2475 REGINA STREET - ADEQUACY OF SERVICES REPORT

Stantec Project No. 160401689

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Prepared for:
Windmill Development Group
400-300 Richmond Road
Ottawa ON K1Z 6X6

Prepared by:
Stantec Consulting Ltd.
300-1331 Clyde Avenue
Ottawa ON K2C 3G4

Project Number: 160401689

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0	Adequacy of Services Report	Micheal Wu, N. Nwanise	16/05/2022	Dustin Thiffault	16/05/2022	Peter Moroz	17/05/2022

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Prepared by: ______________________
Signature

Michael Wu, EIT

Reviewed by: ______________________
Signature

Nwanise Nwanise, EIT

Approved by: ______________________
Signature

Dustin Thiffault, P. Eng.

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1 Introduction

Stantec Consulting Ltd. has been commissioned by Windmill Development Group to prepare the following adequacy of services report in support of the rezoning application for the proposed development located at 2475 Regina Street.

The property area is bound by the Byron Linear Tramway Park and a former Ottawa Transportation Commission streetcar right-of-way to the north, Lincoln Heights Road and Regina Street to the west, Sir John A. Macdonald Parkway and Pinecrest Creek to the east and Richmond Road to the south. There is an existing one-storey long term care home operated by Parkway House within the overall property area, which will be removed to allow for the proposed development. The current site is zoned as “O1”: Parks and Open Space Zone when viewed through GeoOttawa. It is assumed that this zoning is in error, and that the adjacent R5C zoning applies. The key plan is illustrated in **Figure 1**.

The proposed development area (1.04 ha) consists of two 6-storey apartment buildings (Parkside and East Towers); an 18-storey apartment building and a 24-storey apartment building. The buildings are to contain a total of 522 units consisting of 17 studio apartment units, 254 one-bedroom units, 205 two-bedroom units, 34 three-bedroom units, 12 long term care beds, and approximately 2814 m^2 of amenity space. Circulation within the property will be provided by internal roads and pedestrian walkways. Surface parking for 12 vehicles, two levels of underground parking with a total of 253 spaces and 510 bicycle parking spaces are also proposed.

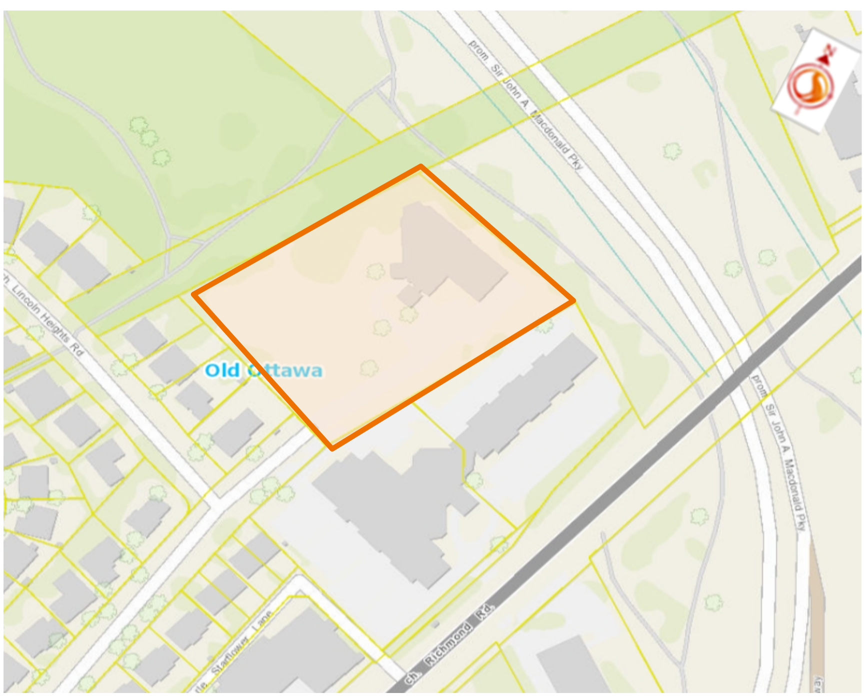


Figure 1: Site map (2475 Regina Street. Proposed Site Highlighted in Orange)

1.1 Objective

This servicing report has been prepared to present a servicing scheme that is free of conflicts and presents the most suitable servicing approach that complies with the relevant city design guidelines. Infrastructure requirements for water supply, sanitary sewer, and storm sewer services are presented in this report.

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

- Potable Water Servicing
    1. Estimate water demands to characterize the feed for the proposed development which will be serviced from the existing 150 mm diameter watermain on Regina Street and/or 203 mm diameter watermain on Lincoln Heights Road.
    2. Watermain servicing for the development is to be able to provide average day, maximum day, and peak hour demands (i.e., non-emergency conditions) at pressures within the allowable range of 276 to 552 kPa (40 to 80 psi)
    3. Under fire flow (emergency) conditions with maximum day demands, the water distribution system is to maintain a minimum pressure greater than 140 kPa (20 psi)
- Wastewater Servicing
    4. Estimate wastewater flows generated by the development and size sanitary sewers which will outlet to the existing 200 mm diameter sanitary sewer located on Regina Street.
- Stormwater Management and Servicing
    5. Determine the stormwater management storage requirements to meet the allowable release rate based on SWM Guidelines for the Pinecrest Creek / Westboro Study Area.
    6. Determine Post development peak 100-year flows
    7. Determine excess stormwater to be detained on-site to meet a 5-year pre-development target release rate.
    8. Define major and minor conveyance systems in conjunction with the preliminary grade control plan.
- Prepare a preliminary grading plan in accordance with the proposed site plan and existing grades.

The accompanying drawings included in **Appendix F** illustrate the preliminary internal servicing scheme for the site.

2 References

Documents referenced in preparation of this Adequacy of Services report for 2475 Regina Street include:

- *City of Ottawa Design Guidelines – Water Distribution*, City of Ottawa. July 2010 (including all subsequent technical bulletins).
- *City of Ottawa Sewer Design Guidelines (SDG),* City of Ottawa, October 2012 (including all subsequent technical bulletins).
- *Geotechnical Investigation,* Proposed Mixed-Use Development 2475 Regina Street, Ottawa, Ontario, Prepared for Parkway House Development Fund LP by Paterson Group, August 2021.
- *Stormwater Management Guidelines for the Pinecrest Creek/Westboro Area Final Report,* Prepared for Planning and Infrastructure, City of Ottawa by J.F. Sabourin and Associates Inc., May 2019.
- *Water Supply for Public Fire Protection,* Fire Underwriters Survey, 2020.

3 Potable Water Servicing

3.1 Background

The subject is located within Pressure Zone 1W of the City of Ottawa's water distribution system. The proposed development will be serviced by the existing 150 mm diameter watermain on Regina Street and 203mm watermain on Lincoln Heights Road. To create a suitable water service connection to the development, two connections to the existing watermains on Lincoln Heights and Regina Street are required to provide redundancy and looping benefits. Additionally, two new fire hydrants are also proposed to be installed on the site as shown on **SSP-1 Drawing** in **Appendix F**. The location of the water services within the property area will be coordinated with the building's architect to accommodate the underground parking structure on Level P1 and P2.

3.2 Water Demands

3.2.1 DOMESTIC WATER DEMANDS

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

Maximum day (MXDY) demands were determined by multiplying the AVDY demands by a factor of 2.5 for residential areas. Peak hourly (PKHR) demands were determined by multiplying the MXDY demands by a factor of 2.2 for residential areas. The estimated demands are summarized in **Table 3–1** below.

Table 3–1: Estimated Water Demands

Unit Type	No. of Units	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Studio	17	24	0.08	0.19	0.42
1 Bedroom	254	356	1.15	2.88	6.34
2 Bedroom	205	431	1.40	3.49	7.67
3 Bedroom	34	105	0.34	0.85	1.88

LTC 1 Bedroom	12	12	0.06	0.08	0.15
Total	**522**	**927**	**3.02**	**7.50**	**16.46**

3.2.2 FIRE FLOW DEMANDS

Fire flow requirements were estimated using Fire Underwriters Survey (FUS) methodology. The 24-storey East Tower and the 7-storey Parkway House are determined to have the largest fire flow demand at approximately 6,000 L/min (100.0 L/s). The FUS estimate considers a building of non-combustible construction type with a two-hour fire separation provided between each floor, but without full protections of vertical openings (one hour fire rating). As a result, the 'gross construction area' of the two largest floors (floors with the largest footprint, 1280 and 1191 m2 respectively) + 50% of the gross construction area of all floors immediately above them up to a maximum of eight was used for the purpose of the FUS calculation, as per page 22 of the *Fire Underwriters Survey's Water Supply for Public Fire Protection*, 2020. Additionally, it is anticipated that the building will be equipped with an automatic sprinkler system that is fully supervised and conforms to the NFPA 13 standard. Detailed fire flow calculations per the FUS methodology are provided in **Appendix A.2**.

3.2.3 BOUNDARY CONDITIONS

The boundary conditions provided by the City of Ottawa on April 21, 2022 as depicted in Table 3–2: Boundary Conditions**Table 3**–2 identifies the hydraulic boundary conditions for the site and have been used to determine the residual watermain pressures on Regina Street and Lincoln Heights Road.

Table 3–2: Boundary Conditions

	Connection at Regina Street	Connection at Lincoln Heights Road
Min. HGL (m)	108.3	108.3
Max. HGL (m)	115.3	115.8
Max. Day + Fire Flow (m)	190.5	190.5

An anticipated finished floor elevation of 65.5m at Regina Street will serve as the ground elevation for the calculation of residual pressures at ground level. On-site pressures are expected to range from **60.6** to **70.5 psi** (417.8 to 486.08 kPa) under normal operating conditions. These values are within the normal operating pressure range of 50 to 80 psi (344.7 to 551.6 kPa) and no less than 40 psi (275.8 kPa), as defined by the City of Ottawa's design guidelines. Booster pumps internal to the buildings will be required to provide adequate pressures for upper storeys. These pumps are to be designed by the buildings' mechanical engineer.

The boundary conditions provided for the proposed development under maximum day demands demonstrate that a fire flow rate of 100.0 L/s is available with a residual pressure above the required minimum 20 psi (137.9 kPa). This demonstrates that sufficient fire flow is available for the proposed development.

Based on these results, there is currently adequate supply and pressure in the water distribution system to meet the domestic and fire flow demands expected from the new development.

4 Wastewater Servicing

As illustrated on **Drawing SA-1**, sanitary servicing for the proposed development will be provided through a 200 mm diameter sanitary sewer flowing into the existing 200 mm diameter sanitary sewer within Regina Street. The existing onsite sanitary sewers are to be removed to allow for the new construction. Two sanitary sewer stubs are proposed to effectively convey wastewater flows from the proposed buildings to the existing manhole on Regina Street as shown in **Appendix C.3**. The location and layout of the sanitary network within the proposed parking structure will be coordinated with the mechanical and structural engineer and addressed at the detailed design stage.

The proposed development is to contain a total estimated population of 915 persons using the City of Ottawa's recommended population densities. The anticipated wastewater peak flow generated from the proposed development is summarized in **Table 4–1,** while the sanitary sewer design sheet is included in **Appendix C.1**.

Table 4–1: Estimated Wastewater Peak Flow

Residential Peak Flows				Infiltration Flow (L/s)	Total Peak Flow (L/s)
No. of Units	Population	Peak Factor	Peak Flow (L/s)		
522	927	3.06	9.18	0.23	9.42

1. Average residential sanitary flow = 280 L/p/day per City of Ottawa Sewer Design Guidelines
2. Peak factor for residential units calculated using Harmon's formula, using a Harmon correction factor of 0.8.
3. Apartment population estimated based on 1.4 persons/unit for studio and one-bedroom apartments, 2.1 persons/unit for two-bedroom apartments, and 3.1 persons/unit for three-bedroom apartments.
4. Infiltration flow = 0.33 L/s/ha

The city has confirmed that the Lincoln Heights Pumping Station is at capacity, and as such, based on the projected wastewater peak flow from the proposed development, the pumping station will need to be upgraded before the proposed development may go ahead. For correspondence with City of Ottawa staff, please see **Appendix C.2**.

A backflow preventer will be required for the proposed buildings in accordance with the City of Ottawa Sewer Design Guidelines. This requirement will be coordinated with the building's mechanical engineer.

The drains within the underground parking garage will need to be pumped and ultimately outlet to the proposed sanitary service. The design of these drains, internal plumbing, and associated pumping system is to be completed by the building's mechanical engineer.

5 Stormwater Management and Servicing

5.1 Existing Conditions and SWM Criteria

The proposed development area (1.040 ha) currently consists of an existing one-storey long term care home on the eastern area of the site, an access road and parking lot, and green landscaped areas. The existing building within the development area will be removed to allow for a 24-storey apartment building. The pre-development imperviousness of the proposed development area is 30.1 % (C = 0.41), while the anticipated post-development imperviousness of the proposed development area is 66.5 % (C = 0.67). **Drawing EXSD-1** (Existing Storm Drainage Plan) in **Appendix D.2** shows the existing drainage plan.

Stormwater runoff from the development site will ultimately be directed to an existing 300 mm diameter storm sewer within Regina Street. A 300 mm diameter storm service lateral is proposed to service the proposed development. Based on the preliminary finished floor elevation of the underground parking and the elevation of the existing storm sewer on Regina Street, it is anticipated that a sump pump will be required as part of the building internal plumbing system. The functional servicing storm drainage plan is shown on **Drawing SD-1** (Functional Storm Drainage Plan) in **Appendix D.5**.

Stormwater to be generated by the proposed development will be controlled on site and will discharge at a restricted release rate to the existing storm sewer on Regina Street via a single connection.

The design methodology for the stormwater management (SWM) component of the development has been determined through assessment of predevelopment conditions and review of the SWM Guidelines for the Pinecrest Creek/Westboro Area, and are as follows:

- Post-development peak flows up to 100-year storm event are to be controlled to the pre-development peak 5-year release rate. Excess stormwater is to be detained on-site with a minimum on-site retention of the 10 mm design storm.
- The 5-year storm event peak release rate was determined using IDF information derived from the Meteorological Services of Canada rainfall data taken from the MacDonald-Cartier International Airport and collected between 1966 to 1997.
- Calculated predevelopment runoff coefficient based on existing imperviousness or 0.5, whichever is less.
- A calculated time of concentration that cannot be less than 10 minutes.
- Quality control measures of 80 % TSS removal are to be provided on-site.

Other criteria considered in the SWM design are described in Sections 5 and 8 of the Ottawa Sewer Design Guidelines (October 2012) and all subsequent technical bulletins.

