Management Guidance Manual – Draft for Consultation,* Ministry of the Environment, Conservation and Parks (MECP), January 2022.
- *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
- *Pre-consultation Meeting Minutes – 2655 Queensview Drive (File No.: PC2025-0063),* City of Ottawa, April 2025.
- *Stormwater Management Design Criteria for the Pinecrest Creek/Westboro Area*, City of Ottawa, May 2020.
- *Geotechnical Investigation – Proposed Residential Development, 2655 Queensview Drive,* Paterson Group, November 2025.

3 Water Servicing

3.1 Background

The proposed building is in Pressure Zone 1W of the City of Ottawa's Water Distribution System and an existing 305 mm diameter municipal watermain is available at the site frontage in Queensview Drive. See **Drawing EX-1** for details. The existing 150mm water service to the existing commercial building is to be blanked at the main.

3.2 Design Criteria

3.2.1 Potable Water Demands and Pressures

The water demands are estimated using the City of Ottawa Water Distribution Guidelines (2025) (see detailed calculations in **Appendix A.1**).

Residential Apartment Population Rate

Studio and One Bedroom	1.4 persons / unit
One Bedroom w/ Study and Two Bedroom	2.1 persons / unit
Three Bedroom	3.1 persons / unit

Residential Apartment Demand

Average Daily (AVDY)	280 L/cap/day
Maximum Daily (MXDY)	2.5 × AVDY
Peak Hour (PKHR)	2.2 × MXDY

Commercial Area Demand

Average Daily (AVDY)	28,000 L/gross ha/day
Maximum Daily (MXDY)	1.5 × AVDY
Peak Hour (PKHR)	1.8 × MXDY

Allowable Water Pressure

MXDY Flow	345 kPa (50 psi) to 552 kPa (80 psi)
PKHR Flow Minimum	276 kPa (40 psi)
MXDY + Fire Flow	140 kPa (20 psi)
Maximum Allowable for Occupied Area	552 kPa (80 psi)

3.2.2 Fire Flow and Hydrant Capacity

Detailed fire flow requirements are assessed using the Fire Underwriters Survey (FUS) methodology (2020).

The minimum number of fire hydrants required to meet the fire flow is assessed using Section 18.5 of the National Fire Protection Agency (NFPA) 1 Fire Code document.

- The aggregate fire flow capacity of all fire hydrants within 305m of the building cannot be less than the required fire flow (RFF).
- Table 18.5.4.3 provides the maximum fire flow capacity for which an individual fire hydrant can be credited based on the unobstructed distance from the hydrant to the building face.

Table 3.1: Table 18.5.4.3 of the NFPA 1 - Maximum Fire Hydrant Fire Flow Capacity

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

3.3 Water Demands

3.3.1 Domestic Water Demands

Based on the unit type breakdown in **Table 1.1** and the population densities attributed to each unit type, the proposed building is estimated to have a total population of 635 persons. The calculated domestic water demands for the proposed residential building is summarized in **Table 3.2** and detailed in **Appendix A.1.** Note that decimal places in the table below may appear to be incorrectly summed due to rounding.

Table 3.2: Estimated Water Demands

Type	Population	Area (ha)	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Residential	635	-	2.1	5.1	11.3
Commercial	-	0.9	0.3	0.4	0.8
Total	**635**	**0.9**	**2.4**	**5.6**	**12.1**

3.3.2 Fire Flow Demands

When determined using the Office of the Fire Marshal (OFM) Fire Protection Water Supply Guidelines under the Ontario Building Code, the estimated fire flow for the site equals or exceeds 9,000 L/min (150.0 L/s). Consequently, the Required Fire Flow (RFF) for the Site is estimated using the Fire Underwriters Survey (FUS) methodology.

As per Section 3.2.2.45 of the Ontario Building Code, the FUS estimate is based on the following characteristics of the proposed building, as confirmed through correspondence with the architect attached in **Appendix A.2**.

1. The buildings are to be of Type IV-C – Mass Timber Construction. As a result, for the purpose of the FUS calculation, the 'effective floor area' was determined to be the sum of all floor areas as per page 22 of the FUS 2020.
2. The buildings will be equipped with automatic sprinkler systems, with 100% floor area coverage, that are fully supervised and conform to the NFPA 13 standard.
3. A 2-hour rated firewall is proposed to run through the middle of the buildings (Gridline E for Building A and Gridline F for Building B in the Site Plan), effectively dividing each building in two portions (north and south).

The required fire flow demand was determined to range between 167 L/s (10,000 L/mi) and 233 L/s (14,000 L/min) for the proposed buildings. The worst-case portion of the building was found to be Building A – North with a required fire flow of 233 L/s (14,000 L/min). The proposed 2-hour firewalls are shown in **Drawing SSP-1** and **GP-1**, while detailed fire flow calculations per the FUS methodology are included in **Appendix A.3**.

3.4 Level of Servicing

The estimated domestic water and fire flow demands were submitted to City of Ottawa staff to assess the level of servicing available from the municipal watermain and hydrants within Queensview Drive. **Table 3.3** outlines the boundary conditions received from the City of Ottawa's modelling group on November 24th, 2025 (see **Appendix A.4**).

Table 3.3: Queensview Drive Boundary Conditions

Connection	Queensview Drive
Minimum HGL (m)	107.8
Maximum HGL (m)	115.1
Max. Day + FF (233 L/s) (m)	94.6

3.4.1 Allowable Domestic Pressures

The proposed finished floor elevation of the first floor of both buildings, 67.45 m, will serve as the ground floor elevation for the calculation of the residual pressures at ground level. As calculated from the boundary conditions, the on-site pressures are expected to range from 396 to 467 kPa (57.4 to 67.7 psi) under normal operating conditions, satisfying the design criteria for domestic flows. The mechanical consultant will determine the domestic booster pump requirements to service the upper floors of the building at a later phase of design.

3.4.2 Allowable Fire Flow Pressures

The RFF of 233 L/s (14,000 L/min) combined with the maximum daily domestic demand (MXDY = 5.6 L/s) can be provided by the 305 mm diameter municipal watermain in Queensview Drive and nearby hydrants while maintaining at least the minimum residual pressure of 20 psi. The mechanical consultant will determine fire pump requirements for the proposed sprinkler system at a later phase of design.

3.4.3 Fire Hydrant Coverage

There are three existing fire hydrants in the proximity of the proposed development site, as shown in **Drawing SSP-1.**

According to the NFPA 1 Table 18.5.4.3, the site's RFF can't be met for building A – north, for the three existing fire hydrants in combination. Consequently, a new hydrant, P HYD-01, is proposed within the grassed boulevard on the south side of the Queensview ROW to the south of Building A, as shown in **Drawing SSP-1**. With the additional hydrant, the RFF for each building portion (A-north, A-south, B-north and B-south) can be met by the three nearest fire hydrants in combination. See **Appendix A.5** for fire hydrant coverage table calculations.

The existing fire hydrants have light blue bonnets. Per Table 4.9 of the City of Ottawa Water Design Guidelines these are classified as Class AA fire hydrants and have available flow rates of 1500 GPM (568 L/min) and above.

Ex HYD-01 is located within a distance of 45 m, unobstructed, of each buildings' Siamese (Fire Department) connection per Section 3.2.5.16 of the Ontario Building Code.

3.5 Proposed Water Servicing

The proposed water servicing is shown on **Drawing SSP-1**. The development will be serviced from the 305 mm diameter watermain on Queensview Drive via two 150 mm water service connections to the main, separated by a new isolation valve to facilitate operations and maintenance of the municipal watermain while sustaining adequate/redundant service to the new development. The proposed water services include a valve on the east service on the property line and a district monitoring (DM) Chamber (as per City Standard W3) just inside the property line on the west service. The two water services combine within the private portion of the site, and a single 150mm water service will penetrate the building foundation. One new municipal fire hydrant is proposed to be installed in the south boulevard of Queensview Drive across from the Building A principal entrance. The mechanical consultant will confirm the building internal service size and pumping requirements at a later phase of design.

4 Wastewater Servicing

4.1 Background

The existing sanitary sewer adjacent to the site includes the 900 mm diameter concrete Pinecrest Sanitary Trunk Sewer within the Queensview Drive ROW which drains eastward to the southeast corner of the site, then bends to the north and runs in an easement along the eastern property line of the subject site to the north. See **Drawing EX-1** for details.

4.2 Design Criteria

As outlined in the City of Ottawa SDG 004 and the MECP Design Guidelines for Sewage Works, the following criteria were used to estimate the 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 diameter
- Minimum grade of sanitary sewer service = 1.0% (2.0% preferred)
- Average wastewater generation = 280 L/person/day
- Peak Factor = based on Harmon Equation; maximum of 4.0 (residential)
- Harmon correction factor = 0.8
- Infiltration allowance = 0.33 L/s/ha
- Minimum cover for sewer service connections – 2.0 m
- Population density for studio and one-bedroom apartments – 1.4 persons/apartment
- Population density for two-bedroom apartments – 2.1 persons/apartment

4.3 Wastewater Generation and Servicing Design

Based on the current Site Plan and unit breakdown as shown in Table 1.1, the estimated peak wastewater flow rate generated from the proposed development is summarized in **Table 4.1** below.

Table 4.1: Estimated Peak Wastewater Flow

Residential			Commercial Peak Flow (L/s)	Infiltration (L/s)	Total Peak Flow (L/s)
Population	Peak Factor	Peak Flow (L/s)			
635	3.33	6.9	0.4	0.3	7.6

Detailed sanitary sewage calculations are included in **Appendix B.1**. The anticipated peak wastewater flows for the proposed development were provided to the City of Ottawa staff on February 2, 2026, (see **Appendix B.2** for email correspondence) to evaluate the adequacy of the receiving municipal sanitary sewer system in the vicinity of the site and downstream network. The City noted the 900 mm diameter

Pinecrest Sanitary Trunk Sewer (PSTS) is presently at capacity. The only available capacity that the City can guarantee is the existing sanitary outflow from the site as it currently operates. Furthermore, the City indicates that if the sanitary outflow from the proposed Site Plan application exceeds the existing sanitary outflow from the site, the City may put the development application on hold by imposing development conditions requiring adequate servicing or by adding a holding symbol to the applicable zoning until there is sufficient capacity.

Stantec obtained the Water Utility Bills for the existing site as it currently operates. The average daily water consumption at the facility was calculated to be approximately 17.4 m^3/day. For a fitness gym, the sanitary sewage rates are expected to have high peak flows due to shower usage at peak gym usage times, resembling residential usage trends and resulting in higher peaking factors than standard commercial retail. Considering the peaking factor of 4.0 and the average operation of the facility for 19 hours per day, the peak sanitary sewer flow rate was estimated to be 1.0 L/s.

The City has also identified infiltration flows as a significant source of sanitary sewage in the Pinecrest Sanitary Trunk Sewer catchment area. Consequently, the infiltration flows under the current site conditions are also estimated to be 0.3 L/s, so the overall sanitary peak outflow from the existing site is estimated to be 1.3 L/s. The anticipated peak sanitary outflow from the proposed Site Plan application (7.6 L/s) exceeds the estimated existing sanitary outflow from the site (1.3 L/s). It is acknowledged that the City may add a holding symbol to the applicable zoning until there is sufficient capacity made available in the trunk sewer.

As indicated in the City correspondence, **Appendix B.2,** the Infrastructure Planning Unit noted that a review of the Pinecrest Sanitary Trunk Sewer is needed to identify whether there is any available capacity for redevelopment in the area. The City is currently implementing a flow monitoring program to monitor existing inflow and infiltration (I&I) for the areas draining to the Pinecrest Sanitary Trunk Sewer. Preliminary results from the monitoring are anticipated in the Fall of 2026. Based on the results of the monitoring, further works (projected in 2027) may be required to remove any identified I&I and make additional flows available for developments in the subject area. Westrich has expressed interest in working with the City of Ottawa Staff to facilitate future works that will help to provide adequate sanitary servicing capacity and ultimately lift the holding symbol. On behalf of Westrich, Stantec will liaise with the Infrastructure Planning Unit and City of Ottawa Staff to follow the outcomes of the I&I study and remain apprised of the potential future servicing improvement works.

4.4 Proposed Sanitary Servicing

The proposed sanitary servicing is shown on **Drawing SSP-1**. A 200 mm diameter sanitary service, complete with a monitoring manhole just inside the property line (as per SDG section 4.4.4.7), is required to service the proposed development, and will connect to the PSTS concrete sanitary sewer. The mechanical consultant will confirm the sanitary lateral size at a later phase of design. The lateral is to be gravity-drained, with a sanitary sump pump required for the subsurface levels. The mechanical consultant will confirm and design the sanitary sump pump at a later phase of design.

The sanitary monitoring manhole is provided per City standard S18.1 and the service lateral connection into the existing 900 mm diameter sanitary sewer in Queensview Drive is to follow City Standard S11.

5 Stormwater Management and Servicing

5.1 Background

The existing storm sewer adjacent of the site comprises of the 1800mm×1100 mm (nominal) concrete box storm trunk sewer within the north boulevard of Queensview Drive. The storm trunk sewer drains eastward to the manhole structure located southeast of the site, then drains north through the 2400x1500 mm concrete box trunk sewer, known as the Queensway Terrace Trunk Sewer (QTTS) which runs parallel to the Pinecrest Sanitary Trunk Sewer (PSTS) on the eastern side of the property line before discharging into the storm trunk sewer in Moncton Road to the northwest. Note that this information is based off the May 2024 As-Built Drawings by J. L. Richards (18409p&p12) and does not coincide with what is shown in GeoOttawa. See **Drawing EX-1** for details.

The bulk of the existing site includes storm drainage areas routed to the minor storm system. The existing flat roof of the building drains to the storm catchbasin manhole in the east asphalt parking lot. The onsite parking lots are curbed and drain to the site's internal storm catch basins and sewer system. The minor system appears to connect to the storm trunk sewer in Queensview Drive. The site's overland flow drains into the Queensview ROW. A small portion at the northwest (rear) property drains north towards the adjacent lot and hydro easement to the north. Most of the rear property drains south into the site, including two external areas, EX CIST 6 and EX CIST7. It is assumed the entire site is currently draining uncontrolled. The existing site has an overall grade change (decrease) of approximately 2.2 m from the northwest corner down to the southeast corner.

5.2 Stormwater Management (SWM) Criteria

The current design practices outlined in the SDG (2025), project pre-consultation notes, the Stormwater Management Design Criteria for the Pinecrest Creek/Westboro Area (PWDC), and consultation with City of Ottawa (Pre-con) staff are combined to establish the Stormwater Management (SWM) criteria for the subject site. The following summarizes the criteria, with the source of each criterion indicated in brackets:

General

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

Storm Sewer & Inlet Controls

- Intensity-Duration-Frequency (IDF) information is to be derived from the Meteorological Services of Canada rainfall data. Historical data must be taken from the MacDonald Cartier Airport, collected between 1966 to 1997 (Pre-con).

- Discharge for the site for each storm event up to the 100-year to be restricted to the more stringent of the following:
  - the pre-development 2-year release rate per the SDG; or,
  - the release rate of 33.5 L/s/ha (PWDC).
- The preferred stormwater system outlet for this site is the 1800 × 1100 mm concrete box storm trunk sewer within Queensview Drive (Pre-con).
- Time of Concentration (Tc) should not be less than 10 minutes (SDG).

This site is subject to Low Impact Development (LID) requirements as indicated in the PWDC document. Referring to *Table 3-1: SWM Design Criteria for the Pinecrest Creek/Westboro Study Area,* this site falls under category 7 for Residential Development Requiring Site Plan Control Approval that is Discharging directly to the Ottawa River Parkway (ORP) Pipe. Category 7 has the provisions as follows:

a) Sites with soil infiltration rates ≥ 1mm/hour:

- Runoff volume reduction:
  Minimum on-site retention of the 10mm design storm
- Water Quality (TSS Removal):
  Inherent TSS removal due to on-site retention of the 10mm design storm
- Water Quantity Flood Flow Management:
  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's Sewer Design Guideline

b) Sites with soil infiltration rates ≤ 1mm/hour:

- Runoff volume reduction:
  If the entire property is underlain by native soils with infiltration rates less than 1 mm/hr, no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. A green roof and/or rain harvesting measures could be implemented to provide further runoff volume reduction.
- Water Quality (TSS Removal):
  Inherent TSS removal from on-site retention in landscaped areas.
- Water Quantity Flood Flow Management:
  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's Sewer Design Guideline

Surface Storage & Overland Flow

- Building openings to be a minimum of 0.30 m above the 100-year water level (SDG).
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.35 m (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 (SDG).

Quality Control

- Quality control measures are not required for this site, as the proposed development has no surficial parking areas or other known sources of contaminants.

5.3 Existing Drainage Conditions

The existing site (0.9 ha) presently comprises of an existing commercial building with paved surface parking area and grassed perimeter. The overall site pre-development runoff coefficient was determined to be C=0.67, more than the maximum runoff coefficient of C=0.50 per the City Design Guidelines. Consequently, the maximum C=0.50 was assigned to the site for the purposes of establishing the predevelopment release rates.

The pre-development release rates for the site have been determined using the rational method, 2-year rainfall event IDF curve, and the drainage characteristics identified above. A time of concentration for the pre-development area was determined using the FAA method and found to be below 10 minutes. Consequently, the minimum 10-minute Tc was assigned. The onsite and external areas contributing to the storm sewer outlet total 0.88ha, which is the area used to determine the peak pre-development flow rate as shown in **Table 5.1.** The pre-development peak flow rate is calculated using the rational method as shown in **Appendix C.1** and 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	2-Year Predevelopment Flow Rate (L/s) C=0.50, A=0.88 ha, t_c=10 min
2-Year	33.8

5.4 Stormwater Management Design

The Modified Rational Method was employed to assess the rate and volume of runoff anticipated during post-development rainfall runoff events. The site was subdivided into sub-catchments (subareas) as defined by the proposed grades, spill elevations, and the location, nature, or presence/absence of drainage infrastructure. Currently, there are no inlet control devices (ICD's) on the proposed inlets (catch basins, area drains, etc.) on this site. Each sub-catchment was assigned a runoff coefficient based on the proposed finished surface(s). A summary of subareas and runoff coefficients is provided in **Table 5.2**. Further details can be found in **Appendix C.1**, while **Drawing SD-1** illustrates the proposed sub-catchments.

Table 5.2: Summary of Subcatchment Areas

Catchment Areas	C	A (ha)	Flow Type	Storage Type	Outlet
CIST 1	0.90	0.013	Controlled	Cistern	Queensview Drive Sewer
CIST 2	0.24	0.022	Controlled	Cistern	Queensview Drive Sewer
CIST 3	0.44	0.007	Controlled	Cistern	Queensview Drive Sewer
CIST 4	0.50	0.011	Controlled	Cistern	Queensview Drive Sewer
CIST 5	0.50	0.011	Controlled	Cistern	Queensview Drive Sewer
CIST 6	0.51	0.010	Controlled	Cistern	Queensview Drive Sewer
CIST 7	0.50	0.011	Controlled	Cistern	Queensview Drive Sewer
CIST 8	0.49	0.012	Controlled	Cistern	Queensview Drive Sewer
CIST 9	0.47	0.018	Controlled	Cistern	Queensview Drive Sewer
CIST 10	0.57	0.020	Controlled	Cistern	Queensview Drive Sewer
CIST 11	0.74	0.047	Controlled	Cistern	Queensview Drive Sewer
CIST 12	0.46	0.020	Controlled	Cistern	Queensview Drive Sewer
CIST 13	0.67	0.037	Controlled	Cistern	Queensview Drive Sewer
CIST 14	0.56	0.064	Controlled	Cistern	Queensview Drive Sewer
CIST 15	0.62	0.030	Controlled	Cistern	Queensview Drive Sewer
CIST 16	0.79	0.022	Controlled	Cistern	Queensview Drive Sewer
CIST 17	0.46	0.063	Controlled	Cistern	Queensview Drive Sewer
EX CIST 6	0.90	0.007	Controlled	Cistern	Queensview Drive Sewer
EX CIST 7	0.90	0.012	Controlled	Cistern	Queensview Drive Sewer
R102A	0.90	0.205	Controlled	Rooftop and Cistern	Queensview Drive Sewer
R102B	0.90	0.239	Controlled	Rooftop and Cistern	Queensview Drive Sewer
UNC-1	0.62	0.055	Uncontrolled	-	Queensview Drive ROW
Total Site	**0.73**	**0.934**	-	-	-

5.4.1 Controlled Drainage Areas

As shown in **Table 5.2** and **Drawing SD-1**, the intent of the SWM design is for the majority of the site's stormwater runoff to be directed to the underground parking level(s) of the building to a stormwater cistern with a mechanically pumped (controlled) outlet. Thus, most of the site's stormwater outflow can be controlled, through the cistern storage alone, or through a combination of rooftop and cistern storage. Two external areas EX CIST 6 and EX CIST 7 are tributary to the on-site stormwater management system and

are included in the site's controlled drainage areas. Per existing conditions, UNC-1 continues to drain uncontrolled towards the Queensview Drive ROW and is therefore excluded from the site's total controlled areas attributable to the allowable release rate.

5.4.2 Allowable Release Rate

Per Figure 3 of the Stormwater Management Design Criteria for the Pinecrest Creek/Westboro Area (City, 2020), the site area is tributary to the Ottawa River Parkway storm sewer pipe, where for quantity control, the more stringent of the following is applied:

- Control post-development discharge for all storm events, up to and including the 100-year storm event, to the pre-development 2-year release rate per the Ottawa Sewer Design Guidelines; or,
- Restriction of discharge from the site to 33.5 L/s/ha

Recall from **Table 5.1**, the peak pre-development release rate from the controlled onsite area is determined to be 33.1 L/s. For the 0.86 ha area considered, this equates to 38.4 L/s/ha. The PWDC target of 33.5 L/s/ha is found to be the more stringent of the two criteria. Consequently, the target release rate for the 2655 Queensview Drive site is to be based on the PWDC target.

The two external drainage areas are conveyed to the building cistern to be routed to the storm sewer outlet on Queensview Drive. As external drainage areas, the runoff from these areas does not need to be controlled on the subject site. Consequently, the anticipated peak runoff from these external drainage areas should be added to the allowable release rate for the site. In **Appendix C.1** the PWDC allowable discharge for the controlled onsite area is found to be 28.8 L/s. The 2-year peak runoff rate from the external areas is 3.6 L/s and when added to the PWDC allowable discharge for the site, the target release rate becomes **32.4 L/s** for all events up to and including the 100-year event.

Runoff coefficient values have been increased by 25% for the post-development 100-year storm event based on the SDG.

5.4.3 Quantity Control

The site requires quantity control measures to meet the restrictive stormwater release criteria. It is proposed to be achieved with two methods of storage:

1. The upper roof areas are to provide rooftop storage to a maximum depth of 150 mm via restricted release roof drains. The roof drains will outlet to the building cistern through the building internal plumbing.
2. The stormwater cistern will be located below the underground parking level. The release rate from the cistern will be restricted by mechanically pumping to a maximum release rate. The cistern will outlet through the building storm service to the existing

A spreadsheet using the Modified Rational Method (MRM) was used to design the roof drain release rates and cistern as shown in **Appendix C.1**.

5.4.3.1 Rooftop Storage

It is proposed to retain stormwater on the building rooftops by installing restricted flow roof drains. The MRM calculations assume the roof areas for Building A and B will be equipped with 8 and 9 roof drains, respectively, as per the roof plans provided by the architect. Discharge from the controlled roof drains will be routed by the mechanical consultant through the building's internal plumbing to the proposed building cistern located below the underground parking level.

The roof drain weir data for standard Watts model roof drains complete with Watts Drainage Adjustable Accutrol Weirs have been used in the MRM calculations. The weir data and the roof plan have been used to calculate a practical roof release rate and detention storage volume for the rooftop areas, with 80 % of the roof area assumed to be available for storage. It should be noted that the Watts model roof drains and Accutrol weir have been used as examples only, and that other products may be specified for use, provided that:

- the peak roof release rate is restricted to match the maximum rate of release indicated in **Table 5.3**,
- sufficient roof storage is provided to meet (or exceed) the required volume of detained stormwater indicated in **Table 5.3**, and
- the maximum ponding depth of 150 mm is not exceeded during a design storm event.

The proposed drain release rates and storage volumes have been calculated based on the roof drain weirs at 50% opened setting. Rooftop storage volumes and controlled release rates are calculated in **Appendix C.1** and summarized in **Table 5.3**.

Table 5.3: Roof Subcatchment Areas Stormwater Management

Design Storm	Storage Depth (mm)	Peak Discharge (L/s)	Volume Stored (m^3)
2-Year (R102A)	99	7.5	23.5
100-Year (R102A)	148	10.0	79.3
2-Year (R102B)	99	8.5	27.9
100-Year (R102B)	149	11.3	93.7

5.4.3.1 Stormwater Cistern

The volume of storage required to attenuate rainfall runoff within the building footprint can't be achieved with the rooftop storage areas alone. Additional stormwater detention storage capacity is to be achieved with a stormwater cistern located below the underground parking level. Outflow from the roof drains and runoff from the rear yards, side yards, and central courtyard are to be directed by the building internal mechanical systems to the cistern. The release rate from the cistern will be restricted by mechanically pumping to a maximum release rate. The cistern will outlet to the building storm service.