5.2 Stormwater Quantity Control

The Modified Rational Method (MRM) was employed to evaluate the rate and volume of runoff expected to be generated during post-development conditions. The pre-development release rate for the area has been determined using the 5-year storm event IDF curves as provided within the City of Ottawa's *Sewer Design Guidelines*. The predevelopment condition runoff coefficient was calculated using the existing conditions of the site as C = 0.41 and used to determine the target release from the site. A time of concentration for the pre-development area (10 minutes) was assigned based on the relatively small size of the site, well-drained impervious area, and its proximity to the existing drainage outlet on Regina Street.

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

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

Where:

Q – Peak flow rate, L/s

C – Site Runoff Coefficient

I – Rainfall intensity, mm/hr (per City of Ottawa IDF curves)

A – Drainage Area, ha

$$\text{Intensity (mm/hr)} = \frac{998.071}{(10+6.053)^{0.814}} = 104.19\ \text{mm/hr}$$

$$Q = 2.78(0.41)(104.19\text{mm/hr})(1.04\ \text{ha}) = \mathbf{123.5\ L/s}$$

Using the Rational Method, pre-development peak flow was determined to be 123.5 L/s. Post development flows shall be restricted to the established target release rate.

5.2.1 ROOFTOP STORAGE

It is anticipated that building rooftops will provide storage for runoff from larger events. Rooftop storage will be achieved by installing restricted flow roof drains. The following calculations assume the roof will be equipped with standard Watts Model R1100 Accuflow Roof Drains or approved equivalent, see **Appendix D.1** for Modified Rational Method design sheet. Controlled roof release is to be directed to the proposed 300mm storm service lateral for the development.

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

Table 5–1: Roof Control Areas

Roof ID	Accutrol Weir Setting	# of Drains	100-yr Release Rate (L/s)	100-yr Storage Required (cu.m)
ROOF-1	25% Open	6	5.5	26.9
ROOF-2	25% Open	8	7.5	52.3
ROOF-3	25% Open	8	7.4	42.4

5.2.2 UNCONTROLLED AREAS

One uncontrolled area (UNC-1) cannot be graded to enter the site storm sewer system and as such will sheet drain to the northern property boundary as per existing conditions (see **Drawing SD-1**). Uncontrolled release rates identified in **Table 5-2** below have been subtracted from the total allowable site release rate.

Table 5–2: Uncontrolled Runoff

Area ID	Area (ha)	Runoff 'C' (100-Year)	100-Year Uncontrolled Runoff (L/s)
UNC-1	0.05	0.25	6.2

5.2.3 SURFACE/SUBSURFACE STORAGE

Surface runoff outside of the extent of the building underground parking areas is anticipated to be provided within ponding areas through the use of inlet control devices (ICDs) on associated catch basins. ICDs are to be sized at detailed design to ensure surface ponding does not occur for design storms up to and including the 2-year event.

Surface runoff within the extent of the building underground parking areas is anticipated to be directed to a perimeter LID feature to the north for storage both at the surface and subsurface, and with eventual controlled release to be recaptured via building plumbing through the use of ICDs on inlet sewers. Storage volumes are required to attenuate peak flows from surface parking lot and landscaped areas within the site to meet the target release rate.

Table 5-3 below demonstrates the anticipated storage and release rates from controlled areas. It is of note that controlled outflow from area CISTRN is proposed to be directed through building internal plumbing with eventual outlet to the 300mm storm sewer on Regina Street, and may require discharge to be pumped should internal building plumbing layout not be conducive to gravity discharge. Details of the LID and onsite storm detention will be provided at detailed design stage and will be coordinated with the architect, structural and mechanical engineer.

Table 5–3: Controlled Flow Area Discharge Rates

Area ID	Storage Type / Location	100-Year Release Rate (L/s)	Required Volume (cu.m)
ICD-1	Surface of access/parking areas	45.0	33.8
CISTRN, LID-1, LID-2	Within LID Feature	52.0	67.6

Based on results presented in **Table 5-4** below, the proposed stormwater management scheme is anticipated to be sufficient to meet the desired target release rate for the site.

Table 5–4: 100-year Storage Volume and Release Rate Summary

Catchment Type	Catchment ID	100-Year Release Rate (L/s)	Storage Required (m^3)
Building Rooftops	ROOF-1, ROOF-2, ROOF-3	20.4	121.6
Surface / Subsurface Storage	CISTRN, ICD-1, LID-1, LID-2	97.0	101.4
Uncontrolled	UNC-1	6.2	-
Total		**123.5**	**223.0**

5.3 Stormwater Quality Control

The Stormwater Management Guidelines for the Pinecrest Creek/Westboro Study Area outlines a requirement for stormwater quality control measures for the site to meet long term removal of 80% TSS. A Stormceptor is proposed at the storm sewer intended to service surface parking areas outside of the limit of underground parking to ensure quality treatment of runoff from proposed access areas. The proposed stormceptor will be sized at the detailed design stage to meet the desired water quality requirement for its assigned capture area. The remainder of drainage directed to the LID feature at the north of the property will require similar levels of quality control treatment, in this case to be provided within a subsurface infiltration trench below the LID feature itself. Required volumes of storage may be determined through Table 3.2 of the MECP's Stormwater Management Planning and Design Manual for infiltration facilities as noted in the table below:

Table 5–5: Quality Control Storage Volumes

Area ID	Storage Type / Location	Tributary Area (ha)	Imperviousness	Required Volume (cu.m)
CISTRN, LID-1, LID-2	Within Subsurface Trench of LID Feature	0.42	62%	13.9

Required storage volumes are assumed to be provided via a 0.5m deep, 3m wide and 70m long clear stone trench (40% porosity) located below connections to internal building plumbing as demonstrated on **Drawing SD-1** to allow retention and treatment of runoff volumes noted above with eventual infiltration of captured flows. The proposed trench would be able to store approximately 42m3 of runoff, with the remaining 25.6m3 required for quantity control volumes noted in the section above provided within a surface swale component of the LID feature. Runoff from controlled roof areas is assumed to be clean and will not require further quality control treatment.

5.4 Runoff Volume Control

The Stormwater Management Guidelines for the Pinecrest Creek/Westboro Study Area outlines a requirement for capture, retention and infiltration of all site runoff for all storm events up to and including the 10mm storm event. Runoff volume reduction is anticipated to be provided by intensive green roof areas for rooftop catchments, and via the northerly LID feature for surface runoff from impervious areas.

Based on rooftop areas as measured from the current site plan, the roofs will be required to capture approximately 34m3 of rainfall within green roof regions. The remaining surface impervious areas (measuring approximately 0.419ha) will require approximately 41.9m3 of storage to meet the desired runoff volume reduction criteria. It is anticipated that the required storage volume will be provided within the LID feature in conjunction with required quality control storage volumes as noted in the section above.

6 Grading and Drainage

The proposed re-development site measures approximately 1.040 ha in area. The existing topography across the site is relatively sloped, and currently drains from south to north, with overland flow generally being directed to the edge of the existing multi-use pathway in the Byron Tramway Linear Park. A preliminary grading plan (see **Drawing GP-1**) has been prepared to verify stormwater management calculations, to allow for positive drainage away from the face of proposed buildings, and adhere to any geotechnical restrictions (see **Section 9**) for the site. Site grading has been established to provide emergency overland flow routes required for stormwater management in accordance with City of Ottawa requirements. No grade raise restriction has been identified for this site.

The subject site is graded to provide an emergency overland flow route to Regina Street and the Byron Linear Tramway Park for storm flows exceeding those generated by the 100-year design storm.

7 Utilities

Hydro Ottawa has existing utility plant in the area, which will be used to service the site. The detailed design of the required utility services will be further investigated as part of the composite utility planning process, which will follow design circulation for the servicing plans. The relocation of existing utilities in conflict with the proposed development will be coordinated with the individual utility providers as part of the site plan approval process.

8 Erosion Control During Construction

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

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit the extent of the exposed soils at any given time.
3. Re-vegetate exposed soils as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with geotextiles, geogrid, or synthetic mulches.
6. Provide sediment traps and basins during dewatering works.
7. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
8. Schedule the construction works at times which avoid flooding due to seasonal rains.

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

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

9 Geotechnical Investigation

Paterson Group (Paterson) was commissioned by Parkway House Development Fund to conduct a geotechnical investigation for the proposed development at 2475 Regina Street in Ottawa, Ontario.

The field program for the geotechnical investigation was carried out from July 29 to August 3, 2021 and consisted of advancing a total of seven boreholes to a maximum depth of 17.5 metres below the existing grade. Locations of the drilled boreholes were determined in the field by Paterson personnel taking into consideration of underground utilities and site features.

Monitoring wells were installed in boreholes BH 1-21, BH 6-21 and BH 7-21 to permit monitoring of the groundwater levels after the completion of the sampling program. Flexible standpipes were also installed in the remaining boreholes. All monitoring wells should be decommissioned in accordance with Ontario Regulations O.Reg 903 by a qualified licensed well technician and prior to construction.

As described in the report by Paterson, the subsurface profile at the test hole locations consists of a topsoil layer underlain by an approximate 0.8 to 1.8 m thick fill layer. The fill material was generally observed to consist of brown silty sand and/or clay with gravel, cobbles, boulders and varying amounts of topsoil and organics. The fill was observed to be underlaid with a hard to very stiff brown silty clay deposit, which was underlaid by a glacial till deposit.

The bedrock was observed to consist of grey quartz sandstone and based on the RQDs of the recovered bedrock core, was generally weathered and of poor quality. At borehole BH 1-21, the bedrock was cored at an approximate depth of 13.8 m and extending to a depth of 17.5 m below the existing ground surface.

Groundwater level readings were measured in the monitoring wells installed at boreholes BH 1-21, BH 6-21 and BH 7-21, as well as the meters installed at the remaining boreholes. Based on these observations, the long-term groundwater level is anticipated at a depth of approximately 4 to 5 m below ground surface. However, as groundwater levels are subject to seasonal fluctuations, they could vary at the time of construction.

Based on Paterson's recommendations, the site is considered suitable for the proposed development. It is recommended that the proposed high-rise buildings be founded on a raft foundation placed on an undisturbed, compact to dense glacial till bearing surface.

Furthermore, it is recommended that the low-rise building and portions of the underground parking levels be supported on a conventional spread footings bearing on undisturbed compact to dense glacial till.

The recommended rigid pavement structure is further presented in **Table 9–1** below.

Table 9–1: Recommended Pavement Structure

Material	Lower Parking Level	Car Only Parking Areas	Access Lanes and Heavy Loading Parking Areas
Exposure Class C2 – 32 MPa Concrete (5 to 8 % Air Entrainment)	125 mm	-	-
Wear Course – HL-3 or Superpave 12.5 Asphaltic Concrete	-	50 mm	40 mm
Wear Course – HL-8 or Superpave 19.0 Asphaltic Concrete	-	-	50 mm
BASE – OPSS Granular A Crushed Stone	300 mm	150 mm	150 mm
SUBBASE – OPSS Granular B Type II	-	300 mm	300 mm

Refer to the full geotechnical report attached in **Appendix E.1** for further details.

10 Approvals and Permits

Ontario Ministry of Environment, Conservation and Parks (MECP) Environmental Compliance Approvals (ECAs, formerly Certificates of Approval (CofA)) under the Ontario *Water Resources Act* are not anticipated for the proposed storm and sanitary sewers servicing the proposed site so long as the development remains under singular ownership. An ECA application may be required for the LID feature given its use in meeting runoff volume targets specified by the City of Ottawa.

A MECP Permit to Take Water (PTTW) may be required for the site as some of the proposed works may be below the groundwater elevation shown in the geotechnical report. The geotechnical consultant shall determine whether a PTTW is required at the detailed design stage/prior to construction. No other approval has been identified to be required at this point.

11 Conclusions

11.1 Potable Water Servicing

Based on the potable water servicing analysis the proposed network can service the subject site and meet all the servicing requirements as per City of Ottawa standards under typical demand conditions (peak hour and minimum hour conditions) as well as under emergency fire demand conditions (maximum day + fire flow).

11.2 Wastewater Servicing

The City has confirmed that the Lincoln Heights Pumping Station is at capacity and would need to be upgraded to accept the peak sanitary flows from the proposed development. Existing on-site sanitary sewers are to be removed, and the proposed sanitary sewer connection will be routed around the underground parking garage limits.

11.3 Stormwater Management and Servicing

The proposed stormwater management plan follows local and provincial standards. Rooftop storage with controlled roof drains, green roof, and surface/subsurface storage via LID feature located north of the underground parking area has been proposed to limit peak storm sewer inflows to the existing 300 mm diameter storm sewers along Regina Street ROW to the required pre-development levels.

11.4 Grading

Grading for the site has been designed to provide an emergency overland flow route as per City requirements and reflects recommendations in the Geotechnical Investigation Report prepared by Paterson Group Inc. in August 2021. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

11.5 Utilities

Hydro Ottawa has existing utility plant in the area, which will be used to service the site. The detailed design of the required utility services will be further investigated as part of the composite utility planning process, which will follow design circulation for the servicing plans. The relocation of existing utilities in conflict with the proposed development will be coordinated with the individual utility providers as part of the site plan approval process.

11.6 Approvals and Permits

An MECP Environmental Compliance Approval is not expected to be required for storm and sanitary sewers within the subject site. An ECA application may be required for the LID feature given its use in meeting runoff volume targets specified by the City of Ottawa. Requirements for a Permit to Take Water will be confirmed by the geotechnical consultant. The Rideau Valley Conservation Authority will need to be consulted to obtain municipal approval for site development. No other approval requirements from other regulatory agencies are anticipated.

APPENDICES

Appendix A Potable Water Servicing

A.1 Water Demand Calculations

2475 Regina St., Ottawa, ON - Domestic Water Demand Estimates

Site Plan provided by Diamond Schmitt Architects (2022-04-06)

Project No. 160401689

Densities as per City Guidelines:		
Apartment Units		
1 Bedroom	1.4	ppu
2 Bedroom	2.1	ppu
3 Bedroom	3.1	ppu
LTC Units		
1 Bedroom	1.0	ppu

Building ID	Amenity areas (m²)	No. of Units	Population	Daily Rate of Demand [1 2] (L/cap/day or L/ha/day)	Avg Day Demand		Max Day Demand [3 4]		Peak Hour Demand [3 4]	
					(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
Apartment Units										
Studio		17	24	280	4.6	0.08	11.6	0.19	25.5	0.42
1 Bedroom		254	356	280	69.1	1.15	172.9	2.88	380.3	6.34
2 Bedroom		205	431	280	83.7	1.40	209.3	3.49	460.4	7.67
3 Bedroom		34	105	280	20.5	0.34	51.2	0.85	112.7	1.88
LTC 1 Bedroom		12	12	400	3.3	0.06	5.0	0.08	9.0	0.15
Total Site :		**522**	**927**		**181.3**	**3.02**	**449.9**	**7.50**	**987.9**	**16.46**

1 Average day water demand for residential areas: 280 L/cap/d

2 Average day water demand for Amenity/common areas: 28,000 L/ha/d (Based on commercial water demand rates)

3 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 for residential
   peak hour demand rate = 2.2 x maximum day demand rate for residential

4 Water demand criteria used to estimate peak demand rates for amenity/common areas 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

A.2 Fire Flow Requirements Per FUS Guidelines

FUS Fire Flow Calculation Sheet

Stantec Project #: 160401689
Project Name: 2475 Regina Street
Date: 2022-04-27
Fire Flow Calculation #: 3
Description: Residential

Notes: 24-storey building with amenities. Information taken from Site plan by Diamond Schmitt Architects dated March 23, 2022.