The stormwater cistern is to provide a minimum active storage volume of 140 m^3 with a maximum controlled release rate of 32.4 L/s. The stormwater cistern is to discharge at the specified controlled release rate using a mechanical pump. Cistern storage volumes and controlled release rates are calculated using the MRM

spreadsheet in **Appendix C.1** and summarized in **Table 5.4** for the 2-year and 100-year return period post-development storm events.

Table 5.4: Proposed Cistern 2- and 100-Year Storage Requirements

Design Storm Return Period	Area ID	Drainage Area (ha)	$Q_{release}$ (L/s)	$V_{required}$ (m^3)	$V_{available}$ (m^3)
2-year	CIST 1 – CIST 17, EX CIT 6, EX CIST 7, R102A, R102B	0.86	32.4	24.1	140.0
100-year				135.4	

The final design of the cistern and outlet pump within the proposed building is to be completed by the mechanical and structural engineers and coordinated with the architect. In the event of a power outage or breakdown of the pump, the cistern will be designed with an emergency overflow spill outlet at or above the 100-year storage level of the cistern. The emergency overflow will spill into a gravity drain designed by the mechanical consultant to convey the 100-year flow rates. The drain inlet/invert will be set to allow flow by gravity through the building internal plumbing system to be wye-teed into the building storm service lateral downstream of the backwater flow prevention valve. The cistern's 100-year storage capacity requirements, allowable release rates, emergency overflow, and gravity drain outlet will need to be coordinated with the structural and mechanical engineering consultants at a later phase of design.

The storm sewer design sheet is attached in **Appendix C.2**.

5.4.3.2 Uncontrolled Areas

There is an uncontrolled subcatchment area, consisting of UNC-1, which drains by overland (surface) flows to the Queensview Drive ROW. The peak post-development release rates from the uncontrolled areas are calculated in **Appendix C.1** and are summarized in **Table 5.5**. Note that the totals may not sum exactly due to rounding.

Table 5.5: Peak Post-Development Uncontrolled Surface Flow Rates

Design Storm	Peak Flow (L/s)
2-Year	7.2
100-Year	21.0

Under existing conditions, approximately 0.18 ha of the site drains uncontrolled towards the Queensview Drive ROW with an imperviousness of C=0.36, as shown in **Drawing EX-1.** This translates to a pre-development peak runoff flow rate of 31.7 L/s to the Queensview Drive ROW during the 100-year storm event. As shown by the peak post-development uncontrolled flow rates in **Table 5.5**, developing the site will reduce uncontrolled discharge to Queensview Drive during the 100-year event by around 34%.

5.4.3.3 Results

The proposed stormwater management plan meets the requirements identified during pre-consultation **Table 5.6** provides a summary of the peak design discharge rates calculated from the MRM analysis, shown in **Appendix C.1**.

Table 5.6: Summary Site 2-Year and 100-Year Event Peak Release Rates

Drainage Areas	2-Year Peak Discharge (L/s)	100-Year Peak Discharge (L/s)
Controlled Cistern to Sewer	32.4	32.4
Uncontrolled Surface Flow to ROW	7.2	21.0

5.4.4 Quality Control

The current Site Plan does not include any onsite uses that would be a significant source of contaminants. Under the PWDC Table 3-1, Condition 7 Water Quality (TSS Removal) requirements, the site will achieve inherent TSS removal from on-site retention in landscaped areas. No additional quality control measures are required.

5.4.5 Low Impact Development

This site is subject to Low Impact Development (LID) requirements as indicated in the PWDC document and as described in **Section 5.2.** The geotechnical investigation on this site (Paterson, Nov. 2025) found that bedrock underlies the site between 1.5m and 4.5m below the existing ground surface. The underground parking structure limits the onsite areas that could be feasible for infiltration on this site. The rear yard was contemplated for an infiltration trench. The boreholes near this area (BH 1-25 and BH 2-25) were found to have fair quality bedrock at depths of 1.60m (66.45m) and 1.47m (65.66) below the existing ground surface, underlain by good to excellent quality bedrock. It is assumed that the soil infiltration rate of the bedrock is less than 1mm/hour. As per the SDG 004, 2025 and the Low Impact Development Technical Guidance Report (2022), these geotechnical conditions are not suitable for an LID infiltration system, since 1.0m clearance between the invert of the infiltration bed and the bedrock cannot be achieved.

Consequently, this site is subject to the runoff volume reduction flow management PWDC Table 3-1, Condition 7(b), sites with soil infiltration rates ≤ 1mm/hour, where no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. On this site, we believe there will be benefits to include amended soil below all landscaped areas, not just the front yard. The extents and specifications for the amended soil will be coordinated with the landscape architect for the next submission. The PWDC Table also recommends the implementation of a green roof and/or rain harvesting measures to provide further runoff volume reduction. A green roof is not

advisable on this site, since the building is a wood frame structure. Rainwater harvesting systems are not currently supported in the site design concept.

5.5 Proposed Stormwater Servicing

The internal stormwater drainage network collecting discharge from the trench drain, area drains throughout the site, the roof drains, and the side yard catch basins will outlet to the stormwater cistern, where stormwater runoff is stored, then mechanically pumped at a controlled release rate to a proposed single 300 mm diameter stormwater building service, complete with full port backwater valve as per City Standard S14.1 and an onsite stormwater monitoring manhole (1200 mm diameter) as per the City of Ottawa SDG. The proposed foundation drain will also outlet to the proposed 300 mm building service, downstream of the full port backwater valve. Stormwater sump(s) and pump(s) are required for the building foundation drain.

The existing grades along the east side of the site and a portion of the adjacent property previously drained west into the subject site, with an existing low point along the property line. To provide positive drainage from the proposed building face, tie-in to the existing grades along the property line, and provide an emergency overland flow route toward the ROW, and a swale is proposed along the east side of the site. The catchbasin located in the east side yard (CB CIST 17) was required to capture and convey the runoff from the proposed drainage swale to the building cistern for quantity control to meet the release rate targets. We understand the construction and development limitations within the City's sanitary easement along the east side of the site and are seeking an exception for the installation of this shallow CB within the easement. The 200mm diameter storm pipe invert is noted as 65.46 m elevation at 1.0m from the building face near the crossing of the sanitary sewer. From the as-built drawings, the top of the sanitary sewer is located at approximately 62.93m. The proposed storm connection will have over two meters of clearance above the top of the sanitary trunk sewer.

Through the stormwater monitor manhole, discharge from the site outlets through a proposed 300 mm diameter storm sewer to the existing 1800 mm × 1100 mm (nominal) box concrete storm trunk sewer in Queensview Drive via a proposed 1829 mm × 2438 mm box manhole. Sizing and design of the service, backwater valves, and the internal drainage system (including the roof drains, roof scuppers, cistern pump, and cistern emergency overflow drain), and confirmation of requirements for the foundation drain sump pump and cistern pump, are to be completed by the mechanical consultant. Design of the stormwater cistern is to be completed by the structural engineer. The proposed stormwater servicing is shown on **Drawings SSP-1** and **SD-1**.

6 Site Grading

The proposed development measures approximately 0.9 ha in area. The existing topography across the site is split by both the existing building and the parking, with the south half sloped south and directing drainage towards the Queensview Drive ROW, while the north half is sloped towards the adjacent properties to the north.

A detailed grading plan (see **Drawing GP-1**) has been prepared to satisfy the stormwater management requirements, as detailed in **Section 5**, allow for positive drainage away from the face of proposed building, and provide minimum cover requirements for water services, storm and sanitary sewers, and footings, where possible. Site grading has been established to provide emergency overland flow routes required for stormwater management in accordance with City of Ottawa SDG.

Queensview Drive is noted to have previously flooded during a 100+ year storm event. The ramp to the underground parking is located at the side of the property with higher elevations, and the ramp has been graded up toward the building for the first 14m to provide protection from the major overland flow anticipated in large storm events. The elevation of the ramp at the grade break is designed to provide over 100mm clearance above the top of curb of the roadway and above the crown of the roadway.

There is a grade difference between the site and the adjacent properties to the west and north. A structural cast in place retaining wall is required at the northwest corner of the site and along the west property line to accommodate the grade difference, as shown in **Drawing GP-1.** The wall has been offset a minimum of 0.15m from the property line. Existing fencing along the north property line and proposed fencing along the west property line is to meet the rail/barrier safety requirements along the wall.

7 Utilities

Overhead (OH) hydro-wires and a natural gas line run parallel to the south side of Queensview Drive, while Bell, cable, and telecom lines run parallel to the north side of the ROW. A hydro corridor with utility poles and overhead wires runs along the west property line (subject to Easement Instrument CR398311) and rear property line (subject to Easement Instrument NS91437). The building has been setback from each corridor in accordance with its respective easement. At the southwest corner of the site there are two communications pedestals on concrete pads. Existing utilities are shown in **Drawing EX-1**.

All utilities within the work area will require relocation during construction. The existing utility poles within the public right of way are to be protected during construction; however, 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.

As the site is surrounded by existing commercial developments, 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.

A HydroOttawa transformer is proposed at the front of the building (to the west of the underground parking ramp). A minimum 3.0 m clearance has been provided from the transformer to all underground services (water, storm, and sanitary). The transformer bollards and a concrete pad are shown in **Drawings SP-1 and GP-1.**

The Pinecrest Sanitary Trunk Sewer passes through the east side of this site, running south to north, nearly parallel with the property line. There is an existing 9.0m easement in favour of the City measured from the east property line into the site. According to the SDG004, 2025, the existing easement is insufficient in width. As a condition of Site Plan Approval, the City requires the transfer of a new 9.0m wide easement that is to be centered over the PSTS and include the area up to the east property line. Refer to the latest Site Plan for the new limits of the easement.

8 Approvals

The proposed development lies on a private site under singular ownership; drains to an approved separated sewer outlet (under the City of Ottawa CLI-ECA); and is not intended to service industrial land or land uses. Therefore, the onsite storage and stormwater management works are exempt from the Ministry of the Environment, Conservation and Parks (MECP) Environmental Compliance Application (ECA) process under Ontario Regulation 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 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. An Erosion Control Plan Concept is provided in **Drawing ECDS-1**. Development and implementation of a detailed erosion and sedimentation control plan is the responsibility of the Contractor at the time of permitting and construction.

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 and slopes as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with geotextiles, geogrid, plastic, or synthetic mulches.
6. Install catch basin inserts or equivalent in all proposed catch basins and catch basin manholes and in all existing catch basins that will receive run-off from the site.
7. A silt fence shall be installed around the perimeter of all and any stockpiles of material to be used or removed from site. (Location to be determined by the Contractor).

8. A visual inspection shall be done daily on sediment control measures and cleaned of any accumulated silt as required. The deposits will be disposed off-site as per the requirements of the contract.

9. Sediment control barriers may only be removed temporarily with approval of Contract Administrator to accommodate construction operations. All affected barriers must be reinstated at night when construction is completed. No removal will occur if there is a significant rainfall event anticipated (>10 mm) unless a new device has been installed to protect existing storm and sanitary sewer systems or downstream watercourses.

10. No refueling or cleaning of equipment is permitted near any existing waterway.

11. The Contractor shall remove sediment control measures when, in the opinion of the Contract Administrator, the measure(s) are no longer required. No control measures shall be permanently removed without prior written authorization from the Contract Administrator.

12. The Contractor shall periodically, or when requested by the Contract Administrator, clean out accumulated sediments as required.

13. The Contractor shall immediately report to the engineer any accidental discharges of sediment material into the watercourse. Appropriate response measures, including any repairs to existing control measures or the implementation of additional control measures, shall be carried out by the Contractor without delay.

14. Install trackout control mats (mud mats) at the entrance/egress to prevent migration of sediment into the public ROW.

15. Stormwater swales to be covered with hydro-seed and mulch.

16. Provide sediment traps and basins during dewatering works.

17. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.

18. 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, at minimum:

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

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

10 Geotechnical Investigation

A geotechnical investigation report was prepared by Paterson Group on November 7, 2025, to provide an assessment of the subsurface conditions found at the site and provide geotechnical design and construction recommendations for the proposed development. Four (4) boreholes, numbered BH 1-25 to BH 4-25, were advanced to a maximum depth of 9.1 metres below the existing ground surface. Excerpts from the geotechnical investigation report are attached in **Appendix D.1**.

The geotechnical investigation identified the following subsurface characteristics:

- Bedrock underlies the site between 1.5m and 4.5m below the existing ground surface.
- Monitoring wells were installed in three (3) boreholes, and a flexible PVC standpipe was installed in BH 1-25. Groundwater was measured on May 27, 2025, and ranged from depths of 1.44 m to 4.48 m below the existing ground surface, which equates to the range of 63.3 m to 65.69 m elevation. Though as groundwater levels are subject to seasonal fluctuations, they could vary at the time of construction.
- No sensitive marine clays were detected on this Site. Consequently, the geotechnical report does not identify any grade raise restrictions.

Based on Paterson's recommendations, the site is suitable for the proposed development. The following recommendations were made for the design and construction:

- Conventional spread footings placed on an undisturbed, compact to very dense glacial till deposit or clean surface sounded bedrock.
- Removal of bedrock and/or large boulders are anticipated to be required for construction and servicing installation.
- Various asphalt pavement structures are recommended for the site, including:
    - Car-Only Parking Areas and Fire-Truck Routes (Table 4)
    - Heavy-Truck Traffic and Loading Areas (Table 5)
    - Car-Only Parking Areas (Podium Deck and Raft Slab), (Table 6)
    - Access Lane, Fire Truck Lane, Ramp, and Heavy Truck Parking Areas (Podium Deck and Raft Slab), (Table 7)

  The asphalt pavement structures are included for reference in **Appendix D.1**.
- A 1.5m thick soil cover or a combination of soil cover in conjunction insulation should be provided over all footings to protect against frost action.
- Due to the varying groundwater elevations measured onsite, foundation drains and waterproofing systems will be required for the proposed structures.
- Due to the proximity of the critical storm and sanitary trunk sewers to this site and the anticipated rock breaking requirements for the underground levels, vibrational monitoring and mitigation measures are recommended, and pre-blast/pre-construction survey requirements are identified.
- Excavation, backfill, and engineered fill specifications are provided.
- Pipe bedding and backfill requirements are provided.
- Site specific sub-excavation requirements, including rock removal provisions are identified.

For further information, please refer to the complete geotechnical report.

11 Closing

This Site Servicing and Stormwater Management report assessed the servicing conditions within and immediately adjacent to the project site and evaluated the feasibility of new stormwater management infrastructure to meet the quantity control objectives for the site. Other SPC design requirements appeared in this report including:

- potable water and fire flow servicing,
- sanitary servicing,
- stormwater servicing,
- grading and drainage,
- utilities considerations,
- approvals,
- best management practices for erosion and sediment control during construction, and
- geotechnical considerations.

This report has demonstrated the proposed site servicing meets the design criteria as outlined during the pre-consultation process and in accordance with the 2025 Design Guidelines. The adjacent municipal infrastructure has been shown to provide adequate servicing for potable water, fire flow servicing, and stormwater servicing; however, it is acknowledged that the City may add a holding symbol to the applicable zoning until there is sufficient capacity made available in the sanitary trunk sewer to receive the anticipated wastewater discharge from this proposed development.

We trust that the information, servicing strategies, and proposed and the proposed stormwater management systems presented in this report are adequate to support the Site Plan Control Application for this infill development. Please contact the author or engineer of record of this report if you have any questions or concerns.

Appendix A Water Servicing

A.1 Water Demand Sheet

Stantec

2655 Queensview Drive, Ottawa, ON - Domestic Water Demand Estimates

Site Plan provided by J+S Architect (2-Feb-2026)

Stantec Project No.: 160402157
Date: 3-Jan-2026
Revision: 0
Revised by: -
Date Revised: -

Designed by: MW
Checked by: AG
Date Checked: 25-May-2026
City File No. PC2025-0063

Population densities per Table 4.1 City of Ottawa Water Distribution Guidelines:			
Studio/1 Bedroom		1.4	ppu
2 Bedroom		2.1	ppu
3 Bedroom		3.1	ppu
Demand conversion factors per Table 4.2 of the City of Ottawa Water Design Guidelines:			
Residential		280	L/cap/day
Commercial		28000	L/ha/day

Unit Type	No. of Units	Population	Area (m²)	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)
Building A									
1 Bedroom	47	66		12.8	0.2	32.1	0.5	70.6	1.2
1 Bedroom + Den[3]	15	32		6.2	0.1	15.6	0.3	34.2	0.6
2 Bedroom	82	172		33.5	0.6	83.7	1.4	184.2	3.1
3 Bedroom	5	16		3.0	0.1	7.5	0.1	16.6	0.3
Building A Subtotal	*149*	*286*		*55.6*	*0.9*	*138.9*	*2.3*	*305.5*	*5.1*
Building B									
Studio	1	2		0.4	0.0	1.0	0.0	2.1	0.0
1 Bedroom	77	108		21.0	0.4	52.5	0.9	115.5	1.9
1 Bedroom + Den[3]	18	38		7.4	0.1	18.5	0.3	40.6	0.7
2 Bedroom	87	183		35.5	0.6	88.8	1.5	195.4	3.3
3 Bedroom	6	19		3.6	0.1	9.0	0.2	19.9	0.3
Building B Subtotal	*189*	*349*		*67.9*	*1.1*	*169.8*	*2.8*	*373.6*	*6.2*
Residential Subtotal	*338*	*635*		*123.5*	*2.1*	*308.7*	*5.1*	*679.1*	*11.3*
Gross Commercial[4]			*9039*	*17.6*	*0.3*	*26.4*	*0.4*	*47.5*	*0.8*
Total	**338**	**635**	**9039**	**141.0**	**2.4**	**335.0**	**5.6**	**726.6**	**12.1**

Notes:

1 The City of Ottawa 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 WDG002 Table 4.2)

2 Water demand criteria used to estimate peak demand rates for commercial area are as follows:
maximum daily demand rate = 1.5 x average day demand rate
peak hour demand rate = 1.8 x maximum day demand rate (as per WDG002 Table 4.2)

3 Assumption that "1 bedroom with den" has density of 2.1 ppu.

4 Gross site area assumed for commercial demands.

A.2 Correspondence with the Architect Confirming Building Construction

Gladish, Alyssa

From: Jeff Shen <jeff.s@jsarchitect.ca>
Sent: Friday, November 14, 2025 6:14 PM
To: Gladish, Alyssa
Cc: Wu, Michael; Ian O'Donnell
Subject: Re: 2655 Queensview Drive, Ottawa - Building Construction Confirmation

Follow Up Flag: Follow up
Flag Status: Completed

Hi Alyssa,

The construction is concrete below grade and Type V – Wood Frame above. Same as Cryville Road.

JEFF SHEN
Architect AIBC, AAA, OAA
M: (604) 210-9698 D: (604) 921-3938 C: (778) 858 -1805
Email: jeff.s@jsarchitect.ca
601-6378 Silver Avenue, Burnaby, BC V5H 0J2
www.jsarchitect.ca

From: Gladish, Alyssa <Alyssa.Gladish@stantec.com>
Sent: Friday, November 14, 2025 9:15 AM
To: Jeff Shen <jeff.s@jsarchitect.ca>
Cc: Wu, Michael <Michael.Wu@stantec.com>; Ian O'Donnell <ian@westrichpacific.com>
Subject: RE: 2655 Queensview Drive, Ottawa - Building Construction Confirmation

Hello Jeff,

In the screenshot provided below on October 22, it appears that the Building B firewall is on gridline F. When we spoke yesterday, I was under the impression it was along gridline E for both Buildings A and B. Please confirm.

We have looked at 1125 Cyrville Road. Please confirm the following applies to this 2655 Queensview Drive Site

1. Type of construction: **We are assuming concrete (Type I) below grade and Ordinary Construction (Type III) with 1-hour fire-rated floor assemblies and exterior above grade. Please confirm.** **Confirmed**
    i. Type I – Fire Resistive Construction Non-Combustible without Fire-Resistance Ratings
    ii. Type II – Noncombustible Construction / Type IV-A – Mass Timber Construction
    iii. Type III – Ordinary Construction / Type IV-C – Mass Timber Construction
    iv. Type IV-B Mass Timber Construction
    v. Type V – Wood Frame / Type IV-D – Mass Timber Construction

2. If Type IV-B, Type II, Type IV-A, or Type I building construction: Will the vertical openings between floors be classified as “protected openings” or “unprotected openings” as per The NBC Division B, Section 3.5. Vertical Transportation. **Not applicable to Type III, no further action.**
3. During the kickoff meeting, it was mentioned that there may be commercial uses adjacent to the main entrance. I don’t see any commercial uses in the floor plates provided in the PDF review set (2025-07-21). Is a commercial occupancy still being considered for this site in addition to the residential occupancy? Can you please provide floor plates that include the commercial spaces? **Confirmed below, no further action.**
4. Will the building be Sprinklered? **Yes**
    1. Sprinkler conforming to NFPA13? **Yes**
    2. Sprinkler to be fully supervised? (Can be remotely supervised) **Yes**
    3. % Coverage by sprinkler system? **100%**
5. Are there any additional details pertinent to the building’s fire resistivity? Fire walls, etc.? **Yes, firewalls have been identified below. Confirm gridlines.**

Thank you,
Alyssa

Alyssa Gladish, E.I.T.
Project Manager

Stantec

From: Jeff Shen <jeff.s@jsarchitect.ca>
Sent: Wednesday, October 22, 2025 8:51 PM
To: Gladish, Alyssa <Alyssa.Gladish@stantec.com>
Cc: Wu, Michael <Michael.Wu@stantec.com>; Ian O'Donnell <ian@westrichpacific.com>
Subject: Re: 2655 Queensview Drive, Ottawa - Building Construction Confirmation

Hi Alyssa,

Please find below the screenshot shows the proposed CRU location.

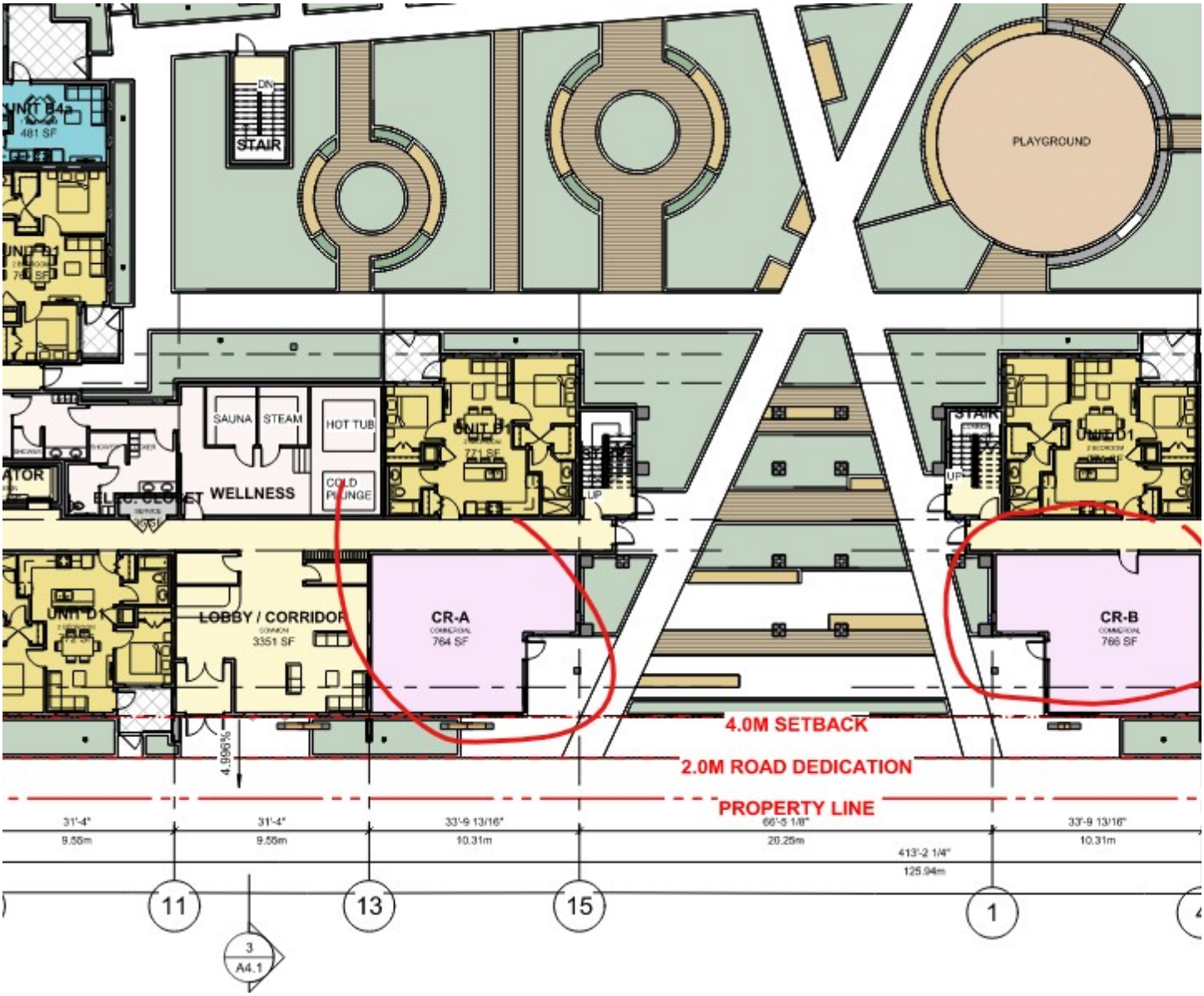
Also this link for a latest full DP set: Queensview - DP Set-2025 1022.pdf

Can you please refer to another project: 1125 Cyrvill Road for the rest of questions? Stantac has been working on and it's under construction. There are very similar regarding your questions.