Step	Task	Notes								Value Used	Req'd Fire Flow (L/min)
1	Determine Type of Construction	Type II - Noncombustible Construction / Type IV-A - Mass Timber Construction								0.8	-
2	Determine Effective Floor Area	Sum of Two Largest Floors + 50% of Six Additional Floors					Vertical Openings Protected?			NO	-
		1191	1191	1191	1191	1191	1191	1191	1191	5955	-
3	Determine Required Fire Flow	(F = 220 x C x $A^{1/2}$). Round to nearest 1000 L/min								-	12000
4	Determine Occupancy Charge	Limited Combustible								-15%	10200
5	Determine Sprinkler Reduction	Conforms to NFPA 13								-30%	-5100
		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	> 30	0	0	0-20	Type V		NO	0%	408
		East	> 30	0	0	0-20	Type V		NO	0%	
		South	20.1 to 30	13	17	> 100	Type I-II - Unprotected Openings		NO	4%	
		West	20.1 to 30	14	7	81-100	Type I-II - Unprotected Openings		YES	0%	
7	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min									6000
		Total Required Fire Flow in L/s									100.0
		Required Duration of Fire Flow (hrs)									2.00
		Required Volume of Fire Flow (m^3)									720

A.3 Boundary Conditions

From:	Thiffault, Dustin
To:	Moroz, Peter; Wu, Michael
Subject:	RE: 2475 Regina - Water boundary conditions
Date:	Thursday, 21 April, 2022 15:19:11
Attachments:	image001.png Water Supply for Public Fire Protection in Canada 2020.pdf

I feel that the 200mm line on Lincoln Heights is probably well looped given two 200mm connections to the 300mm line further west on Regina in conjunction with the smaller main within Assaly to the 300mm line on Carling, so I'm a bit doubtful that bumping up the Assaly main would have much of an effect on overall system pressure.

One possibility to check could be from that the City presented an HGL of 87.6 at the Lincoln fields main (or alternately the Regina main within the City ROW and not at the dead end connection) with the fire + MXDY draw of 207L/s. Based on our site elevation, this would leave a pressure head of 31.1psi at the main, implying 11 or so psi of headloss along the internal 200mm loop. If we want to demonstrate more flow across that line, we could try to model it ourselves based on our own estimated lengths of main (as well as the revised two-main connection loop at the corner of Regina/Lincoln Fields) to see if we have less of a modeled headloss, and therefore more flow available at the site.

I was casually reviewing the FUS guidelines in preparation for our meeting with John Bougadis earlier, and noticed that the FUS has updated their guidelines recently (within the last two weeks) and have removed the draft note from their new 202 guidelines, so I guess they're official now. Micheal, can you have a look and see if there are revisions to our FUS calcs? I see that now buildings with 2hr rated floor assemblies are considered fire-resistive, so we may be just fine with 207L/s.

Cheers,

Dustin Thiffault P.Eng.
Project Engineer

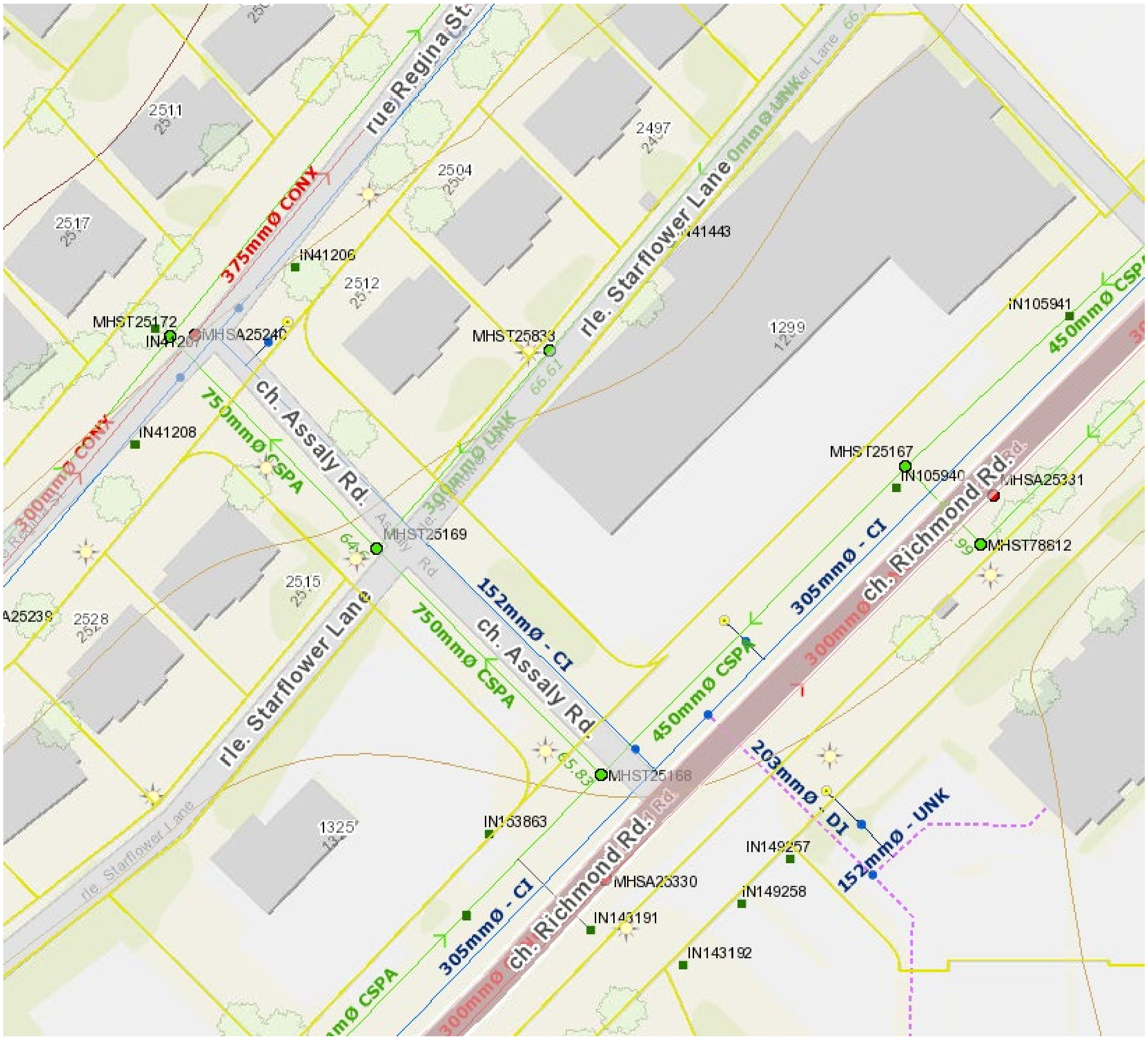
Mobile: 343-996-2211
dustin.thiffault@stantec.com

Stantec
300-1331 Clyde Avenue
Ottawa ON K2C 3G4

From: Moroz, Peter <peter.moroz@stantec.com>
Sent: Thursday, April 21, 2022 2:02 PM
To: Wu, Michael <Michael.Wu@stantec.com>; Thiffault, Dustin <Dustin.Thiffault@stantec.com>
Subject: RE: 2475 Regina - Water boundary conditions

I wonder if we increased the link on Assaly Rd. to 300mm if we could get the flows?

Dustin, what are our options here?



Peter

Peter Moroz P.Eng., MBA
Managing Principal, Community Development

Stantec
400 - 1331 Clyde Avenue Ottawa ON K2C 3G4
Cell: (613) 294-2851

peter.moroz@stantec.com

From: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Sent: Thursday, April 21, 2022 1:19 PM
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Moroz, Peter <peter.moroz@stantec.com>; Thiffault, Dustin <Dustin.Thiffault@stantec.com>
Subject: Fw: 2475 Regina - Water boundary conditions

Hello Michael,
The system will not be able to provide the required fire flow.

The following are boundary conditions, HGL, for hydraulic analysis at 2475 Regina Street (zone 1W) assumed a looped private network to be connected to the 203 mm watermain on 2475 Regina Street and the 203 mm on Lincoln Heights Road (see attached PDF for location).

Both Connections
Minimum HGL: 108.3 m
Maximum HGL: 115.8 m
Available fire flow at 20 psi: 207 L/s, assuming a ground elevation of 65.7 m (Connection 1)
HGL of Connection 2 when Connection 1 is at 20 psi: 87.6 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.

Thanks
Eric Surprenant, CET
Sr, Project Manager, Infrastructure Projects, West
Planning, Infrastructure & Economic Development
613 580-2424 ext.: 27794

Please take note that due to current COVID situation, I am working remotely and Phone communications and messaging may not be reliable at this time. Preferred method of communications will be e-mails during this period. If your preference is telephone communication, please indicate this via e-mail and provide a contact telephone number.

Absence Alert:

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: April 14, 2022 10:25
To: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Cc: Moroz, Peter <peter.moroz@stantec.com>; dustin.thiffault@stantec.com <dustin.thiffault@stantec.com>
Subject: RE: 2475 Regina - Water boundary conditions

Hi Eric:

Attached is the updated boundary condition map with the connection points for the proposed 203 mm diameter watermain on site.

Please let me know if you have any more questions and comments.

Best regards,

Michael Wu, EIT
Civil Engineering Intern, Community Development

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

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

From: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Sent: Thursday, 14 April, 2022 09:41
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Moroz, Peter <peter.moroz@stantec.com>; Thiffault, Dustin <Dustin.Thiffault@stantec.com>
Subject: Re: 2475 Regina - Water boundary conditions

Hello Michael,

If you could update your map to include / confirm what the watermain size(s) are proposed for your site and confirm the connection points also on the map.

Understood that one of the connection points will be at dead end of Regina however, if you could mark and confirm the connection points.
Thanks
Eric Surprenant, CET
Sr, Project Manager, Infrastructure Projects, West
Planning, Infrastructure & Economic Development
613 580-2424 ext.: 27794

Please take note that due to current COVID situation, I am working remotely and Phone communications and messaging may not be reliable at this time. Preferred method of communications will be e-mails during this period. If your preference is telephone communication, please indicate this via e-mail and provide a contact telephone number.

Absence Alert:

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: April 14, 2022 09:03
To: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Cc: Moroz, Peter <peter.moroz@stantec.com>; dustin.thiffault@stantec.com <dustin.thiffault@stantec.com>
Subject: RE: 2475 Regina - Water boundary conditions

Hi Eric:

While I have attached a map with the watermain location, size, and potential connection locations when I submitted the 2475 Regina Street boundary condition request to Wendy Tse two weeks ago, I am happy to attach the map, the calculation sheets, and the site development statistics again for your reference.

The proposed development, consisting of three blocks of apartment buildings with a total of 510 apartment units and projected to serve 915 residents, would be served by a looped watermain. We intend to connect to the existing 203 mm diameter watermain on Regina Street, and the existing 203 mm diameter watermain on Lincoln Heights Road.

The water demand for the proposed development are as follows:

- Average Day Demand: 2.97 L/s (178.0 L/min)
- Maximum Day Demand: 7.42 L/s (444.9 L/min)
- Peak Hour Demand: 16.31 L/s (978.9 L/min)
- Fire Flow Demand: 283.3 L/s (17000 L/min) (Based on FUS1999)

We appreciate your time looking into this for us, and feel free to reach out to me if you have any more questions or comments.

Best regards,

Michael Wu, EIT
Civil Engineering Intern, Community Development

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

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

From: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Sent: Thursday, 14 April, 2022 07:29
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Moroz, Peter <peter.moroz@stantec.com>
Subject: 2475 Regina - Water boundary conditions

Hello Michael, in order to provide the Boundary Conditions for the above location we would require a figure with the proposed watermain location, size and connection locations.

If you could provide this information at your earliest convenience,
Best regards,

Eric Surprenant, CET
Sr, Project Manager, Infrastructure Projects, West
Planning, Infrastructure & Economic Development
613 580-2424 ext.: 27794

Please take note that due to current COVID situation, I am working remotely and Phone communications and messaging may not be reliable at this time. Preferred method of communications will be e-mails during this period. If your preference is telephone communication, please indicate this via e-mail and provide a contact telephone number.

Absence Alert:

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Appendix B Site Plan

diamond schmitt

LEVEL 01 FLOOR PLAN P102

Windmill OTTAWA PARKWAY 03/24/2022 1 : 500

Appendix C Sanitary Servicing

C.1 Sanitary Sewer Design Sheet

SUBDIVISION: Job Name	
DATE:	2022-05-16
REVISION:	1
DESIGNED BY:	MW
CHECKED BY:	-

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401689

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 / ONE BEDROOM	1.4	INSTITUTIONAL	28,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 2 BEDROOM	2.1	INFILTRATION	0.33 l/s/Ha	HARMON CORRECTION FACTOR	0.8
PERSONS / 3 BEDROOM	3.1	PERSONS - LTC ONE BEDROOM	1.0		

LOCATION			RESIDENTIAL AREA AND POPULATION										COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)	
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	UNITS 1 BEDROOM	2 BEDROOM	3 BEDROOM	LTC 1 BEDROOM	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)
PROPOSED DEVELOPMENT	BLDG	EX. SAN	0.326	271	205	34	12	927	0.326	927	3.057	9.183	0.000	0.000	0.000	0.000	0.000	0.000

LOCATION	INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE							
AREA ID NUMBER	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)
PROPOSED DEVELOPMENT	0.000	0.000	0.709	0.709	0.000	0.709	0.709	0.234	9.417	178.505	200	PVC	SDR 35	1.00	33.4	28.16%	1.05

VEL. (ACT.) (m/s)
0.76

C.2 Correspondence with City on Sanitary Sewer Capacity

From:	Moroz, Peter
To:	Surprenant, Eric; Wu, Michael
Cc:	Nwanise, Nwanise; Thiffault, Dustin; Tse, Wendy
Subject:	RE: 2475 Regina Street sanitary sewer capacity
Date:	Wednesday, 4 May, 2022 08:50:11

Eric, thank you for confirming. We will include this information in our servicing report. I assume this will need to be dealt with as part of conditions of approval. We will advise the client.

thx

Peter

Peter Moroz P.Eng., MBA
Managing Principal, Community Development

Stantec
400 - 1331 Clyde Avenue Ottawa ON K2C 3G4
Cell: (613) 294-2851

peter.moroz@stantec.com

From: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Sent: Wednesday, May 4, 2022 8:07 AM
To: Wu, Michael <Michael.Wu@stantec.com>
Cc: Moroz, Peter <peter.moroz@stantec.com>; Nwanise, Nwanise <Nwanise.Nwanise@stantec.com>; Thiffault, Dustin <Dustin.Thiffault@stantec.com>; Tse, Wendy <Wendy.Tse@ottawa.ca>
Subject: Re: 2475 Regina Street sanitary sewer capacity

Hello Michael and all,

We have had some back and forth with our Operations Group our Environmental Services Department and the Lincoln Height pumping station is at capacity and homes have experienced flooding during peak wet weather flows. Based on this information, we cannot allow development to go ahead until the station is upgraded. I am inquiring further as to any anticipated timing on the Lincoln Heights pump station upgrades.

Thanks

Eric Surprenant, CET
Sr, Project Manager, Infrastructure Projects, West
Planning, Infrastructure & Economic Development
613 580-2424 ext.: 27794

Please take note that due to current COVID situation, I am working remotely and Phone communications and messaging may not be reliable at this time. Preferred method of communications will be e-mails during this period. If your preference is telephone communication, please indicate this via e-mail and provide a contact telephone number.

Absence Alert:

From: Wu, Michael <Michael.Wu@stantec.com>
Sent: April 28, 2022 13:51
To: Surprenant, Eric <Eric.Surprenant@ottawa.ca>
Cc: Moroz, Peter <peter.moroz@stantec.com>; Nwanise, Nwanise <Nwanise.Nwanise@stantec.com>; dustin.thiffault@stantec.com <dustin.thiffault@stantec.com>
Subject: 2475 Regina Street sanitary sewer capacity

Hi Eric:

Hope you are doing well.

We are preparing an adequacy of services report in support of the proposed development on 2475 Regina Street. We would like to confirm if there is sufficient capacity downstream of the existing 200 mm diameter sanitary sewer on Regina Street and 375 mm diameter sanitary sewer on Lincoln Heights Road to receive an additional flow of 9.307 L/s from the site being the estimated peak flow from the development.

The proposed development area (1.035 ha) consists of a 7-storey building with six stories of residential units, an 18-storey residential high-rise building, and a 24-storey residential high-rise building. The building is to contain a total of 510 units consisting of 17 studio units, 254 one-bedroom units, 205 two-bedroom units, and 34 three-bedroom units. Internal circulation in the proposed development will be provided by access lanes for vehicles, surface parking for 12 vehicles, and two levels of underground parking with pedestrian access to the building.

Please find our sanitary sewer design sheet and location map attached for your information.

Thank you.

Michael Wu, EIT
Civil Engineering Intern, Community Development

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

Stantec

300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

C.3 Sanitary Sewer Functional Servicing Plan

Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

Legend

R100A / 0.31 / 50 — SANITARY DRAINAGE AREA ID#; POPULATION COUNT; SANITARY DRAINAGE AREA ha.

G100A / 0.11 / INFL — SANITARY DRAINAGE AREA ID#; INFILTRATION RATE OF 0.33 L/s/Ha APPLIED; SANITARY DRAINAGE AREA ha.

SANITARY DRAINAGE AREA

PROPOSED SANITARY MH AND SEWER

EXISTING SANITARY MH AND SEWER

Notes

SANITARY STATS

PARKWAY HOUSE
25 - 1 BEDROOM UNITS @ 1.4 PPU = 35 PEOPLE
26 - 2 BEDROOM UNITS @ 2.1 PPU = 55 PEOPLE
8 - 3 BEDROOM UNITS @ 3.1 PPU = 25 PEOPLE
12 - LONG TERM CARE UNITS @ 1.0 PPU = 12 PEOPLE

WEST TOWER
71 - 1 BEDROOM UNITS @ 1.4 PPU = 99 PEOPLE
82 - 2 BEDROOM UNITS @ 2.1 PPU = 172 PEOPLE
17 - STUDIO UNITS @ 1.4 PPU = 24 PEOPLE
12 - 3 BEDROOM UNITS @ 3.1 PPU = 37 PEOPLE

EAST TOWER
158 - 1 BEDROOM UNITS @ 1.4 PPU = 221 PEOPLE
106 - 2 BEDROOM UNITS @ 2.1 PPU = 223 PEOPLE
9 - 3 BEDROOM UNITS @ 3.1 PPU = 28 PEOPLE

TOTAL POPULATION = 931

Revision	By	Appd.	YY.MM.DD
1 ISSUED FOR REVIEW	JP	DT	22.05.16

File Name: 160401689-FSG.dwg	Dwn.	Chkd.	Dsgn.	YY.MM.DD
	JP	DT	JP	22.04.18

Permit-Seal

Client/Project

WINDMILL DEVELOPMENT GROUP LTD.

2475 REGINA STREET

Ottawa, ON

Title

FUNCTIONAL SANITARY DRAINAGE PLAN

Project No. 160401689

Scale 1:300 0 3 9 15m

Drawing No. SA-1

Sheet 5 of 5

Revision 1

W:\active\160401689\design\drawing\160401689-FSG.dwg
2022/05/17 9:21 AM By: Pallang, Jupiter

ORIGINAL SHEET - ARCH D

Appendix D Stormwater Servicing and Management

D.1 Modified Rational Method Calculations

Stormwater Management Calculations

File No: **160401689**
Project: **2475 Regina Street**
Date: **18-May-22**

SWM Approach:
Post-development to Pre-development flows

Post-Development Site Conditions:

Overall Runoff Coefficient for Site and Sub-Catchment Areas

Runoff Coefficient Table

Sub-catchment Area Catchment Type	ID / Description		Area (ha) "A"		Runoff Coefficient "C"	"A x C"		Overall Runoff Coefficient
Controlled - Tributary	ICD-1	Hard	0.168		0.9	0.151		
		Soft	0.062		0.2	0.012		
		Subtotal		0.23			0.1633	0.710
Uncontrolled - Tributary	LID-2	Hard	0.004		0.9	0.004		
		Soft	0.026		0.2	0.005		
		Subtotal		0.03			0.009	0.300
Uncontrolled - Tributary	LID-1	Hard	0.000		0.9	0.000		
		Soft	0.070		0.2	0.014		
		Subtotal		0.07			0.014	0.200
Uncontrolled - Tributary	UNC-1	Hard	0.000		0.9	0.000		
		Soft	0.050		0.2	0.010		
		Subtotal		0.05			0.01	0.20
Roof	ROOF-2	Hard	0.140		0.9	0.126		
		Soft	0.000		0.2	0.000		
		Subtotal		0.14			0.126	0.90
Roof	ROOF-1	Hard	0.080		0.9	0.072		
		Soft	0.000		0.2	0.000		
		Subtotal		0.08			0.072	0.90
Uncontrolled - Tributary	CSTN	Hard	0.247		0.9	0.222		
		Soft	0.073		0.2	0.015		
		Subtotal		0.32			0.2368	0.74
Roof	ROOF - 3	Hard	0.120		0.9	0.108		
		Soft	0.000		0.2	0.000		
		Subtotal		0.12			0.108	0.90
Total				**1.040**			**0.739**	
Overall Runoff Coefficient= C:								**0.71**

Total Roof Areas	**0.340 ha**	
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.650 ha**	
Total Tributary Area to Outlet	**0.990 ha**	
Total Uncontrolled Areas (Non-Tributary)	**0.050 ha**	1.185491071
Total Site	**1.040 ha**	

Stormwater Management Calculations

Project #160401689, 2475 Regina Street
Modified Rational Method Calculatons for Storage

2 yr Intensity
City of Ottawa

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

a =	732.951
b =	6.199
c =	0.81

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

5 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 1.04
C: 0.41

Target release from site

tc (min)	I (5 yr) (mm/hr)	Qtarget (L/s)
10	104.19	**123.51**

2 YEAR Modified Rational Method for Entire Site

Subdrainage Area: ICD-1 — Controlled - Tributary
Area (ha): 0.23
C: 0.71

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	34.87	34.87	0.00	**0.00**
20	52.03	23.62	23.62	0.00	**0.00**
30	40.04	18.18	18.18	0.00	**0.00**
40	32.86	14.92	14.92	0.00	**0.00**
50	28.04	12.73	12.73	0.00	**0.00**
60	24.56	11.15	11.15	0.00	**0.00**
70	21.91	9.95	9.95	0.00	**0.00**
80	19.83	9.00	9.00	0.00	**0.00**
90	18.14	8.24	8.24	0.00	**0.00**
100	16.75	7.60	7.60	0.00	**0.00**
110	15.57	7.07	7.07	0.00	**0.00**
120	14.56	6.61	6.61	0.00	**0.00**

Storage: e Above CB

Orifice Equation: = CdA(2gh)^0.5 — Where C = 0.61
Orifice Diameter: 127.00 mm
Invert Elevation 0.00 m
T/G Elevation 1.38 m
Max Ponding Depth 0.00 m
Downstream W/L 0.00 m

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	1.38	1.38	40.21	0.00	34.00	OK

Subdrainage Area: LID-2 — Uncontrolled - Tributary
Area (ha): 0.03
C: 0.30

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	1.92	1.92		
20	52.03	1.30	1.30		
30	40.04	1.00	1.00		
40	32.86	0.82	0.82		
50	28.04	0.70	0.70		
60	24.56	0.61	0.61		
70	21.91	0.55	0.55		
80	19.83	0.50	0.50		
90	18.14	0.45	0.45		
100	16.75	0.42	0.42		
110	15.57	0.39	0.39		
120	14.56	0.36	0.36		

Subdrainage Area: LID-1 — Uncontrolled - Tributary
Area (ha): 0.07
C: 0.20

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	2.99	2.99		
20	52.03	2.03	2.03		
30	40.04	1.56	1.56		
40	32.86	1.28	1.28		
50	28.04	1.09	1.09		
60	24.56	0.96	0.96		
70	21.91	0.85	0.85		
80	19.83	0.77	0.77		
90	18.14	0.71	0.71		
100	16.75	0.65	0.65		
110	15.57	0.61	0.61		
120	14.56	0.57	0.57		

Project #160401689, 2475 Regina Street
Modified Rational Method Calculatons for Storage

100 yr Intensity
City of Ottawa

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

a =	1735.688
b =	6.014
c =	0.820

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

100 YEAR Predevelopment Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 1.04
C: 0.41

Target release from site

tc (min)	I (100 yr) (mm/hr)	Q100yr (L/s)
10	104.19	**123.51**

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: ICD-1 — Controlled - Tributary
Area (ha): 0.23
C: 0.89

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	101.33	45.02	56.31	**33.78**
20	119.95	68.07	45.02	23.05	27.66
30	91.87	52.13	45.02	7.11	12.80
40	75.15	42.64	42.64	0.00	0.00
50	63.95	36.29	36.29	0.00	0.00
60	55.89	31.72	31.72	0.00	0.00
70	49.79	28.25	28.25	0.00	0.00
80	44.99	25.53	25.53	0.00	0.00
90	41.11	23.33	23.33	0.00	0.00
100	37.90	21.51	21.51	0.00	0.00
110	35.20	19.98	19.98	0.00	0.00
120	32.89	18.67	18.67	0.00	0.00

Storage: Surface Storage Above CB

Orifice Equation: Q = CdA(2gh)^0.5 — Where C = 0.61
Orifice Diameter: 127.00 mm
Invert Elevation 0.00 m
T/G Elevation 1.38 m
Max Ponding Depth 0.35 m
Downstream W/L 0.00 m

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	1.73	1.73	45.02	33.78	34.00	OK
					0.22	

Subdrainage Area: LID-2 — Uncontrolled - Tributary
Area (ha): 0.03
C: 0.38

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	5.58	5.58		
20	119.95	3.75	3.75		
30	91.87	2.87	2.87		
40	75.15	2.35	2.35		
50	63.95	2.00	2.00		
60	55.89	1.75	1.75		
70	49.79	1.56	1.56		
80	44.99	1.41	1.41		
90	41.11	1.29	1.29		
100	37.90	1.19	1.19		
110	35.20	1.10	1.10		
120	32.89	1.03	1.03		

Subdrainage Area: LID-1 — Uncontrolled - Tributary
Area (ha): 0.07
C: 0.25

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	8.69	8.69		
20	119.95	5.84	5.84		
30	91.87	4.47	4.47		
40	75.15	3.66	3.66		
50	63.95	3.11	3.11		
60	55.89	2.72	2.72		
70	49.79	2.42	2.42		
80	44.99	2.19	2.19		
90	41.11	2.00	2.00		
100	37.90	1.84	1.84		
110	35.20	1.71	1.71		
120	32.89	1.60	1.60		

Stormwater Management Calculations

Project #160401689, 2475 Regina Street
Modified Rational Method Calculatons for Storage

Subdrainage Area: UNC-1 Uncontrolled - Tributary
Area (ha): 0.05
C: 0.20

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	2.14	2.14		
20	52.03	1.45	1.45		
30	40.04	1.11	1.11		
40	32.86	0.91	0.91		
50	28.04	0.78	0.78		
60	24.56	0.68	0.68		
70	21.91	0.61	0.61		
80	19.83	0.55	0.55		
90	18.14	0.50	0.50		
100	16.75	0.47	0.47		
110	15.57	0.43	0.43		
120	14.56	0.40	0.40		

Subdrainage Area: ROOF-2 Roof
Area (ha): 0.14 Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.81	26.90	6.04	20.86	12.52	89.38	0.00
20	52.03	18.23	6.17	12.06	14.47	94.46	0.00
30	40.04	14.03	6.15	7.88	14.18	93.71	0.00
40	32.86	11.51	6.08	5.44	13.05	90.76	0.00
50	28.04	9.82	5.98	3.85	11.54	86.82	0.00
60	24.56	8.60	5.87	2.74	9.85	82.43	0.00
70	21.91	7.68	5.75	1.93	8.09	77.84	0.00
80	19.83	6.95	5.61	1.34	6.43	72.13	0.00
90	18.14	6.36	5.42	0.93	5.02	64.97	0.00
100	16.75	5.87	5.25	0.61	3.68	58.14	0.00
110	15.57	5.45	5.09	0.36	2.40	51.67	0.00
120	14.56	5.10	4.83	0.27	1.92	47.89	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	94.46	0.09	6.17	14.47	56.00	0.00