There is firewall for each building in the middle.

Let me know if you need anything.


13'-5 1/8"
4.09m
22'-9"
6.93m
31'-4"
9.55m
22'-3"
6.78m
31'-4"
9.55m
31'-4"
9.55m
6.78m
UNIT D1
774 SF
UNIT B1
532 SF
UNIT D1
774 SF
UNIT D1
774 SF
UNIT B1
532 SF
UNIT D1
774 SF
UNIT D1
772 SF
UNIT D1
773 SF
UNIT C1
954 SF
UNIT B4c
847 SF
UNIT B6
961 SF
UNIT B1
532 SF
UNIT D1
774 SF
UNIT D5
775 SF
STAIR
UNIT D5
UNIT C1
UNIT B4
538 SF
UNIT B4a
461 SF
UNIT B6
961 SF
UNIT D1
774 SF
27'-10 1/2"
8.50m
22'-3"
6.78m
31'-4"
9.55m
32'-9 13/16"
10.00m
32'-11"
10.03m
31'-5 13/16"
9.60m
CORRIDOR
2344 SF
UNIT C1
954 SF
UNIT B3
647 SF
ELEVATOR
ELEC. CLOSET
UNIT D6
765 SF
UNIT B1
771 SF
STAIR
UNIT B1
532 SF
UNIT B1
773 SF
UNIT F1
UNIT B1
532 SF
UNIT D1
774 SF
UNIT D1
774 SF
UNIT D2
765 SF
UNIT D2
UNIT D1
773 SF
15'-1 7/8"
4.62m
17'-4 1/2"
5.30m
15'-0 1/4"
4.58m
22'-3"
6.78m
31'-4"
9.55m
31'-4"
9.55m
33'-9 13/16"
10.31m



JEFF SHEN
Architect AIBC, AAA, OAA
M: (604) 210-9698 D: (604) 921-3938 C: (778) 858 -1805
Email: jeff.s@jsarchitect.ca
601-6378 Silver Avenue, Burnaby, BC V5H 0J2
www.jsarchitect.ca

From: Gladish, Alyssa <Alyssa.Gladish@stantec.com>
Sent: Wednesday, October 22, 2025 12:43 PM
To: Jeff Shen <jeff.s@jsarchitect.ca>
Cc: Wu, Michael <Michael.Wu@stantec.com>; Ian O'Donnell <ian@westrichpacific.com>
Subject: 2655 Queensview Drive, Ottawa - Building Construction Confirmation

Hello Jeff,

To calculate the Fire Flow Requirements for 2655 Queensview Drive, we require confirmation of some of the Building Construction and Fire Resistive Design components.

Can you please confirm or provide the following information:

1. Type of construction: We are assuming concrete (Type I) below grade and wood framed walls and floors (Type V) above grade. Please confirm.
    i. Type I – Fire Resistive Construction Non-Combustible without Fire-Resistance Ratings
    ii. Type II – Noncombustible Construction / Type IV-A – Mass Timber Construction
    iii. Type III – Ordinary Construction / Type IV-C – Mass Timber Construction
    iv. Type IV-B Mass Timber Construction
    v. Type V – Wood Frame / Type IV-D – Mass Timber Construction
2. If Type IV-B, Type II, Type IV-A, or Type I building construction: Will the vertical openings between floors be classified as “protected openings” or “unprotected openings” as per The NBC Division B, Section 3.5. Vertical Transportation.
3. During the kickoff meeting, it was mentioned that there may be commercial uses adjacent to the main entrance. I don’t see any commercial uses in the floor plates provided in the PDF review set (2025-07-21). Is a commercial occupancy still being considered for this site in addition to the residential occupancy? Can you please provide floor plates that include the commercial spaces?
4. Will the building be Sprinklered?
    1. Sprinkler conforming to NFPA13?
    2. Sprinkler to be fully supervised? (Can be remotely supervised)
    3. % Coverage by sprinkler system?
5. Are there any additional details pertinent to the building’s fire resistivity? Fire walls, etc.?

Thank you kindly!
Alyssa

Alyssa Gladish, E.I.T.
Project Manager

Direct: (780) 917-8567
Mobile: (587) 721-1241
alyssa.gladish@stantec.com

Stantec

With every community, we redefine what's possible.

Alyssa Gladish, E.I.T.
Project Manager

Stantec

Caution: This email originated from outside of Stantec. Please take extra precaution.
Attention: Ce courriel provient de l'extérieur de Stantec. Veuillez prendre des précautions supplémentaires.
Atención: Este correo electrónico proviene de fuera de Stantec. Por favor, tome precauciones adicionales.

A.3 Fire Flow Demand Sheet (2020 FUS)

Stantec

FUS Fire Flow Calculation Sheet - 2020 FUS Guidelines

Stantec Project #: 160402157
Project Name: 2655 Queensview Drive
City File No.: PC2025-0063
Date: 10/27/2025
Fire Flow Calculation #: 1
Description: Building A (North)

Designed by: MW
Checked by: AG
Date Checked: 5/25/2026
Revision: 0
Revised by: -
Date Revised: -

Notes: 6-Storey Residential - 1-hour Rated Floor Assemblies/Exterior. Firewall on Gridline E of site plan reduces northern building footprint to 1100 m².

Step	Task	Notes										Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type III - Ordinary Construction / Type IV-C - Mass Timber Construction										1	-
2	Determine Effective Floor Area	Sum of All Floor Areas										-	-
		1110	1110	1110	1110	1110	1110					6662	-
3	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min										-	18000
4	Determine Occupancy Charge	Limited Combustible										-15%	15300
5	Determine Sprinkler Reduction	Conforms to NFPA 13										-30%	-7650
		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	59	2	> 100	Type V	NO				10%	6120
		East	3.1 to 10	25	6	> 100	Type III-IV - Unprotected Openings	NO				15%	
		South	0 to 3	18	6	> 100	Type III-IV - Unprotected Openings	YES				0%	
		West	3.1 to 10	3	2	0-20	Type V	NO				15%	
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min											14000
		Total Required Fire Flow in L/s											233.3
		Required Duration of Fire Flow (hrs)											3.00
		Required Volume of Fire Flow (m^3)											2520

Stantec

FUS Fire Flow Calculation Sheet - 2020 FUS Guidelines

Stantec Project #: 160402157
Project Name: 2655 Queensview Drive
City File No.: PC2025-0063
Date: 10/27/2025
Fire Flow Calculation #: 1
Description: Building B (North)

Designed by: MW
Checked by: AG
Date Checked: 5/25/2026
Revision: 0
Revised by: -
Date Revised: -

Notes: 6-Storey Residential - 1-hour Rated Floor Assemblies/Exterior. Firewall on Gridline E of site plan reduces northern building footprint to 927 m².

Step	Task	Notes										Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type III - Ordinary Construction / Type IV-C - Mass Timber Construction										1	-
2	Determine Effective Floor Area	Sum of All Floor Areas										-	-
		927	927	927	927	927	927					5564	-
3	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min										-	16000
4	Determine Occupancy Charge	Limited Combustible										-15%	13600
5	Determine Sprinkler Reduction	Conforms to NFPA 13										-30%	-6800
		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	58	2	> 100	Type III-IV - Unprotected Openings	NO				5%	2720
		East	20.1 to 30	10	2	0-20	Type III-IV - Unprotected Openings	NO				0%	
		South	0 to 3	18	6	> 100	Type III-IV - Unprotected Openings	YES				0%	
		West	3.1 to 10	33	6	> 100	Type III-IV - Unprotected Openings	NO				15%	
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min											10000
		Total Required Fire Flow in L/s											166.7
		Required Duration of Fire Flow (hrs)											2.00
		Required Volume of Fire Flow (m^3)											1200

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FUS Fire Flow Calculation Sheet - 2020 FUS Guidelines

Stantec Project #: 160402157
Project Name: 2655 Queensview Drive
City File No.: PC2025-0063
Date: 10/27/2025
Fire Flow Calculation #: 1
Description: Building A (South)

Designed by: MW
Checked by: AG
Date Checked: 5/25/2026
Revision: 0
Revised by: -
Date Revised: -

Notes: 6-Storey Residential - 1-hour Rated Floor Assemblies/Exterior. Firewall on Gridline E of site plan reduces southern building footprint to 733 m².

Step	Task	Notes										Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type III - Ordinary Construction / Type IV-C - Mass Timber Construction										1	-
2	Determine Effective Floor Area	Sum of All Floor Areas										-	-
		733	733	733	733	733	733					4397	-
3	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min										-	15000
4	Determine Occupancy Charge	Limited Combustible										-15%	12750
5	Determine Sprinkler Reduction	Conforms to NFPA 13										-30%	-6375
		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	0 to 3	18	6	> 100	Type III-IV - Unprotected Openings	YES				0%	4080
		East	3.1 to 10	25	6	> 100	Type III-IV - Unprotected Openings	NO				15%	
		South	> 30	0	0	0-20	Type V	NO				0%	
		West	3.1 to 10	24.4	2	41-60	Type V	NO				17%	
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min											10000
		Total Required Fire Flow in L/s											166.7
		Required Duration of Fire Flow (hrs)											2.00
		Required Volume of Fire Flow (m^3)											1200

Stantec

FUS Fire Flow Calculation Sheet - 2020 FUS Guidelines

Stantec Project #: 160402157
Project Name: 2655 Queensview Drive
City File No.: PC2025-0063
Date: 10/27/2025
Fire Flow Calculation #: 1
Description: Building B (South)

Designed by: MW
Checked by: AG
Date Checked: 5/25/2026
Revision: 0
Revised by: -
Date Revised: -

Notes: 6-Storey Residential - 1-hour Rated Floor Assemblies/Exterior. Firewall on Gridline E of site plan reduces southern building footprint to 1278 m².

Step	Task	Notes										Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type III - Ordinary Construction / Type IV-C - Mass Timber Construction										1	-
2	Determine Effective Floor Area	Sum of All Floor Areas										-	-
		1278	1278	1278	1278	1278	1278					7665	-
3	Determine Required Fire Flow	($F = 220 \times C \times A^{1/2}$). Round to nearest 1000 L/min										-	19000
4	Determine Occupancy Charge	Limited Combustible										-15%	16150
5	Determine Sprinkler Reduction	Conforms to NFPA 13										-30%	-8075
		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	0 to 3	18	6	> 100	Type III-IV - Unprotected Openings	YES				0%	2584
		East	20.1 to 30	15	2	21-49	Type III-IV - Unprotected Openings	NO				1%	
		South	> 30	0	0	0-20	Type V	NO				0%	
		West	3.1 to 10	25	6	> 100	Type III-IV - Unprotected Openings	NO				15%	
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min											11000
		Total Required Fire Flow in L/s											183.3
		Required Duration of Fire Flow (hrs)											2.00
		Required Volume of Fire Flow (m^3)											1320

A.4 Hydraulic Boundary Conditions

Wu, Michael

From: Candow, Julie <julie.candow@ottawa.ca>
Sent: November 24, 2025 11:49
To: Wu, Michael
Cc: Gladish, Alyssa
Subject: RE: PC2025-0063 - 2655 Queensview Drive Boundary Conditions Request
Attachments: 2655 Queensview Drive November 2025.pdf

Hi Michael,

Please see attached figure and below boundary condition results for 2655 Queensview.

The following are boundary conditions, HGL, for hydraulic analysis at 2655 Queensview Drive (zone 1W) assumed to be dually connected via the 305 mm watermain on Queensview Drive. (see attached PDF for location).

Minimum HGL = 107.8 m
Maximum HGL = 115.1 m
Max Day + Fire Flow (233.3 L/s) = 94.6 m

These are for current conditions and are based on computer model simulation.

Disclaimer: The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation.