Subdrainage Area: ROOF-1 Roof
Area (ha): 0.08 Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.81	15.37	4.48	10.90	6.54	86.57	0.00
20	52.03	10.41	4.52	5.89	7.07	89.00	0.00
30	40.04	8.02	4.46	3.55	6.39	85.90	0.00
40	32.86	6.58	4.37	2.21	5.30	80.92	0.00
50	28.04	5.61	4.26	1.35	4.05	75.23	0.00
60	24.56	4.92	4.09	0.83	2.98	65.95	0.00
70	21.91	4.39	3.92	0.47	1.97	56.96	0.00
80	19.83	3.97	3.73	0.24	1.15	49.25	0.00
90	18.14	3.63	3.45	0.19	1.00	45.52	0.00
100	16.75	3.35	3.21	0.14	0.87	42.36	0.00
110	15.57	3.12	3.00	0.11	0.76	39.65	0.00
120	14.56	2.91	2.82	0.09	0.66	37.29	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	89.00	0.09	4.52	7.07	32.00	0.00

Subdrainage Area: CSTN Uncontrolled - Tributary
Area (ha): 0.32
C: 0.74

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	76.81	107.1	52.0	55.1	**33.1**
20	52.03	78.0	52.0	26.0	31.2
30	40.04	63.9	52.0	11.9	21.4
40	32.86	55.4	52.0	3.4	8.3
50	28.04	49.8	52.0	0.0	0.0
60	24.56	45.7	52.0	0.0	0.0
70	21.91	42.6	52.0	0.0	0.0
80	19.83	40.1	52.0	0.0	0.0
90	18.14	38.1	52.0	0.0	0.0
100	16.75	36.5	52.0	0.0	0.0
110	15.57	35.1	52.0	0.0	0.0
120	14.56	33.9	52.0	0.0	0.0

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
5-year Water Level	N/A	N/A	52.0	34.0	68.0	OK

Project #160401689, 2475 Regina Street
Modified Rational Method Calculatons for Storage

Subdrainage Area: UNC-1 Uncontrolled - Tributary
Area (ha): 0.05
C: 0.25

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	6.20	6.20		
20	119.95	4.17	4.17		
30	91.87	3.19	3.19		
40	75.15	2.61	2.61		
50	63.95	2.22	2.22		
60	55.89	1.94	1.94		
70	49.79	1.73	1.73		
80	44.99	1.56	1.56		
90	41.11	1.43	1.43		
100	37.90	1.32	1.32		
110	35.20	1.22	1.22		
120	32.89	1.14	1.14		

Subdrainage Area: ROOF-2 Roof
Area (ha): 0.14 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	178.56	69.50	7.07	62.42	37.45	130.35	0.00
20	119.95	46.68	7.34	39.35	47.22	140.69	0.00
30	91.87	35.76	7.44	28.32	50.97	144.67	0.00
40	75.15	29.25	7.47	21.78	52.26	146.04	0.00
50	63.95	24.89	7.47	17.42	52.26	146.04	0.00
60	55.89	21.75	7.45	14.30	51.49	145.23	0.00
70	49.79	19.38	7.42	11.96	50.24	143.89	0.00
80	44.99	17.51	7.37	10.14	48.65	142.22	0.00
90	41.11	16.00	7.33	8.67	46.84	140.30	0.00
100	37.90	14.75	7.27	7.48	44.87	138.21	0.00
110	35.20	13.70	7.22	6.48	42.79	136.00	0.00
120	32.89	12.80	7.16	5.64	40.63	133.71	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	146.04	0.15	7.47	52.26	56.00	0.00

Subdrainage Area: ROOF-1 Roof
Area (ha): 0.08 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	178.56	39.71	5.28	34.43	20.66	128.97	0.00
20	119.95	26.68	5.45	21.23	25.47	137.90	0.00
30	91.87	20.43	5.50	14.93	26.88	140.50	0.00
40	75.15	16.71	5.50	11.21	26.91	140.56	0.00
50	63.95	14.22	5.48	8.75	26.24	139.32	0.00
60	55.89	12.43	5.44	6.99	25.17	137.34	0.00
70	49.79	11.07	5.39	5.68	23.86	134.90	0.00
80	44.99	10.01	5.34	4.67	22.39	132.18	0.00
90	41.11	9.14	5.29	3.86	20.83	129.28	0.00
100	37.90	8.43	5.23	3.20	19.20	126.27	0.00
110	35.20	7.83	5.16	2.67	17.63	122.53	0.00
120	32.89	7.32	5.08	2.24	16.11	118.33	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	140.56	0.14	5.50	26.91	32.00	0.00

Subdrainage Area: CSTN Uncontrolled - Tributary
Area (ha): 0.32
C: 0.93

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	161.2	52.0	109.2	65.5
20	119.95	108.3	52.0	56.3	**67.6**
30	91.87	82.9	52.0	30.9	55.7
40	75.15	67.8	52.0	15.8	38.0
50	63.95	57.7	52.0	5.7	17.2
60	55.89	50.5	52.0	0.0	0.0
70	49.79	45.0	52.0	0.0	0.0
80	44.99	40.6	52.0	0.0	0.0
90	41.11	37.1	52.0	0.0	0.0
100	37.90	34.2	52.0	0.0	0.0
110	35.20	31.8	52.0	0.0	0.0
120	32.89	29.7	52.0	0.0	0.0

LID storage volume

	Stage	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Volume Check
100-year Water Level	N/A	N/A	52.0	67.6	68.0	OK

Stormwater Management Calculations

Project #160401689, 2475 Regina Street
Modified Rational Method Calculatons for Storage

Subdrainage Area: ROOF - 3   Roof
Area (ha): 0.12   Maximum Storage Depth: 150 mm
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	76.81	23.1	6.0	17.1	10.2	87.9	0.00
20	52.03	15.6	6.1	9.5	**11.4**	91.5	0.00
30	40.04	12.0	6.0	6.0	10.8	89.5	0.00
40	32.86	9.9	5.9	3.9	9.4	85.4	0.00
50	28.04	8.4	5.8	2.6	7.8	80.5	0.00
60	24.56	7.4	5.7	1.7	6.1	75.2	0.00
70	21.91	6.6	5.5	1.1	4.6	66.9	0.00
80	19.83	6.0	5.3	0.7	3.3	58.9	0.00
90	18.14	5.4	5.1	0.4	2.0	51.3	0.00
100	16.75	5.0	4.8	0.3	1.6	47.2	0.00
110	15.57	4.7	4.5	0.2	1.4	44.2	0.00
120	14.56	4.4	4.2	0.2	1.3	41.6	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	91.52	0.09	6.1	11.4	48.0	0.0

SUMMARY TO OUTLET

		Vrequired	Vavailable*		
Tributary Area	0.990 ha				
Total 5yr Flow to Sewer	109 L/s	67	238	m^3	Ok
Non-Tributary Area	0.150 ha				
Total 5yr Flow Uncontrolled	2 L/s				
Total Area	1.140 ha				
Total 5yr Flow	111 L/s				
Target	124 L/s				

Project #160401689, 2475 Regina Street
Modified Rational Method Calculatons for Storage

Subdrainage Area: ROOF - 3   Roof
Area (ha): 0.12   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	178.56	59.6	7.1	52.5	31.5	129.6	0.00
20	119.95	40.0	7.3	32.7	39.3	139.2	0.00
30	91.87	30.6	7.4	23.3	41.9	142.4	0.00
40	75.15	25.1	7.4	17.7	**42.4**	143.1	0.00
50	63.95	21.3	7.4	14.0	41.9	142.4	0.00
60	55.89	18.6	7.3	11.3	40.7	141.0	0.00
70	49.79	16.6	7.3	9.3	39.1	139.0	0.00
80	44.99	15.0	7.2	7.8	37.3	136.8	0.00
90	41.11	13.7	7.2	6.5	35.3	134.3	0.00
100	37.90	12.6	7.1	5.5	33.2	131.7	0.00
110	35.20	11.7	7.0	4.7	31.0	129.0	0.00
120	32.89	11.0	7.0	4.0	28.8	126.3	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	143.09	0.14	7.4	42.4	48.0	0.0

SUMMARY TO OUTLET

		Vrequired	**Vavailable***		
Tributary Area	0.990 ha				
Total 100yr Flow to Sewer	117 L/s	223	238	m^3	Ok
Non-Tributary Area	0.120 ha				
Total 100yr Flow Uncontrolled	6 L/s				
Total Area	1.110 ha				
Total 100yr Flow	124 L/s				
Target	124 L/s				

Project #160401689, 2475 Regina Street
Roof Drain Design Sheet, Area ROOF-3
Standard Watts Model R1100 Accuflow 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	27	0	0	0.025
0.050	0.0006	0.0050	2	0.050	107	2	2	0.050
0.075	0.0007	0.0057	6	0.075	240	4	6	0.075
0.100	0.0008	0.0063	14	0.100	427	8	14	0.100
0.125	0.0009	0.0069	28	0.125	667	14	28	0.125
0.150	0.0009	0.0076	48	0.150	960	20	48	0.150

Drawdown Estimate

Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.6	308.2	1.6	0.08561
5.8	743.6	4.2	0.29217
14.0	1303.2	8.2	0.65418
27.6	1953.3	13.6	1.19675
47.8	2671.1	20.2	1.93872

Rooftop Storage Summary

Total Building Area (sq.m)		1200
Assume Available Roof Area (sq.	80%	960
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)		48
Estimated 100 Year Drawdown Time (h)		1.7

* 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.

From Watts Drain Catalogue

Head (m) L/s	Open	75%	50%	**25%**	Closed
0.025	0.3155	0.31545	0.31545	**0.31545**	0.31545
0.050	0.6309	0.6309	0.6309	**0.6309**	0.6309
0.075	0.9464	0.86749	0.78863	**0.70976**	0.6309
0.100	1.2618	1.10408	0.94635	**0.78863**	0.6309
0.125	1.5773	1.34067	1.10408	**0.86749**	0.6309
0.150	1.8927	1.57726	1.2618	**0.94635**	0.6309

Calculation Results

	5yr	**100yr**	**Available**
Qresult (cu.m/s)	0.006	0.007	-
Depth (m)	0.092	0.143	0.150
Volume (cu.m)	11.4	42.4	48.0
Draintime (hrs)	0.5	1.7	

Project #160401689, 2475 Regina Street
Roof Drain Design Sheet, Area ROOF-1
Standard Watts Model R1100 Accuflow 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	0.0019	0	0.025	18	0	0	0.025
0.050	0.0006	0.0038	1	0.050	71	1	1	0.050
0.075	0.0007	0.0043	4	0.075	160	3	4	0.075
0.100	0.0008	0.0047	9	0.100	284	5	9	0.100
0.125	0.0009	0.0052	19	0.125	444	9	19	0.125
0.150	0.0009	0.0057	32	0.150	640	13	32	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.0	274.0	1.0	0.0761
3.9	661.0	2.8	0.2597
9.3	1158.4	5.5	0.58149
18.4	1736.2	9.0	1.06378
31.9	2374.3	13.5	1.72331

Rooftop Storage Summary

Total Building Area (sq.m)		800
Assume Available Roof Area (sq.r	80%	640
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		6
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		32
Estimated 100 Year Drawdown Time (h)		1.5

* 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.

From Watts Drain Catalogue

Head (m) L/s	Open	75%	50%	**25%**	Closed
0.025	0.3155	0.31545	0.31545	**0.31545**	0.31545
0.050	0.6309	0.6309	0.6309	**0.6309**	0.6309
0.075	0.9464	0.86749	0.78863	**0.70976**	0.6309
0.100	1.2618	1.10408	0.94635	**0.78863**	0.6309
0.125	1.5773	1.34067	1.10408	**0.86749**	0.6309
0.150	1.8927	1.57726	1.2618	**0.94635**	0.6309

Calculation Results

	5yr	**100yr**	**Available**
Qresult (cu.m/s)	0.005	0.005	-
Depth (m)	0.089	0.141	0.150
Volume (cu.m)	7.1	26.9	32.0
Draintime (hrs)	0.4	1.5	

Project #160401689, 2475 Regina Street
Roof Drain Design Sheet, Area ROOF-2
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	31	0	0	0.025
0.050	0.0006	0.0050	2	0.050	124	2	2	0.050
0.075	0.0007	0.0057	7	0.075	280	5	7	0.075
0.100	0.0008	0.0063	17	0.100	498	10	17	0.100
0.125	0.0009	0.0069	32	0.125	778	16	32	0.125
0.150	0.0009	0.0076	56	0.150	1120	24	56	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.8	359.6	1.8	0.09988
6.7	867.5	4.9	0.34086
16.3	1520.5	9.6	0.76321
32.1	2278.8	15.8	1.39621
55.7	3116.3	23.6	2.26184

Rooftop Storage Summary

Total Building Area (sq.m)		1400
Assume Available Roof Area (sq.m)	80%	1120
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)		56
Estimated 100 Year Drawdown Time (h)		2.1

* 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.

From Watts Drain Catalogue

Head (m) L/s	Open	75%	50%	**25%**	Closed
0.025	0.3155	0.31545	0.31545	**0.31545**	0.31545
0.050	0.6309	0.6309	0.6309	**0.6309**	0.6309
0.075	0.9464	0.86749	0.78863	**0.70976**	0.6309
0.100	1.2618	1.10408	0.94635	**0.78863**	0.6309
0.125	1.5773	1.34067	1.10408	**0.86749**	0.6309
0.150	1.8927	1.57726	1.2618	**0.94635**	0.6309

Calculation Results

	5yr	100yr	Available
Qresult (cu.m/s)	0.006	0.007	-
Depth (m)	0.094	0.146	0.150
Volume (cu.m)	14.5	52.3	56.0
Draintime (hrs)	0.7	2.1	

D.2 Correspondence with Rideau Valley Conservation Authority (RVCA)

Wu, Michael

From:	Wu, Michael
Sent:	Thursday, 5 May, 2022 15:19
To:	jamie.batchelor@rvca.ca
Cc:	Nwanise, Nwanise; Moroz, Peter; Thiffault, Dustin
Subject:	2475 Regina Street Stormwater Quality Control Criteria
Attachments:	160401689-FSG-SD-1.pdf; 2475 Regina Street Site Map.pdf

Good afternoon, Jamie.

I hope you are well.

I am writing to request stormwater quality control criteria for a proposed development at 2475 Regina Street. The site is bound by Regina Street to the west, the Byron Linear Tramway Park to the north, the Sir John A. Macdonald (Ottawa River) Parkway to the east, and Richmond Road to the south. Stantec is preparing an adequacy of services report in support of a re-zoning application.

The proposed development area (1.04 ha) comprises of a 7-storey building with six storeys of residential units, an 18-storey residential high-rise building, and a 24-storey residential high-rise building. The buildings are to contain a total of 510 residential units consisting of 17 studio apartment units, 254 one-bedroom units, 205 two-bedroom units, and 34 three-bedroom units, and approximately 1751 m^2 of exterior amenity space. Internal circulation in the proposed development will be provided by access lanes for vehicles, surface parking for 12 vehicles, and two levels of underground parking with pedestrian access to the building and 510 bicycle parking spaces.

A location map and storm drainage plan are attached for your information.

Thank you for your time in looking into this on our behalf. Please do not hesitate to reach out to me if you have any questions or require any additional information.

Regards,

Michael Wu, EIT
Civil Engineering Intern, Community Development

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

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

Wu, Michael

From: Wu, Michael
Sent: Tuesday, 10 May, 2022 11:31
To: jamie.batchelor@rvca.ca
Cc: Moroz, Peter; Thiffault, Dustin; Nwanise, Nwanise
Subject: Follow-up on 2475 Regina Street Stormwater Quality Control Criteria Request

Importance: High

Good morning, Jamie.

I want to follow up on the stormwater quality control criteria request for the proposed development at 2475 Regina Street submitted on May 5th.

Is there any additional information you would like us to provide to supplement the request at this time, or if there is a timeline on when can we expect the stormwater quality control criteria for the site?

Best regards,

Michael Wu, EIT
Civil Engineering Intern, Community Development

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

Stantec
300 - 1331 Clyde Avenue
Ottawa ON K2C 3G4

D.3 SWM Guidelines for the Pinecrest Creek/Westboro Study Area (From JFSA Stormwater Management Guidelines for the Pinecrest Creek/Westboro Area Final Report, May 2019)

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

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

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

Development Type		Runoff Volume Reduction	Water Quality	Water Quantity	
			TSS Removal	Flood Control	Erosion Control
Commercial/Institutional and Industrial Developments - discharging directly to Ottawa River Parkway pipe (ORPP)					
5	all soil infiltration rates	A minimum on-site retention of the 10 mm design storm; refer to LID references(i) for guidance on prudent approach to planning infiltration-based LID best management practices. Assumptions re: non-viability of infiltration measures must be substantiated. A green roof, rain harvesting measures and/or a combination of detention/retention measures(ii) could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS; some of which would be accomplished by on-site retention of first 10 mm of rainfall.	The more stringent of the following criteria will govern: i) 1:100 year discharge from site not to exceed 33.5 L/s/ha) or; ii) Requirements of City of Ottawa Sewer Design Guidelines.	Not applicable
Residential Development Requiring Site Plan Control Approval - discharging upstream of Ottawa River Parkway pipe (ORPP) inlet					
6	all soil infiltration rates	A minimum on-site retention of the 10 mm design storm; refer to LID references(i) for guidance on prudent approach to planning infiltration-based LID best management practices. Assumptions re: non-viability of infiltration measures must be substantiated. A green roof, rain harvesting measures and/or a combination of detention/retention measures(ii) could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS; some of which would be accomplished by on-site retention of first 10 mm of rainfall and detention of the 25 mm design storms(iii).	The more stringent of the following criteria will govern: i) 1:100 year discharge from site not to exceed 33.5 L/s/ha); or ii) Requirements of City of Ottawa Sewer Design Guidelines.	Control (detain) the runoff from the 25 mm design storm(iii) such that the peak outflow from the site does not exceed 5.8 L/s/ha.
Residential Development Requiring Site Plan Control Approval - discharging directly to Ottawa River Parkway pipe (ORPP)					
7	all soil infiltration rates	A minimum on-site retention of the 10 mm design storm; refer to LID references(i) for guidance on prudent approach to planning infiltration-based LID best management practices. Assumptions re: non-viability of infiltration measures must be substantiated. A green roof, rain harvesting measures and/or a combination of detention/retention measures(ii) could be implemented to provide further runoff volume reduction.	On-site removal of 80% of TSS; some of which would be accomplished by on-site retention of first 10 mm of rainfall.	The more stringent of the following criteria will govern: i) 1:100 year discharge from site not to exceed 33.5 L/s/ha); or ii) Requirements of City of Ottawa Sewer Design Guidelines.	Not applicable

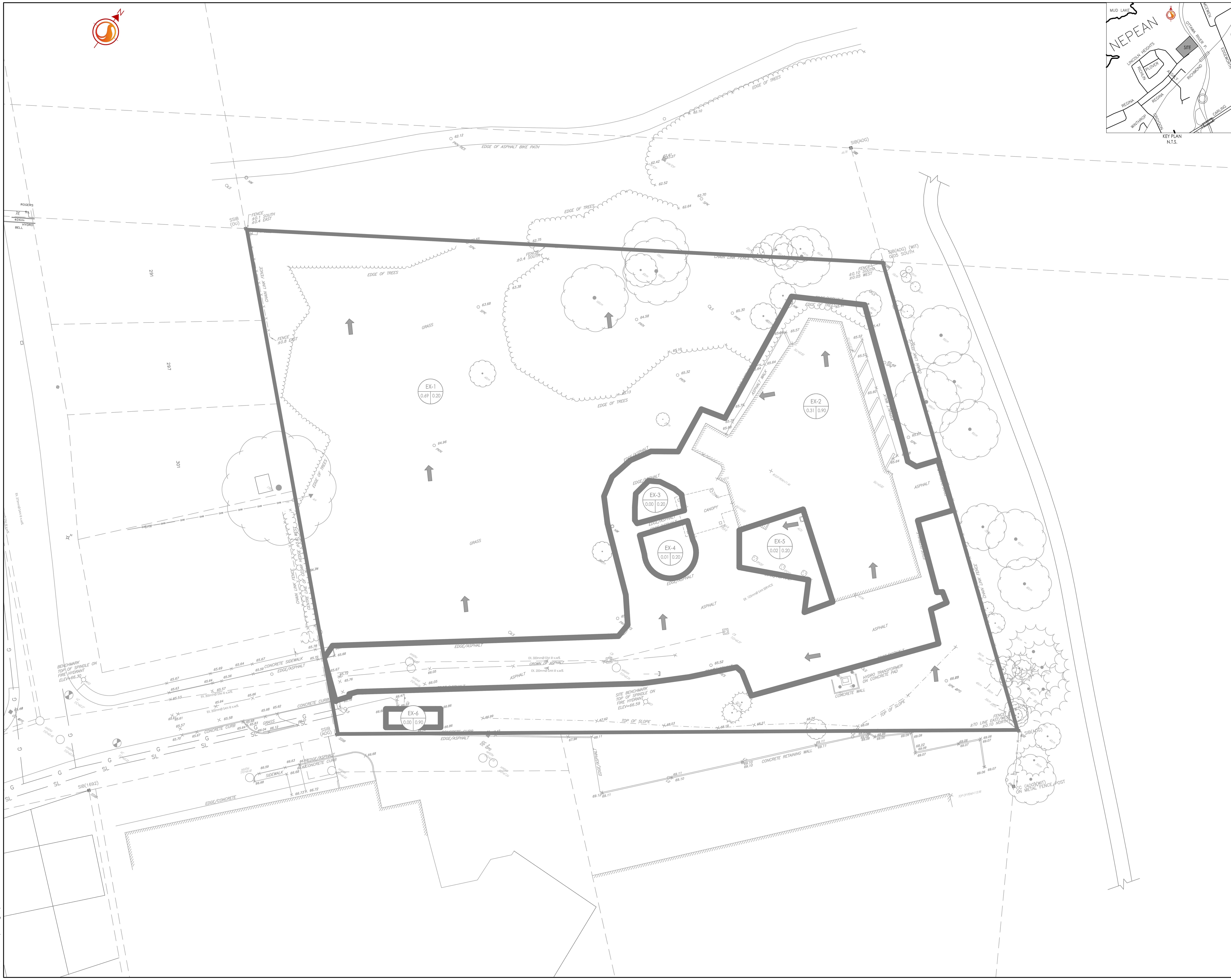
Notes:

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

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

(iii) 25 mm 4-hour Chicago design storm

D.4 Existing Storm Drainage Plan



Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

Legend

- L101A — EX.AREA ID
- 1.00 | 0.85 — EX.RUNOFF COEFFICIENT
- EX.STORM DRAINAGE AREA ha.
- EX.STORM DRAINAGE BOUNDARY
- EX.DIRECTION OF OVERLAND FLOW
- EX.STORM MH AND SEWER
- EX. CATCHBASIN

Revision	By	Appd.	YY.MM.DD
1 ISSUED FOR REVIEW	JP	DT	22.04.30

File Name: 160401689-FSG.dwg	Dwn.	Chkd.	Dsgn.	YY.MM.DD
	JP	DT	JP	22.04.18

Permit-Seal

Client/Project
WINDMILL DEVELOPMENT GROUP LTD.

2475 REGINA STREET

Ottawa, ON

Title
EXISTING
STORM DRAINAGE PLAN

Project No. 160401689

Scale 1:300 0 3 9 15m

Drawing No. EXSD-1

Sheet 3 of 4

Revision 1

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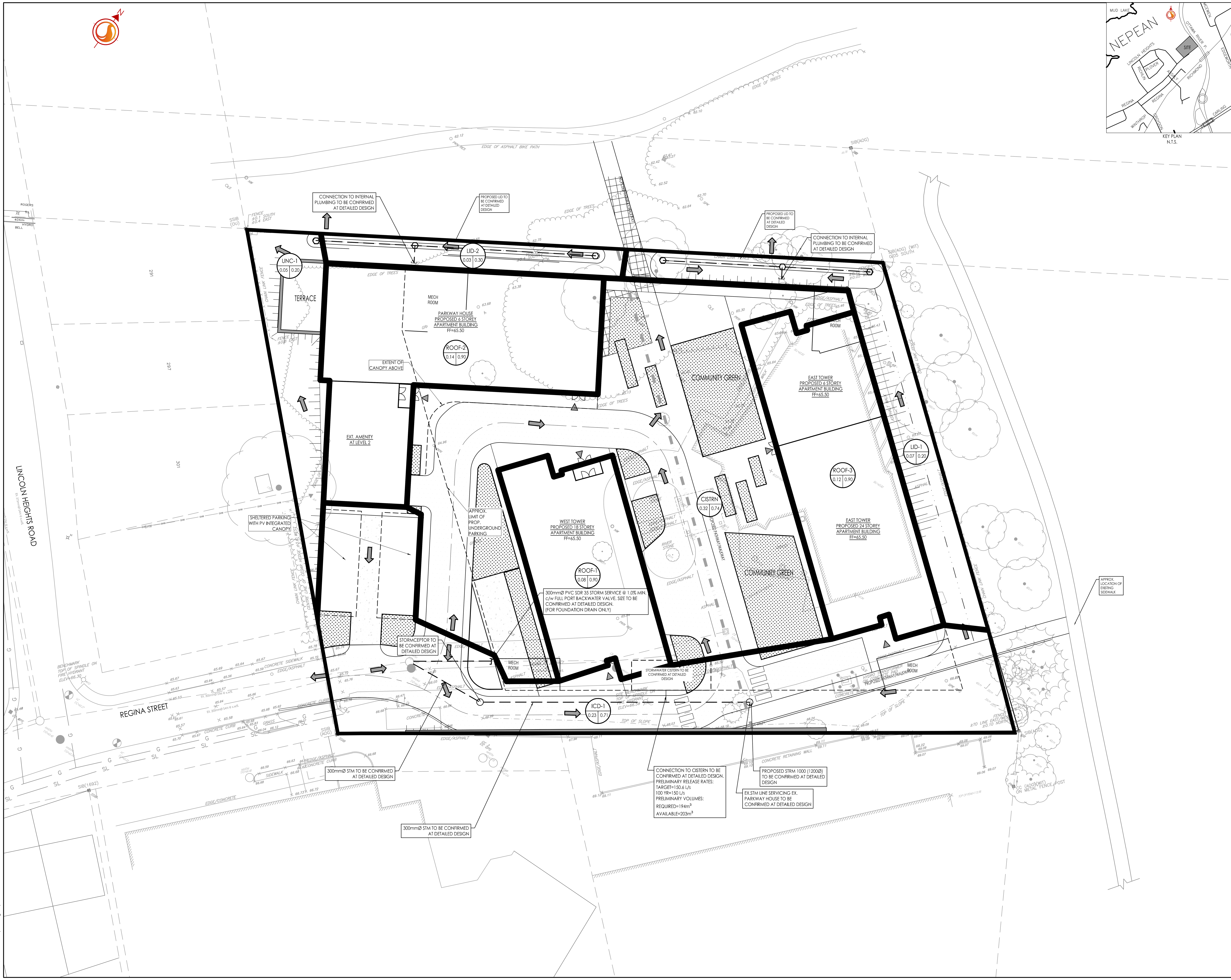
2022/05/03 1:25 PM By: Paluni, Jupiter

ORIGINAL SHEET - ARCH D

D.5 Functional Storm Drainage Plan



Stantec

Stantec Consulting Ltd.
400 - 1331 Clyde Avenue
Ottawa ON
Tel. 613.722.4420
www.stantec.com

Legend

- L101A — AREA ID
- 1.00 / 0.85 — RUNOFF COEFFICIENT
- STORM DRAINAGE AREA ha.
- STORM DRAINAGE BOUNDARY
- DIRECTION OF OVERLAND FLOW
- PROPOSED STORM MH AND SEWER
- EXISTING STORM MH AND SEWER
- EXISTING CATCHBASIN
- PHASE 2
- PHASE LINES

Revision	By	Appd.	YY.MM.DD
1 ISSUED FOR REVIEW	JP	DT	22.05.16

File Name: 160401689-FSG.dwg	Dwn.	Chkd.	Dsgn.	YY.MM.DD
	JP	DT	JP	22.04.18

Permit-Seal

Client/Project
WINDMILL DEVELOPMENT GROUP LTD.