Julie Candow, P.Eng
Project Manager
Development Review – West Branch
Planning, Development and Building Services Dept.
110 Laurier Avenue West, 4th Floor East
Ottawa, ON K1P 1J1
613.580.2424 ext. 13850

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: November 17, 2025 4:54 PM
To: Candow, Julie <julie.candow@ottawa.ca>
Cc: Gladish, Alyssa <alyssa.gladish@stantec.com>
Subject: RE: PC2025-0063 - 2655 Queensview Drive Boundary Conditions Request

ATTENTION : Ce courriel provient d'un expéditeur externe. Ne cliquez sur aucun lien et n'ouvrez pas de pièce jointe, excepté si vous connaissez l'expéditeur.

Hi Julie, yes, the intent is for a pair of service connections at the same connection location.

Michael Wu, EIT
Civil Engineering Intern
He, him

Stantec

From: Candow, Julie <julie.candow@ottawa.ca>
Sent: November 17, 2025 16:50
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Gladish, Alyssa <Alyssa.Gladish@stantec.com>
Subject: RE: PC2025-0063 - 2655 Queensview Drive Boundary Conditions Request

Hi Michael,

As per technical bulletin ISTB 2021-03, residential areas serving 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. You have noted one connection location in your attached sketch, will the second connection location be paired with it?

Thanks,

Julie Candow, P.Eng
Project Manager
Development Review – West Branch
Planning, Development and Building Services Dept.
110 Laurier Avenue West, 4th Floor East
Ottawa, ON K1P 1J1
613.580.2424 ext. 13850

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: November 14, 2025 3:46 PM
To: Candow, Julie <julie.candow@ottawa.ca>
Cc: Gladish, Alyssa <alyssa.gladish@stantec.com>
Subject: PC2025-0063 - 2655 Queensview Drive Boundary Conditions Request

Good afternoon, Julie:

We are requesting hydraulic boundary conditions and the maximum available fire flow at 20 psi from the 300 mm diameter watermain on Queensview Drive. The request is in support of a site

plan control (SPC) application for the proposed development at 2655 Queensview Drive, which will see mixed-use development and service a total population of 633 persons and a gross area of 0.9 ha for commercial services.

The development is projected to be serviced from the 300 mm diameter watermain on Queensview Drive, with the connection location circled in **red** in the attached map. The water and fire flow demands for the development are as follows:

- Average Day Demands (AVDY): 2.3 L/s
- Maximum Day Demands (MXDY): 5.6 L/s
- Peak Hour Demands (PKHR): 12.1 L/s
- Fire Flow Demands: 233 L/s (14,000 L/min)

Attached are the calculation sheets for your reference.

We appreciate you taking the time to look into the matter, please let us know if you have any questions or require further information.

Thank you,

Michael Wu, EIT
Civil Engineering Intern
He, him

Direct: (613) 738 6033
michael.wu@stantec.com

Stantec

With every community, we redefine what's possible.

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Caution: This email originated from outside of Stantec. Please take extra precaution.

Attention: Ce courriel provient de l'extérieur de Stantec. Veuillez prendre des précautions supplémentaires.

Atención: Este correo electrónico proviene de fuera de Stantec. Por favor, tome precauciones adicionales.

Boundary Conditions for 2655 Queensview Drive

N

Dual Connections

Legend

- Public
- Private

City of Ottawa

A.5 Fire Hydrant Coverage Sheet

Stantec

Project: **2655 Queensview Drive** 160402157

TABLE 1:
FIRE HYDRANT COVERAGE TABLE

Revision: 01 Prepared By: MW

Revision Date: 5/25/2026 Checked By: AG

Description	Hydrants[1]				Total Available Fire Flow (L/min)	Total Required Fire Flow[2] (L/min)
	EX HYD-01	EX HYD-02	EX HYD-03	P HYD-01		
Building A - South						
Distance from building (m)	34.8	57.2	-	22.5	-	-
Maximum fire flow capacity[3] (L/min)	5,678	5,678	-	5,678	**17,034**	10,000
Building A - North						
Distance from building (m)	62.0	94.0	-	75.1	-	-
Maximum fire flow capacity[3] (L/min)	5,678	3,785	-	5,678	**15,141**	14,000
Building B - South						
Distance from building (m)	41.9	-	55.6	40.4	-	-
Maximum fire flow capacity[3] (L/min)	5,678	-	5,678	5,678	**17,034**	11,000
Building B - North						
Distance from building (m)	71.0	-	89.6	85.5	-	-
Maximum fire flow capacity[3] (L/min)	5,678	-	3,785	3,785	**13,248**	10,000

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 Drawing SSP-1.
2. See FUS Calculations, Appendix B.2 for fire flow requirements.
3. See NFPA 1 Table 18.5.4.3 for maxiumim fire flow capacity of hydrants by distance to building.

Appendix B Wastewater Servicing

B.1 Sanitary Sewer Design Sheet

Stantec

SUBDIVISION: **2655 Queensview Drive**

DATE: 1/20/2026
REVISION: 1
DESIGNED BY: MW
CHECKED BY: AG

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160402157

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 BED	1.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 2 BED	2.1	INFILTRATION	0.33 l/s/Ha	HARMON CORRECTION FACTOR	0.8
PERSONS / 3 BED	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 BED	UNITS 2 BED	UNITS 3 BED	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)
			(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)
R3A, R3B, G3A	3	1	0.45	125	202	11	635	0.45	635	3.33	6.9	0.92	0.92	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.4	0.92	0.92	0.3	7.6	1.1	200	PVC	SDR 35	1.00	33.4	**22.76%**	1.05
	1	EX SAN	0.00	0	0	0	0	0.45	635	3.33	6.9	0.00	0.92	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.4	0.00	0.92	0.3	7.6	10.1	200	PVC	SDR 35	1.00	33.4	**22.76%**	1.05

B.2 City Correspondence for Receiving Sewer Capacity

From: Candow, Julie <julie.candow@ottawa.ca>
Sent: Monday, February 2, 2026 3:27 PM
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Gladish, Alyssa <Alyssa.Gladish@stantec.com>; Roy, Jean-Miguel <Jean-Miguel.Roy@ottawa.ca>; Baldwin, Kimberley <Kimberley.Baldwin@ottawa.ca>; Davidson, Amanda <amanda.davidson@ottawa.ca>
Subject: RE: PC2025-0063 - 2655 Queensview Drive Downstream Sanitary Sewer Capacity Confirmation

Hi Michael,

In speaking with the Infrastructure Planning unit, they have noted that the Pinecrest Sanitary Sewer Trunk is at capacity. The only sanitary sewer allocation we can guarantee at this time would be the existing sanitary sewer outflow of the site as it currently operates (sanitary sewer flows from the 2655 Queensview site after redevelopment would remain the same).

The Infrastructure Planning unit has noted a review of the existing sanitary sewer model for the Pinecrest Sanitary Sewer Trunk is needed to identify whether there is any available capacity for redevelopment in the area. We have been advised that the next step will be a flow monitoring program to identify existing Inflow & Infiltration (I&I) for areas draining to the Pinecrest Trunk Sewer. This flow monitoring is expected to start Spring 2026. Preliminary results are expected by fall 2026 which will give us a basic understanding of whether additional flows can be made available for developments in our subject area. Based on the results of the monitoring, further works (projected 2027) may be required to remove any identified I&Is.

As we move through site plan approval, we can keep you informed on the sanitary sewer capacity of the Pinecrest Trunk as more information becomes available. For the time being, there is no surplus of sanitary capacity in the trunk sewer that we can allocate to your site.
If the sanitary outflow from the proposed site plan application exceeds the existing sanitary outflow from the site, the City may put the development application on hold by imposing development conditions requiring adequate servicing or by adding a holding symbol to applicable zoning until there is sufficient capacity.

Let me know if you have any questions on the above.

Julie Candow, P.Eng
Project Manager
Development Review – West Branch
Planning, Development and Building Services Dept.
110 Laurier Avenue West, 4th Floor East
Ottawa, ON K1P 1J1
613.580.2424 ext. 13850

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: February 02, 2026 11:37 AM
To: Candow, Julie <julie.candow@ottawa.ca>
Cc: Gladish, Alyssa <alyssa.gladish@stantec.com>
Subject: PC2025-0063 - 2655 Queensview Drive Downstream Sanitary Sewer Capacity Confirmation

Good morning, Julie:

We are looking to confirm the capacity of the downstream sanitary sewer to receive peak flows from the proposed development at 2655 Queensview Drive. A total peak flow of 7.6 L/s is estimated based on a population of 635 persons and gross commercial area of 0.9 ha.

Attached is the calculation sheet for your reference and review.

Please let us know if you have any questions or comments.

Thanks,

Michael Wu, EIT
Civil Engineering Intern
He, him

Direct: (613) 738 6033
michael.wu@stantec.com

Appendix C Stormwater Management

C.1 Modified Rational Method

Stormwater Management Calculations

File No: **160402157**
Project: **2655 Queensview Drive**
Date: **09-Mar-26**

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
External - Tributary	EX CIST 6 & EX CIST 7	Hard	0.019		0.9	0.017		
		Soft	0.000		0.2	0.000		
		Subtotal		0.019			0.017	0.900
Cistern	CIST 1 - CIST 17	Hard	0.218		0.9	0.197		
		Soft	0.198		0.2	0.040		
		Subtotal		0.417			0.236	0.57
Roof	R102A	Hard	0.205		0.9	0.184		
		Soft	0.000		0.2	0.000		
		Subtotal		0.205			0.184	0.90
Roof	R102B	Hard	0.239		0.9	0.215		
		Soft	0.000		0.2	0.000		
		Subtotal		0.239			0.215	0.90
Uncontrolled - Non-Tributary	UNC-1	Hard	0.033		0.9	0.029		
		Soft	0.022		0.2	0.004		
		Subtotal		0.055			0.034	0.62
Total				**0.934**			**0.686**	
Overall Runoff Coefficient= C:								**0.73**

Total Roof Areas	**0.444 ha**
Total Tributary Onsite Surface Areas (Controlled)	**0.417 ha**
Total Onsite Tributary Area to Storm Service	**0.861 ha**
Total Tributary External Areas	**0.019 ha**
Total Tributary Area to Storm Service	**0.879 ha**
Total Uncontrolled Areas (Non-Tributary)	**0.055 ha**
Total Study Area	**0.934 ha**

Project #160402157, 2655 Queensview Drive
Roof Drain Design Sheet, Area R102A
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m) Increment	Volume (cu. m) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0025	0	0.025	46	0	0	0.025
0.050	0.0006	0.0050	3	0.050	182	3	3	0.050
0.075	0.0008	0.0063	10	0.075	410	7	10	0.075
0.100	0.0009	0.0076	24	0.100	729	14	24	0.100
0.125	0.0011	0.0088	47	0.125	1138	23	47	0.125
0.150	0.0013	0.0101	82	0.150	1639	35	82	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
2.7	526.4	2.7	0.14622
9.9	1143.0	7.2	0.46371
23.9	1854.9	14.0	0.97895
47.1	2621.1	23.1	1.70704
81.6	3421.4	34.5	2.65745

Rooftop Storage Summary

Total Building Area (sq.m)		2049.26
Assume Available Roof Area (sq.m	80%	1639.408
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		8
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		82
Estimated 100 Year Drawdown Time (h)		2.6

* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).

* Note: Number of drains can be reduced if multiple-notch drain used.

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.007	0.010	-
Depth (m)	0.099	0.148	0.150
Volume (cu.m)	23.5	79.3	82.0
Draintime (hrs)	1.0	2.6	

Adjustable Accutrol Weir Flow Rate Settings From Watts Drain Catalogue

Head (m)	L/s Open	75%	**50%**	25%	Closed
0.025	0.3154	0.31542	**0.31542**	0.31542	0.31542
0.05	0.6308	0.63083	**0.63083**	0.63083	0.31542
0.075	0.9462	0.8674	**0.78854**	0.70969	0.31542
0.1	1.2617	1.10396	**0.94625**	0.78854	0.31542
0.125	1.5771	1.34052	**1.10396**	0.8674	0.31542
0.15	1.8925	1.57708	**1.26167**	0.94625	0.31542

Project #160402157, 2655 Queensview Drive
Roof Drain Design Sheet, Area R102B
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain

Rating Curve				Volume Estimation				
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m) Increment	Volume (cu. m) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0028	0	0.025	53	0	0	0.025
0.050	0.0006	0.0057	4	0.050	212	3	4	0.050
0.075	0.0008	0.0071	12	0.075	478	8	12	0.075
0.100	0.0009	0.0085	28	0.100	850	16	28	0.100
0.125	0.0011	0.0099	55	0.125	1328	27	55	0.125
0.150	0.0013	0.0114	96	0.150	1912	40	96	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
3.1	545.8	3.1	0.15162
11.5	1185.2	8.4	0.48085
27.9	1923.4	16.4	1.01512
54.9	2718.0	27.0	1.77012
95.2	3547.9	40.3	2.75564

Rooftop Storage Summary

Total Building Area (sq.m)		2390.6
Assume Available Roof Area (sq.m)	80%	1912.48
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		9
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		96
Estimated 100 Year Drawdown Time (h)		2.7

* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).

* Note: Number of drains can be reduced if multiple-notch drain used.

Adjustable Accutrol Weir Flow Rate Settings From Watts Drain Catalogue					
Head (m)	L/s Open	75%	**50%**	25%	Closed
0.025	0.3154	0.3154	**0.3154**	0.3154	0.3154
0.05	0.6308	0.6308	**0.6308**	0.6308	0.3154
0.075	0.9462	0.8674	**0.7885**	0.7097	0.3154
0.1	1.2617	1.104	**0.9462**	0.7885	0.3154
0.125	1.5771	1.3405	**1.104**	0.8674	0.3154
0.15	1.8925	1.5771	**1.2617**	0.9462	0.3154

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.008	0.011	-
Depth (m)	0.099	0.149	0.150
Volume (cu.m)	27.9	93.7	95.6
Draintime (hrs)	1.0	2.7	

Stormwater Management Calculations

Project #160402157, 2655 Queensview Drive
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	76.8
20	52.0
30	40.0
40	32.9
50	28.0
60	24.6
70	21.9
80	19.8
90	18.1
100	16.7
110	15.6
120	14.6

2 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Onsite Area Tributary to the Outlet
Onsite Controlled
Area (ha): 0.861
C: 0.50

PWDC Allowable Discharge (L/s/ha)	Q_{PWDC} (L/s)	Q_{target} Q_{PWDC} + Q_{EXT} (L/s)
33.5	28.8	32.4

Predevelopment Onsite Area Tributary to the Outlet

tc (min)	I (2 yr) (mm/hr)	Q2-yr (L/s)	Q2-yr (L/s/ha)	Q2-yr +Q_{EXT} (L/s/ha)
10	76.8	33.1	38.4	36.6

2 YEAR Modified Rational Method for Entire Site

Subdrainage Area: CIST 1 - CIST 17 — Cistern
Includes controlled release rate from roof areas and external flow contributions.
Area (ha): 0.42
C: 0.57

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.8	69.2	32.4	36.8	22.1
20	52.0	52.5	32.4	20.1	**24.1**
30	40.0	44.1	32.4	11.7	21.1
40	32.9	38.9	32.4	6.5	15.7
50	28.0	35.3	32.4	2.9	8.7
60	24.6	32.5	32.4	0.1	0.5
70	21.9	30.3	30.3	0.0	0.0
80	19.8	28.5	28.5	0.0	0.0
90	18.1	27.0	27.0	0.0	0.0
100	16.7	25.7	25.7	0.0	0.0
110	15.6	24.5	24.5	0.0	0.0
120	14.6	23.5	23.5	0.0	0.0

Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
32.4	24.1	140.0	OK

115.9

Subdrainage Area: R102A — Roof
Area (ha): 0.20 — Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.8	39.4	7.1	32.3	19.4	91.2	0.0
20	52.0	26.7	7.5	19.2	23.1	97.8	0.0
30	40.0	20.5	7.5	13.0	**23.5**	98.5	0.0
40	32.9	16.9	7.4	9.4	22.6	97.0	0.0
50	28.0	14.4	7.3	7.1	21.2	94.6	0.0
60	24.6	12.6	7.1	5.4	19.6	91.6	0.0
70	21.9	11.2	7.0	4.2	17.8	88.5	0.0
80	19.8	10.2	6.8	3.3	16.0	85.3	0.0
90	18.1	9.3	6.7	2.6	14.2	82.1	0.0
100	16.7	8.6	6.5	2.1	12.5	79.0	0.0
110	15.6	8.0	6.4	1.6	10.8	75.9	0.0
120	14.6	7.5	6.2	1.3	9.4	72.1	0.0

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level	98.5	0.1	7.5	23.5	82.0	0.0

Project #160402157, 2655 Queensview Drive
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	178.6
20	120.0
30	91.9
40	75.1
50	64.0
60	55.9
70	49.8
80	45.0
90	41.1
100	37.9
110	35.2
120	32.9

100 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Onsite Area Tributary to the Outlet

Controlled Area (ha): 0.861
C: 0.50

PWDC Allowable Discharge (L/s/ha)	Q_{PWDC} (L/s)	Q_{target} Q_{PWDC} + Q_{EXT} (L/s)
33.5	28.8	32.4

Q2-yr +Q_{EXT} (L/s/ha)
42.3

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: CIST 1 - CIST 17 — Cistern
Includes controlled release rate from roof areas and external flow contributions.
Area (ha): 0.42
C: 0.71

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	175.2	32.4	142.8	85.7
20	120.0	125.3	32.4	92.8	111.4
30	91.9	101.2	32.4	68.8	123.9
40	75.1	86.8	32.4	54.4	130.6
50	64.0	77.1	32.4	44.7	134.1
60	55.9	70.0	32.4	37.6	**135.4**
70	49.8	64.6	32.4	32.2	135.2
80	45.0	60.3	32.4	27.9	133.8
90	41.1	56.7	32.4	24.3	131.4
100	37.9	53.8	32.4	21.4	128.2
110	35.2	51.2	32.4	18.8	124.2
120	32.9	49.0	32.4	16.6	119.7

Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
32.4	135.4	140.0	OK

4.6

Subdrainage Area: R102A — Roof
Area (ha): 0.20 — Maximum Storage Depth: 150 mm
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.6	101.7	9.1	92.6	55.6	130.9	0.0
20	120.0	68.3	9.7	58.7	70.4	141.6	0.0
30	91.9	52.3	9.9	42.4	76.4	146.0	0.0
40	75.1	42.8	10.0	32.8	78.8	147.7	0.0
50	64.0	36.4	10.0	26.4	**79.3**	148.1	0.0
60	55.9	31.8	10.0	21.9	78.7	147.7	0.0
70	49.8	28.4	9.9	18.4	77.4	146.7	0.0
80	45.0	25.6	9.9	15.8	75.7	145.4	0.0
90	41.1	23.4	9.8	13.6	73.6	144.0	0.0
100	37.9	21.6	9.7	11.9	71.3	142.3	0.0
110	35.2	20.1	9.6	10.4	68.9	140.5	0.0
120	32.9	18.7	9.5	9.2	66.4	138.7	0.0

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	148.1	0.1	10.0	79.3	82.0	0.0

Stormwater Management Calculations

Project #160402157, 2655 Queensview Drive
Modified Rational Method Calculations for Storage

Subdrainage Area: R102B — Roof
Area (ha): 0.24 — Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.8	45.9	8.0	37.9	22.7	91.5	0.0
20	52.0	31.1	8.4	22.7	27.2	98.3	0.0
30	40.0	24.0	8.5	15.5	**27.9**	99.3	0.0
40	32.9	19.7	8.4	11.3	27.0	98.0	0.0
50	28.0	16.8	8.3	8.5	25.5	95.7	0.0
60	24.6	14.7	8.1	6.6	23.7	92.9	0.0
70	21.9	13.1	7.9	5.2	21.7	89.9	0.0
80	19.8	11.9	7.8	4.1	19.7	86.8	0.0
90	18.1	10.9	7.6	3.3	17.6	83.7	0.0
100	16.7	10.0	7.4	2.6	15.6	80.6	0.0
110	15.6	9.3	7.2	2.1	13.7	77.6	0.0
120	14.6	8.7	7.1	1.6	11.8	74.5	0.0

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
2-year Water Level	99.3	0.1	8.5	27.9	95.6	0.0

Subdrainage Area: EX CIST 6 & EX CIST 7 — External - Tributary
Area (ha): 0.019
C: 0.9

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qext (L/s)
10	76.8	3.6	3.6
20	52.0	2.4	2.4
30	40.0	1.9	1.9
40	32.9	1.5	1.5
50	28.0	1.3	1.3
60	24.6	1.1	1.1
70	21.9	1.0	1.0
80	19.8	0.9	0.9
90	18.1	0.8	0.8
100	16.7	0.8	0.8
110	15.6	0.7	0.7
120	14.6	0.7	0.7

Subdrainage Area: UNC-1 — Uncontrolled - Non-Tributary
Area (ha): 0.05
C: 0.62

tc (min)	I (2 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)
10	76.8	7.2	7.2
20	52.0	4.9	4.9
30	40.0	3.8	3.8
40	32.9	3.1	3.1
50	28.0	2.6	2.6
60	24.6	2.3	2.3
70	21.9	2.1	2.1
80	19.8	1.9	1.9
90	18.1	1.7	1.7
100	16.7	1.6	1.6
110	15.6	1.5	1.5
120	14.6	1.4	1.4

SUMMARY TO OUTLET

			Vrequired	Vavailable*		
Tributary Area	0.861	ha				
Total 2yr Flow to Sewer	32.4	L/s	0	140	m^3	Ok
Target	32.4	L/s				
Non-Tributary Area	0.055	ha				
Total 2yr Flow Uncontrolled to ROW	7.2	L/s				

Project #160402157, 2655 Queensview Drive
Modified Rational Method Calculations for Storage

Subdrainage Area: R102B — Roof
Area (ha): 0.24 — Maximum Storage Depth: 150 mm
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.6	118.7	10.3	108.4	65.0	131.0	0.0
20	120.0	79.7	10.9	68.8	82.6	141.9	0.0
30	91.9	61.1	11.2	49.9	89.8	146.4	0.0
40	75.1	49.9	11.3	38.7	92.8	148.3	0.0
50	64.0	42.5	11.3	31.2	**93.7**	148.8	0.0
60	55.9	37.1	11.3	25.9	93.2	148.5	0.0
70	49.8	33.1	11.2	21.9	91.8	147.7	0.0
80	45.0	29.9	11.2	18.7	90.0	146.5	0.0
90	41.1	27.3	11.1	16.2	87.7	145.1	0.0
100	37.9	25.2	11.0	14.2	85.2	143.5	0.0
110	35.2	23.4	10.9	12.5	82.5	141.9	0.0
120	32.9	21.9	10.8	11.1	79.7	140.1	0.0

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	148.8	0.1	11.3	93.7	95.6	0.0

Subdrainage Area: EX CIST 6 & EX CIST 7 — External - Tributary
Area (ha): 0.019
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qext (L/s)
10	178.56	9.23	9.2
20	119.95	6.20	6.2
30	91.87	4.75	4.7
40	75.15	3.88	3.9
50	63.95	3.30	3.3
60	55.89	2.89	2.9
70	49.79	2.57	2.6
80	44.99	2.32	2.3
90	41.11	2.12	2.1
100	37.90	1.96	2.0
110	35.20	1.82	1.8
120	32.89	1.70	1.7

Subdrainage Area: UNC-1 — Uncontrolled - Non-Tributary
Area (ha): 0.05
C: 0.78

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)
10	178.6	21.0	21.0
20	120.0	14.1	14.1
30	91.9	10.8	10.8
40	75.1	8.8	8.8
50	64.0	7.5	7.5
60	55.9	6.6	6.6
70	49.8	5.9	5.9
80	45.0	5.3	5.3
90	41.1	4.8	4.8
100	37.9	4.5	4.5
110	35.2	4.1	4.1
120	32.9	3.9	3.9

SUMMARY TO OUTLET

			Vrequired	Vavailable*		
Tributary Area	0.861	ha				
Total 100yr Flow to Sewer	32.4	L/s	0	140	m^3	Ok
Target	32.4	L/s				
Non-Tributary Area	0.055	ha				
Total 100yr Flow Uncontrolled to ROW	21.0	L/s				

C.2 Storm Sewer Design Sheet

Stantec

2655 Queensview Drive

DATE: 2026-03-27
REVISION: 1
DESIGNED BY: MJS
CHECKED BY: MW

FILE NUMBER: **160402157**

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
MINIMUM COVER: 2.00 m
TIME OF ENTRY 10 min
BEDDING CLASS = B

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)
R102B, R102A, CIST 1 - CIST 17, EX CIST 6, EX CIST 7	102	101	0.44	0.00	0.00	0.00	0.44	0.58	0.00	0.00	0.00	0.253	0.253	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	29.5	29.5	83.4	1.2	300	300	CIRCULAR	PVC	-	1.00	96.2	**86.71%**	1.37
	101	100	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.253	0.000	0.000	0.000	0.000	0.000	0.000	10.01	76.75	104.12	122.05	178.43	0.0	29.5	83.3	4.7	300	300	CIRCULAR	PVC	-	1.00	96.2	**86.67%**	1.37
																				10.07									300	300							

Appendix D External Report Excerpts

D.1 Geotechnical Report Excerpts

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a one-storey commercial building located within the middle of the site, with the remainder of the site being occupied by asphalt-surfaced parking areas, landscaped areas, and mature trees. The site is bordered by residential properties to the north, by commercial properties to the east and west, and by Queensview Drive, followed by a commercial property to the south.