2475 REGINA STREET

Ottawa, ON

Title
FUNCTIONAL
STORM DRAINAGE PLAN

Project No. 160401689
Scale 1:300 (0 3 9 15m)

Drawing No. SD-1
Sheet 4 of 5
Revision 1

ORIGINAL SHEET - ARCH D

Appendix E External Reports

E.1 Geotechnical Investigation (Paterson, 2021)

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Noise and Vibration Studies

Paterson Group Inc.
Consulting Engineers
154 Colonnade Road South
Ottawa (Nepean), Ontario
Canada K2E 7J5

Tel: (613) 226-7381
Fax: (613) 226-6344
www.patersongroup.ca

patersongroup

Geotechnical Investigation

Proposed Mixed-Use Development
2475 Regina Street
Ottawa, Ontario

Prepared For

Parkway House Development Fund LP

August 18, 2021

Report: PG5901-1

Table of Contents

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Appendices

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

Appendix 2 Figure 1 - Key Plan
Figures 2 & 3 - Seismic Shear Wave Velocity Profiles
Drawing PG5901-1 - Test Hole Location Plan

1.0 Introduction

Paterson Group (Paterson) was commissioned by Parkway House Development Fund LP to conduct a geotechnical investigation for the proposed mixed-use development to be located at 2475 Regina Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

- ❑ Determine the subsoil and groundwater conditions at this site by means of boreholes.
- ❑ Provide geotechnical recommendations pertaining to design of the proposed development including construction considerations which may affect the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the conceptual site plan, it is understood that the proposed development will consist of two high-rise buildings, one low-rise building with an approximate footprint of 1375 m^2, and several townhouses. A shared underground parking garage will occupy nearly the entire site footprint, with 1 level of underground parking at the eastern half of the site, and 2 levels of underground parking on the western half. Associated access lanes, walkways, and landscaped areas are also anticipated at finished grades. It is also expected that the proposed buildings will be municipally serviced.

Construction of the proposed development will involve demolition of the existing building presently located at the site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out from July 29 to August 3, 2021, and consisted of advancing a total of seven (7) boreholes to a maximum depth of 17.5 m below existing grade. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features. The borehole locations are shown on Drawing PG5901-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were drilled using a track-mounted drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of drilling to the required depths at the selected locations, and sampling and testing the overburden.

Sampling and In Situ Testing

Soil samples were recovered from the boreholes using two different techniques, namely, sampled directly from the auger flights (AU) or collected using a 50 mm diameter split-spoon (SS) sampler. Rock cores (RC) were obtained using a 47.6 mm inside diameter coring equipment. All samples were visually inspected and initially classified on site. The auger and split-spoon samples were placed in sealed plastic bags, and rock cores were placed in cardboard boxes. All samples were transported to our laboratory for further examination and classification. The depths at which the auger, split spoon and rock core samples were recovered from the boreholes are shown as AU, SS and RC, respectively, on the Soil Profile and Test Data sheets presented in Appendix 1.

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

Bedrock samples were recovered at borehole BH 1-21 using a core barrel and diamond drilling techniques. The depths at which rock core samples were recovered from the boreholes are shown as RC on the Soil Profile and Test Data sheets in Appendix 1.

A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The recovery value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the drilled section (core run). The RQD value is the ratio, in percentage, of the total length of intact rock pieces longer than 100 mm in one core run over the length of the core run. These values are indicative of the quality of the bedrock.

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

Groundwater

Monitoring wells were installed in boreholes BH 1-21, BH 6-21, and BH 7-21. The remaining boreholes were fitted with flexible polyethylene standpipes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Monitoring Well Installation

Typical monitoring well construction details are described below:

- ❑ Slotted 32 mm diameter PVC screen at the base of each borehole.
- ❑ 51 mm diameter PVC riser pipe from the top of the screen to the ground surface.
- ❑ No. 3 silica sand backfill within annular space around screen.
- ❑ Bentonite hole plug directly above PVC slotted screen.
- ❑ Clean backfill from top of bentonite plug to the ground surface.

Refer to the Soil Profile and Test Data sheets in Appendix 1 for specific well construction details.

Sample Storage

All samples will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless we are otherwise directed.

3.2 Field Survey

The borehole locations were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevation at each test hole location were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5901-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

Soil samples were recovered from the subject site and visually examined in our laboratory to review the results of the field logging.

3.4 Analytical Testing

One (1) soil sample was submitted for analytical testing to assess the corrosion potential for exposed ferrous metals and the potential of sulphate attacks against subsurface concrete structures. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The eastern portion of the subject site is occupied by a single-storey building and associated asphalt-paved access lanes and parking areas. The remainder of the subject site general consists of grassed areas with mature trees.

The site is bordered by the Sir John A. Macdonald Parkway to the east, vacant land and paved walking pathways to the north, Regina Street and residential dwellings to the west, and a multi-storey building followed by Richmond Road to the south. The ground surface across the majority of the subject site is relatively level and at-grade with Regina Street at approximate geodetic elevation 66.0 m, however, within the northwest corner of the site, the grade slopes downward gently from southeast to northwest from approximate geodetic elevation 66.0 to 63.5 m.

4.2 Subsurface Profile

Overburden

Generally, the soil profile at the borehole locations consists of a topsoil layer underlain by an approximate 0.8 to 1.8 m thick fill layer. The fill material was generally observed to consist of brown silty sand and/or clay with gravel, cobbles, boulders and varying amounts of topsoil and organics.

A hard to very stiff brown silty brown silty clay deposit was observed underlying the fill at BH 2-21.

A glacial till deposit was observed underlying either the fill or silty clay deposit at all boreholes at depths ranging from approximately 0.8 to 3.4 m below the existing ground surface. The glacial till deposit was generally observed to consist of a brown to grey silty sand to silty clay with gravel, cobbles, and boulders. Boulders were cored from approximate depths of 7.6 to 11.5 m at borehole BH 1-21 in order to advance the borehole.

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

Bedrock

The bedrock was cored at borehole BH 1-21 commencing at an approximate depth of 13.8 and extending to a depth of 17.5 m. The bedrock was observed to consist of grey quartz sandstone and, based on the recovered bedrock core, was generally weathered and of poor quality to an approximate depth of 14.4 m, becoming fair to good in quality with depth.

Based on available geological mapping and coring records, the bedrock in the subject area consists of Paleozoic shale of the Rockcliffe formation, with an overburden drift thickness of 5 to 15 m.

4.3 Groundwater

Groundwater levels were measured on August 11, 2021 within the installed monitoring wells and standpipes. The measured groundwater levels noted at that time are presented in Table 1 below

Table 1 – Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level		Dated Recorded
		Depth (m)	Elevation (m)	
BH 1-21*	66.09	7.74	58.35	August 11, 2021
BH 2-21	65.81	3.55	62.26	
BH 3-21	64.98	6.72	58.26	
BH 4-21	64.59	7.12	57.47	
BH 5-21	63.84	Dry	-	
BH 6-21*	63.62	5.20	58.42	
BH 7-21*	65.14	Dry	-	

Note: The ground surface elevation at each borehole location was surveyed using a handheld GPS using a geodetic datum.
*Denotes Groundwater Monitoring Well

The groundwater can also be estimated based on the colouring, consistency and moisture levels of the recovered samples. Based on these observations, the long-term groundwater table can be expected at approximately 4 to 5 m below ground surface. The recorded groundwater levels are also provided on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should be noted that 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

From a geotechnical perspective, the subject site is considered suitable for the proposed development. It is recommended that the proposed high-rise buildings each be founded on a raft foundation placed on an undisturbed, compact to dense glacial till bearing surface. It is further recommended that the low-rise buildings, as well as portions of the underground parking levels which extend beyond the footprints of the high-rise buildings, be supported on conventional spread footings bearing on undisturbed, compact to dense glacial till.

Where loose and/or soft glacial till is encountered at the underside of footing or raft, it should be sub-excavated to the undisturbed, compact to dense glacial till and re-instated with engineered fill.

Further, it is anticipated that cobbles and boulders will be encountered frequently throughout servicing trenches and building excavations. All contractors should be prepared for the removal of boulders and potentially oversized boulders throughout the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

Topsoil and deleterious fill, such as those containing organic materials, should be stripped from under any buildings, paved areas, pipe bedding and other settlement sensitive structures.

Existing foundation walls and other construction debris should be entirely removed from within the building perimeters. Under paved areas, existing construction remnants such as foundation walls should be excavated to a minimum of 1 m below final grade.

Protection of Subgrade (Raft Foundation)

Where a raft foundation is used, the raft subgrade would consist of a glacial till deposit, and it is recommended that a minimum 75 mm thick lean concrete mud slab be placed on the undisturbed glacial till subgrade shortly after the completion of the excavation. The main purpose of the mud slab is to reduce the risk of disturbance of the subgrade under the traffic of workers and equipment.

The final excavation to the raft bearing surface level and the placing of the mud slab should be done in smaller sections to avoid exposing large areas of the glacial till to potential disturbance due to drying.

Fill Placement

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

Non-specified existing fill along with site-excavated soil could be placed as general landscaping fill and beneath exterior parking where settlement of the ground surface is of minor concern. These materials should be spread in lifts with a maximum thickness of 300 mm and compacted by the tracks of the spreading equipment to minimize voids. If this material is to be used to build up the subgrade level for areas to be paved, it should be compacted in thin lifts to at least 95% of the material's SPMDD.

Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane, such as Miradrain G100N or Delta Drain 6000, connected to a perimeter drainage system.

5.3 Foundation Design

Conventional Shallow Foundations

For the low-rise building and portions of the underground parking levels located beyond the footprints of the proposed high-rise buildings, it is recommended that conventional spread footings placed on an undisturbed, compact to dense glacial till bearing surface can be designed using a bearing resistance value at serviceability limit states (SLS) of **200 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **300 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen, or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

Footings bearing on an undisturbed soil bearing surface and designed using the bearing resistance values provided above will be subjected to potential post-construction total and differential settlements of 25 to 20 mm, respectively.

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an undisturbed glacial till bearing surface, above the groundwater table, when a plane extending down and out from the bottom edge of the footing at a minimum of 1.5H:1V passes only through in situ soil of the same or higher capacity as the bearing medium soil.

Raft Foundation - High-Rise Building with One Underground Parking Level

The proposed high-rise buildings are recommended to each be supported on a raft foundation. For 1 underground parking level, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 62 to 61 m.

The maximum SLS contact pressure is **350 kPa** for a raft foundation bearing on the undisturbed, compact to dense glacial till. It should be noted that the weight of the raft slab and everything above has to be included when designing with this value. The loading conditions for the contact pressure are based on sustained loads, that are generally taken to be 100% Dead Load and 50% Live Load. The factored bearing resistance (contact pressure) at ULS can be taken as **500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be 14 MPa/m for a contact pressure of 350 kPa. The design of the raft foundation is required to consider the relative stiffness of the reinforced concrete slab and the supporting bearing medium. A common method of modeling the soil structure interaction is to consider the bearing medium to be elastic and to assign a subgrade modulus. However, glacial till is not elastic and limits have to be placed on the stress ranges of a particular modulus.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

Raft Foundation - High-Rise Building with Two Underground Parking Levels

Where 2 levels of underground parking underlie the proposed high-rise building, it is anticipated that the excavation will extend to a depth such that the underside of the raft slab would be placed between geodetic elevations of 59 to 58 m.

For this case, the maximum SLS contact pressure is **400 kPa** for a raft foundation bearing on the undisturbed, compact to dense glacial till. The factored bearing resistance (contact pressure) at ULS can be taken as **600 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value at ULS.

The modulus of subgrade reaction was calculated to be 16 MPa/m for a contact pressure of 400 kPa.

The proposed building can be designed using the above parameters with total and differential settlements of 25 and 20 mm, respectively.

5.4 Design for Earthquakes

Shear wave velocity testing was completed at the subject site to accurately determine the applicable seismic site classification for the proposed development in accordance with Table 4.1.8.4.A of the Ontario Building Code 2012. The shear wave velocity testing was completed by Paterson personnel. The results of the shear wave velocity test are provided in Figures 2 and 3 in Appendix 2.

Field Program

The seismic array testing location was placed within the central area of the site in an approximate north-south direction as presented in Drawing PG5901-1 - Test Hole Location Plan in Appendix 2. Paterson field personnel placed 24 horizontal 2.4 Hz. geophones mounted to the surface by means of two 75 mm ground spikes attached to the geophone land case. The geophones were spaced at 2 m intervals and connected by a geophone spread cable to a Geode 24 Channel seismograph. The seismograph was also connected to a computer laptop and a hammer trigger switch attached to a 12-pound dead blow hammer. The hammer trigger switch sends a start signal to the seismograph. The hammer is used to strike an I-Beam seated into the ground surface, which creates a polarized shear wave. The hammer shots are repeated between four (4) and eight (8) times at each shot location to improve signal to noise ratio.

The shot locations are also completed in forward and reverse direction (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations were 15, 3, and 2 m away from the first and last geophones, and at the centre of the seismic array.

Data Processing and Interpretation

Interpretation for the shear wave velocity results were completed by Paterson personnel. Shear wave velocity measurement was made using reflection/refraction methods. The interpretation is performed by recovering arrival times from direct and refracted waves.

The interpretation is repeated at each shot location to provide an average shear wave velocity, V_{s30}, of the upper 30 m profile, immediately below the building's foundation. The layer intercept times, velocities from different layers and critical distances are interpreted from the shear wave records to compute the bedrock depth at each location.

The bedrock velocity was interpreted using the main refractor wave velocity, which is considered a conservative estimate of the bedrock velocity due to the increasing quality of the bedrock with depth. It should be noted that as bedrock quality increases, the bedrock shear wave velocity also increases.

Seismic Site Class

For this scenario, the V_{S30} was calculated as follows:

$$V_{s30} = \frac{Depth_{of\ interest}(m)}{\left(\frac{Depth_{Layer1}(m)}{V_{s_{Layer1}}(m/s)} + \frac{Depth_{Layer2}(m)}{V_{s_{Layer2}}(m/s)}\right)}$$

$$V_{s30=} \frac{30\ m}{\left(\frac{9\ m}{420\ m/s} + \frac{21\ m}{2{,}464\ m/s}\right)}$$

$$V_{s30=}\ 1{,}002\ \ m/s$$

The average shear wave velocity, V_{s30}, is **1,002 m/s**, which is high enough for a seismic Site Class B. However, the Ontario Building Code (OBC) 2012 also requires that the foundations be within 3 m of the bedrock surface to achieve a Site Class B, which is not the case for the proposed development at this site. Therefore, a **Site Class C** is applicable for seismic design of the proposed buildings as per Table 4.1.8.4.A of the OBC 2012. The soils underlying the subject site are not susceptible to liquefaction.

5.5 Basement Slab

With the removal of all topsoil and deleterious fill from within the footprint of the proposed building, the native soil will be considered an acceptable subgrade on which to commence backfilling for floor slab construction. It is understood that the underground level(s) will be mostly parking and the recommended pavement structures noted in Section 5.7 will be applicable.