The ground surface across the subject site gradually slopes downward toward the east, with approximate geodetic elevations ranging from 67.1 m to 68.5 m. The subject site is situated approximately 0.5 m above the grade of the surrounding roadways and properties. Reference should be made to Figure 1 – Key Plan in Appendix 2 of the current report.

4.2 Subsurface Profile

Overburden

Generally, the subsurface soil profile encountered at the test hole locations consists of a thin layer of asphalt and/or topsoil followed by fill extending to a maximum depth of 2.4 m below the existing grade, underlain by glacial till deposit over bedrock surface. The glacial till deposit was generally observed to consist of undisturbed, compact to very dense, brown to grey silty sand with variable amounts of clay, gravel, cobbles, and boulders.

Further, an undisturbed, compact, brown to grey silty sand layer was encountered only in BH 3-25 at a depth ranging from 1.1 to 2.4 m below the existing ground surface. The silty sand layer is underlain by an undisturbed, very dense, grey glacial till deposit to a depth of 2.9 m below the existing ground surface at BH 3-25. Furthermore, the Glacial till is underlain by an undisturbed, brown silty sand, trace silt, and gravel to a depth of 3.7 m below the existing ground surface at BH 3-25. The sand layer is underlain by a glacial till deposit over bedrock at BH 3-25.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Bedrock

Bedrock consisting of grey limestone was encountered at boreholes BH 1-25 to BH 4-25 at depths ranging from 2.1 to 4.5 m below the existing ground surface. In addition, black shale bedrock was encountered only at borehole BH 4-25 at a depth between 2.4 and 3.7 m below the existing ground surface over grey limestone. The bedrock was cored at the locations of all boreholes. RDQ values indicate that the bedrock consists of very poor to excellent quality. The upper 1 to 1.5 m of the bedrock at was found to contain variable amounts of white spots likely containing calcium carbonates. The calcium carbonate spots may weaken the bedrock if exposed to excessive amounts of water.

Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at each test hole location.

Based on available geological mapping, the bedrock consists of shale of the Rockcliffe Formation and is expected to be encountered at depths ranging from 3 to 5 m.

4.3 Groundwater

Groundwater levels were recorded at each test hole location and presented in Table 1 below. The groundwater level readings are presented in the Soil Profile and Test Data sheets in Appendix 1.

Table 1 – Summary of Groundwater Levels

Borehole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Date Recorded
		Depth (m)	Elevation (m)	
BH 1-25	68.05	4.48	63.57	May 27, 2025
BH 2-25 *	67.13	1.44	65.69	
BH 3-25 *	67.19	3.85	63.34	
BH 4-25 *	68.50	4.13	64.37	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.
* - A monitoring well has been installed in the borehole.

It should be noted that surface water can become trapped within a backfilled borehole, that can lead to higher than typical groundwater level observations. The long-term groundwater levels can also be estimated based on the observed colour, consistency, and moisture content of the recovered soil samples.

Based on these observations, the long-term groundwater table can be expected at an approximate geodetic elevation between **63.3 and 64.3 m**. Groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater levels could vary at the time of construction.

5.0 Discussion

5.1 Geotechnical Assessment

Based on the available information, it is understood that the proposed development will consist of two 6-storey residential buildings, with one or two underground levels. From a geotechnical perspective, the subject site is suitable for the proposed development. It is recommended that the proposed residential buildings be founded on conventional spread footings bearing on an undisturbed, compact to very dense glacial till deposit, or clean surface sounded bedrock. External structures such as slab on grade buildings or canopies can be designed to be founded over an undisturbed, compact to dense silty sand bearing surface.

It is anticipated that the removal of bedrock and/or large boulders will be required for building construction and servicing installation. Therefore, the contractor should be prepared for bedrock removal and the presence of large boulders within the subject site.

Due to the absence of the sensitive silty clay deposit within the subject site, the proposed development will not be subjected to permissible grade raise restrictions.

The above and other considerations are further discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

Asphalt, topsoil, construction debris, and deleterious fill, such as those containing organic materials, should be removed from within the perimeter of the proposed buildings and from under paved areas, pipe bedding, or other settlement sensitive structures. Existing foundation walls and other construction debris should be entirely removed from within the building footprints. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below the final grade.

Care should be taken not to disturb adequate bearing soils below the founding level during site preparation activities. Disturbance of the subgrade may result in having to sub-excavate the disturbed material and the placement of additional suitable fill material.

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot ac \cdot \gamma \cdot H^2/g$ 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 0.351 g according to the OBC 2024. 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 the OBC 2024.

5.7 Pavement Structure

Pavement Structure Over Overburden

The following pavement structures may be considered for rigid pavement, car-only parking, and heavy traffic areas. The proposed pavement structures are shown in Tables 3, 4, and 5.

Table 3 - Recommended Rigid Pavement Structure - Lower Level

Thickness (mm)	Material Description
125	**Rigid Concrete Pavement -** 32 MPa concrete with air entrainment
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Table 4 - Recommended Pavement Structure - Car-Only Parking Areas and Fire-Truck Routes	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, fill or OPSS Granular B Type I or II material placed over in situ soil.	

Table 5 - Recommended Pavement Structure - Heavy-Truck Traffic and Loading Areas	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
450	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either in situ soil, 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 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 compaction equipment.

Pavement Structure Over Podium Deck Area and Raft Foundations

It is anticipated that the podium deck structure may be provided for landscaping or to accommodate car-only parking areas, access lanes, fire truck lanes, and loading areas. Based on the concrete slab subgrade for this area and/or over basements located over a raft slab, the pavement structure indicated in the following tables may be considered for design purposes:

Table 6 - Recommended Pavement Structure - Car-Only Parking Areas (Podium Deck and Raft Slab)	
Thickness (mm)	**Material Description**
50	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
200**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab *If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab. **Thickness is dependent on grade of insulation as noted in paragraphs below.	

Table 7 - Recommended Pavement Structure – Access Lane, Fire Truck Lane, Ramp, and Heavy Truck Parking Areas (Podium Deck and Raft Slab)	
Thickness (mm)	**Material Description**
40	**Wear Course** - HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Wear Course** - HL-8 or Superpave 19.0 Asphaltic Concrete
300**	**Base** - OPSS Granular A Crushed Stone
See Below*	**Thermal Break*** - Rigid insulation (See Paragraph Below)
n/a	**Waterproofing Membrane and Protection Board**
SUBGRADE – Reinforced Concrete Podium Deck or Raft Slab * If specified by others, not required from a geotechnical perspective. Also not required in basements over a raft slab. **Thickness is dependent on grade of insulation as noted in paragraphs below.	

The transition between the pavement structure over the podium deck subgrade and soil subgrade beyond the footprint of the podium deck is recommended to be transitioned to match the pavement structures provided in the following section. For this transition, a 5H:1V frost taper is recommended between the two subgrade surfaces. Further, the base layer thickness should be increased to a minimum thickness of 600 mm below the top of the podium slab a minimum of 1.5 m horizontally from the face of the foundation wall prior to providing the recommended taper.

Should the proposed podium deck be specified by others to be provided a thermal break by the use of a layer of rigid insulation below the pavement structure, its placement within the pavement structure is recommended to be as per the above-noted tables. The layer of rigid insulation is recommended to consist of a DOW Chemical High Load (HI-40) extruded polystyrene. The pavement structures' base layer thickness in Tables 6 and 7 may be reduced by 25 mm if HI-100 is considered for this project. It should be noted that Styrofoam rigid insulation is not considered suitable for this application.

7.0 Recommendations

It is recommended that the following be carried out by Paterson once preliminary and future details of the proposed development have been prepared:

- ❑ Review preliminary and detailed grading, servicing, landscaping, and structural plan(s) from a geotechnical perspective.
- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if not designed by Paterson, prior to construction, if applicable.
- ❑ Review of architectural plans pertaining to waterproofing systems, underfloor drainage systems, and waterproofing details for elevator shafts.

It is a requirement for the foundation design data provided herein to be applicable that a material testing and observation program be performed by the geotechnical consultant. The following aspects of the program should be performed by Paterson:

- ❑ Review and inspection of the installation of the foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Observation of driving and re-striking of all pile foundations.
- ❑ Sampling and testing of the concrete and fill materials.
- ❑ 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 that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

All excess soil must be handled as per ***Ontario Regulation 406/19: On-Site and Excess Soil Management***.

D.2 SWM Guidelines for the Pinecrest Creek/Westboro Area Table 3-1

Table 3.1: SWM Guidelines for the Pinecrest Creek / Westboro Study Area

	Development Type	Runoff Volume Reduction	Water Quality	Water Quantity	
			TSS Removal	Flood Flow Management	Erosion Control
Commercial/Institutional and Industrial Developments - <u>discharging directly to Ottawa River Parkway (ORP) pipe</u> *					
5	a) sites with soil infiltration rates ≥ 1 mm/hour	Minimum on-site retention of the 10 mm design storm.	On-site removal of 80% of TSS.	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's Sewer Design Guideline (Section 8.3.7.3, revised Sept. 2008).	Not applicable
	b) site's soil infiltration rates < 1 mm/hour	If the entire property is underlain by native soils with infiltration rates less than 1 mm/hr, no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. A green roof and/or rain harvesting measures could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS.	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's Sewer Design Guideline (Section 8.3.7.3, revised Sept. 2008).	Not applicable
Residential Development Requiring Site Plan Control Approval - <u>discharging upstream of Ottawa River Parkway (ORP) pipe inlet</u>					
6	a) sites with soil infiltration rates ≥ 1 mm/hour	Minimum on-site retention of the 10 mm design storm.	Inherent TSS removal due to on-site retention of the 10 mm and detention of the 25 mm design storms.	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's Sewer Design Guideline (Section 8.3.7.3, revised Sept. 2008).	Control (detain) the runoff from the 25 mm design storm such that the peak outflow from the site does not exceed 5.8 L/s/ha.
	b) site's soil infiltration rates < 1 mm/hour	If the entire property is underlain by native soils with infiltration rates less than 1 mm/hr, no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. A green roof and/or rain harvesting measures could be implemented to provide further runoff volume reduction.	Inherent TSS removal due to on-site retention in landscaped areas and detention of the 25 mm design storm.	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's Sewer Design Guideline (Section 8.3.7.3, revised Sept. 2008).	Control (detain) the runoff from the 25 mm design storm such that the peak outflow from the site does not exceed 5.8 L/s/ha.
Residential Development Requiring Site Plan Control Approval - <u>discharging directly to Ottawa River Parkway (ORP) pipe</u>					
7	a) sites with soil infiltration rates ≥ 1 mm/hour	Minimum on-site retention of the 10 mm design storm.	Inherent TSS removal due to on-site retention of the 10 mm design storm.	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's Sewer Design Guideline (Section 8.3.7.3, revised Sept. 2008).	Not applicable
	b) site's soil infiltration rates < 1 mm/hour	If the entire property is underlain by native soils with infiltration rates less than 1 mm/hr, no infiltrating SWM measures may be used. A minimum depth of 300 mm of amended soil shall be provided below all front yard landscaped areas. A green roof and/or rain harvesting measures could be implemented to provide further runoff volume reduction.	Inherent TSS removal from on-site retention in landscaped areas.	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's Sewer Design Guideline (Section 8.3.7.3, revised Sept. 2008).	Not applicable

***Infiltration measures should not be used on sites or source areas where the land use or activity could generate higher concentrations of hydrocarbons, trace metals or toxicants than are found in typical stormwater runoff (e.g., vehicle refueling, handling areas for hazardous materials, etc.). This would include retail gasoline outlet sites due to the potential for spills. In addition, these measures should be sited so that they will not receive runoff from high traffic areas where large amounts of de-icing salts are used. The design of these systems shall be in accordance with the guidance in the Stormwater Management Planning and Design Manual (MOE, 2003) and the Low Impact Development Stormwater Management Planning and Design Guide (CVC & TRCA, 2010).

Note: For a mixed use property, if surface parking has been provided the site will be considered commercial. If surface parking has not been provided, the site will be considered residential for the purposes of applying the SWM criteria in this table.

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Stantec Consulting Ltd.
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4
stantec.com