However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

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

In consideration of the groundwater conditions encountered during the field investigation, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a sump pit, should be provided in the subfloor fill under the lower basement floor (discussed further in Subsection 6.1).

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3 where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight.

Lateral Earth Pressures

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

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

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

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

Seismic Earth Pressures

The total seismic force (P_{AE}) includes both the earth force component (P_o) and the seismic component (ΔP_{AE}).

The seismic earth force (ΔP_{AE}) can be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

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

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

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

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

5.7 Rock Anchor Design

Overview of Anchor Features

The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or a 60 to 90 degree pullout of rock cone with the apex of the cone near the middle of the bonded length of the anchor. Interaction may develop between the failure cones of anchors that are relatively close to one another resulting in a total group capacity smaller than the sum of the load capacity of each individual anchor.

A third failure mode of shear failure along the grout/steel interface should be reviewed by the structural engineer to ensure all typical failure modes have been reviewed.

The anchor should be provided with a bonded length at the base of the anchor which will provide the anchor capacity, as well an unbonded length between the rock surface and the top of the bonded length.

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems, such as those available from Dywidag Systems or Williams Form Engineering Corp. Recognizing the importance of the anchors for the long term performance of the foundation of the proposed building, the rock anchors for this project are recommended to be provided with double corrosion protection.

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of sandstone ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. Based on existing bedrock information, a Rock Mass Rating (RMR) of 65 was assigned to the bedrock, and Hoek and Brown parameters (m and s) were taken as 0.575 and 0.00293, respectively.

Recommended Rock Anchor Lengths

Parameters used to calculate rock anchor lengths are provided in Table 2 on the following page:

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Sandstone Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone bedrock	50 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 75 mm and 125 mm diameter hole are provided in Table 3 on the next page. The factored tensile resistance values given in Table 2 are based on a single anchor with no group influence effects. A detailed analysis of the anchorage system, including potential group influence effects, could be provided once the details of the loading for the proposed building are determined.

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes.

The geotechnical capacity of each rock anchor should be proof tested at the time of construction. More information on testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the lowest level of the underground parking structure should consist of Category C2, 32 MPa concrete at 28 days with air entrainment of 5 to 8%. The recommended rigid pavement structure is further presented in Table 4 below. The flexible pavement structure presented in Tables 5 and 6 should be used for exterior, at grade parking areas and access lanes, respectively.

Table 4 - Recommended Rigid Pavement Structure - Lower Parking Level

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

To control cracking due to shrinking of the concrete floor slab, it is recommended that strategically located saw cuts be used to create control joints within the concrete floor slab of the lower underground parking level. The control joints are generally recommended to be located at the center of the column lines and spaced at approximately 24 to 36 times the slab thickness (for example, a 0.15 m thick slab should have control joints spaced between 3.6 and 5.4 m). The joints should be cut between 25 and 30% of the thickness of the concrete floor slab and completed as early as 4 hours after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

Table 5 - Recommended Pavement Structure – Car Only Parking Areas

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 - OPSS Granular B Type I or II placed over in-situ soil, or concrete fill.	

Table 6 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

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
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type I or II placed over in-situ soil, or concrete fill.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project. The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

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

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Water Suppression System and Foundation Drainage

For the proposed underground parking levels, it is anticipated that the building foundation walls will be placed in close proximity to the site boundaries. Therefore, it is recommended that the foundation wall be blind poured against a drainage system and waterproofing system fastened to the temporary shoring system.

Waterproofing of the foundation wall is recommended, and the membrane is to be installed starting at 4 m below grade down to the founding elevation. The waterproofing membrane should also be extended horizontally below the proposed footings a minimum of 600 mm away from the face of the excavation. The membrane will serve as a water infiltration suppression system.

It is also recommended that the composite drainage system, such as Delta Drain 6000 or equivalent, be installed between the waterproofing membrane and the foundation wall, and extend from the exterior finished grade to the founding elevation (underside of footing or raft slab). The purpose of the composite drainage system is to direct any water infiltration resulting from a breach of the waterproofing membrane to the building sump pit.

It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the foundation wall at the footing or raft slab interface to allow the infiltration of water to flow to an interior perimeter underslab drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

Foundation Raft Slab Construction Joints

It is expected that the raft slab, where utilized, will be poured in sections. For the construction joint at each pour, a rubber water stop along with a chemical grout (Xypex or equivalent) should be applied to the entire vertical joint of the slab. Furthermore, a rubber water stop should be incorporated in the horizontal interface between the foundation wall and the raft slab.

Sub-slab Drainage

Sub-slab drainage will be required to control water infiltration below the lowest underground parking level slab. For design purposes, it is recommended that a 150 mm diameter perforated pipe be placed at 6 m centres. The final spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

Perimeter foundations of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover should be provided for adequate frost protection for heated structures.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

However, the foundations are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of the excavation should either be cut back at acceptable slopes or be retained by shoring systems from the beginning of the excavation until the structure is backfilled. However, for most of the site, insufficient room will be available to permit the building excavation to be constructed by open-cut methods (i.e., unsupported excavations).

Unsupported Excavations

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should be kept away from the excavation sides.

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

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Temporary Shoring

Due to the anticipated proximity of the proposed development to the property boundaries, temporary shoring may be required to support the overburden soils. The shoring requirements will depend on the depth of the excavation and the proximity of the adjacent structures. However, it should be noted that the observed bouldery conditions can lead to the creation of voids and other unstable conditions during installation of the temporary shoring as boulders shift within the fine soil matrix. Furthermore, it may be difficult to develop the required anchor strength in soil due to variations in soil conditions.

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner’s representative prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressures acting on the temporary shoring system may be calculated using the parameters outlined in Table 7 on the next page.

Table 7 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System	
Parameter	**Value**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Unit Weight (γ), kN/m^3	21
Submerged Unit Weight(γ'), kN/m^3	13

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

The dry unit weight should be used above the groundwater level while the effective unit weight should be used below the groundwater level.

The hydrostatic groundwater pressure should be added to the earth pressure distribution wherever the effective unit weights are used for earth pressure calculations. If the groundwater level is lowered, the dry unit weight for the soil should be used full weight, with no hydrostatic groundwater pressure component. For design purposes, the minimum factor of safety of 1.5 should be calculated.

6.4 Pipe Bedding and Backfill

Bedding and backfill materials should be in accordance with the most recent Material Specifications and Standard Detail Drawings from the Department of Public Works and Services, Infrastructure Services Branch of the City of Ottawa. At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe.

Cover material, from the spring line to at least 300 mm above the obvert of the pipe, should consist of OPSS Granular A or Granular B Type II with a maximum size of 25 mm. The bedding and cover materials should be placed in maximum 225 mm thick lifts compacted to 99% of the material's SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

It is anticipated that groundwater infiltration into the excavations should be controllable using open sumps. The contractor should be prepared to direct water away from all bearing surfaces and subgrades, regardless of the source, to prevent disturbance to the founding medium.

Permit to Take Water

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

For typical ground or surface water volumes being pumped during the construction phase, typically between 50,000 to 400,000 L/day, it is required to register on the Environmental Activity and Sector Registry (EASR). A minimum of two to four weeks should be allotted for completion of the EASR registration and the Water Taking and Discharge Plan to be prepared by a Qualified Person as stipulated under O.Reg. 63/16. If a project qualifies for a PTTW based upon anticipated conditions, an EASR will not be allowed as a temporary dewatering measure while awaiting the MECP review of the PTTW application.

Impacts on Neighbouring Properties

Since the proposed development will be founded below the long-term groundwater level, a waterproofing membrane system has been recommended to lessen the effects of water infiltration. Any long-term dewatering of the site will therefore be minimal and will have no adverse effects to the surrounding buildings or structures. The short-term dewatering during the excavation program, which is expected to be minimal, will be managed by the excavation contractor.

6.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site consist of frost susceptible materials. In the presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions. Additional information could be provided, if required.

Precaution must be taken where excavations are carried out in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil.
Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

It is recommended that the following be completed once the master plan and site development are determined.

- ❑ Review of the geotechnical aspects of the excavation contractor's shoring design, if required, prior to construction.
- ❑ Review of waterproofing details for the elevator shaft and building sump pits.
- ❑ Review and inspection of the foundation waterproofing system and all foundation drainage systems.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ 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.
- ❑ Complete a full inspection program of the installation of the perimeter and underground floor drainage system during construction.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ 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.

8.0 Statement of Limitations

The recommendations provided herein are in accordance with our present understanding of the project. Paterson requests permission to review the recommendations when the drawings and specifications are completed.

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

The recommendations provided herein should only be used by the design professionals associated with this project. They are not intended for contractors bidding on or undertaking the work. The latter should evaluate the factual information provided in this report and determine the suitability and completeness for their intended construction schedule and methods. Additional testing may be required for their purposes.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Parkway House Development Fund LP or their agents is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Aug. 18, 2021
S. S. DENNIS
100519516
PROVINCE OF ONTARIO

Kevin A. Pickard, EIT. Scott S. Dennis, P.Eng.

Report Distribution:

- ❑ Parkway House Development Fund LP (email copy)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL TESTING RESULTS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 1-21

BORINGS BY Track-Mount Power Auger

DATE July 29, 2021

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	66.09
TOPSOIL (to 0.15)						
FILL: Brown silty clay, trace sand topsoil (to 0.91)	SS	1	42	9		
GLACIAL TILL: Compact, brown silty sand, some clay, gravel, cobbles and boulders - clay content increasing with depth (to 4.00)	SS	2	75	23	1	65.09
	SS	3	42	14	2	64.09
	SS	4	83	11		
	SS	5	33	13	3	63.09
GLACIAL TILL: Very stiff, brown silty clay, some sand, gravel, cobbles and boulders - grey by 4.6m depth (to 7.00)	SS	6	83	9	4	62.09
	SS	7	33	8	5	61.09
	SS	8	50	4	6	60.09
	SS	9	75	6		
GLACIAL TILL: Compact to dense, grey silty sand, some clay, gravel, cobbles and boulders	SS	10	75	19	7	59.09
	RC	1	50		8	58.09
	RC	2	21		9	57.09
	RC	3	50		10	56.09
					11	55.09

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 1-21

BORINGS BY Track-Mount Power Auger

DATE July 29, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							
GLACIAL TILL: Compact to dense, grey silty sand, some clay, gravel, cobbles and boulders - cored through boulders from 7.62 to 11.5m depth.						11	55.09
						12	54.09
						13	53.09
13.79							
BEDROCK: Poor to good quality, grey quartz sandstone		RC	4	100	26	14	52.09
		RC	5	100	59	15	51.09
						16	50.09
		RC	6	100	81	17	49.09
17.47							
End of Borehole (GWL @ 7.74m - August 11, 2021)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 2-21

BORINGS BY Track-Mount Power Auger

DATE July 30, 2021

SOIL DESCRIPTION	Sample TYPE	Sample NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	65.81
TOPSOIL (to 0.15)						
FILL: Brown silty sand, trace gravel and organics (0.15 to 1.22)	SS	1	42	10		
	SS	2	12	10	1	64.81
Hard to very stiff, brown **SILTY CLAY,** some sand (1.22 to 3.35)	SS	3	83	10	2	63.81
	SS	4	83	8	3	62.81
GLACIAL TILL: Brown silty clay with sand, gravel, cobbles and boulders - grey by 4.6m depth (3.35 to 6.70)	SS	5	75	9		
	SS	6	75	28	4	61.81
	SS	7	58	11	5	60.81
	SS	8	58	10	6	59.81
	SS	9	62	18		

End of Borehole

Practical refusal to augering at 6.70m depth

(GWL @ 3.55m - August 11, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

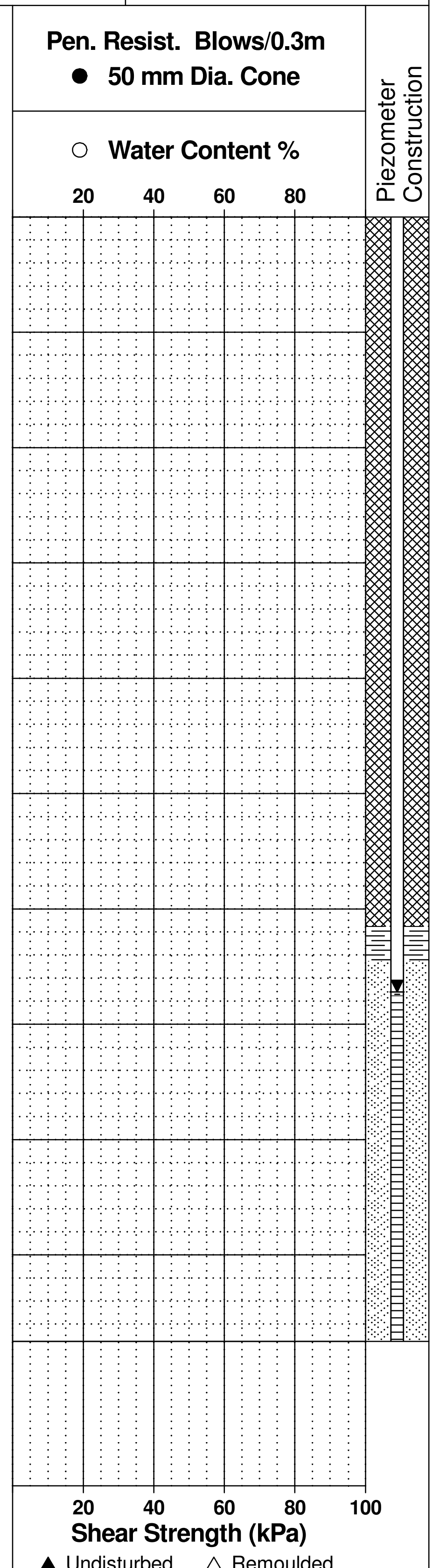
BORINGS BY Track-Mount Power Auger

DATE July 30, 2021

FILE NO. PG5901

HOLE NO. BH 3-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.98
TOPSOIL 0.20							
FILL: Brown silty sand with cobbles, occasional boulders 0.91		AU	1				
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, some clay		SS	2	75	18	1	63.98
- clay content decreasing with depth		SS	3	62	28	2	62.98
- grey by 4.6m depth		SS	4	75	16	3	61.98
		SS	5	79	32	4	60.98
		SS	6	62	19	5	59.98
		SS	7	62	10	6	58.98
		SS	8	62	21	7	57.98
		SS	9	75	30	8	56.98
		SS	10	73	82	9	55.98
9.75		SS	11	83	18		
End of Borehole							
(GWL @ 6.72m - August 11, 2021)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

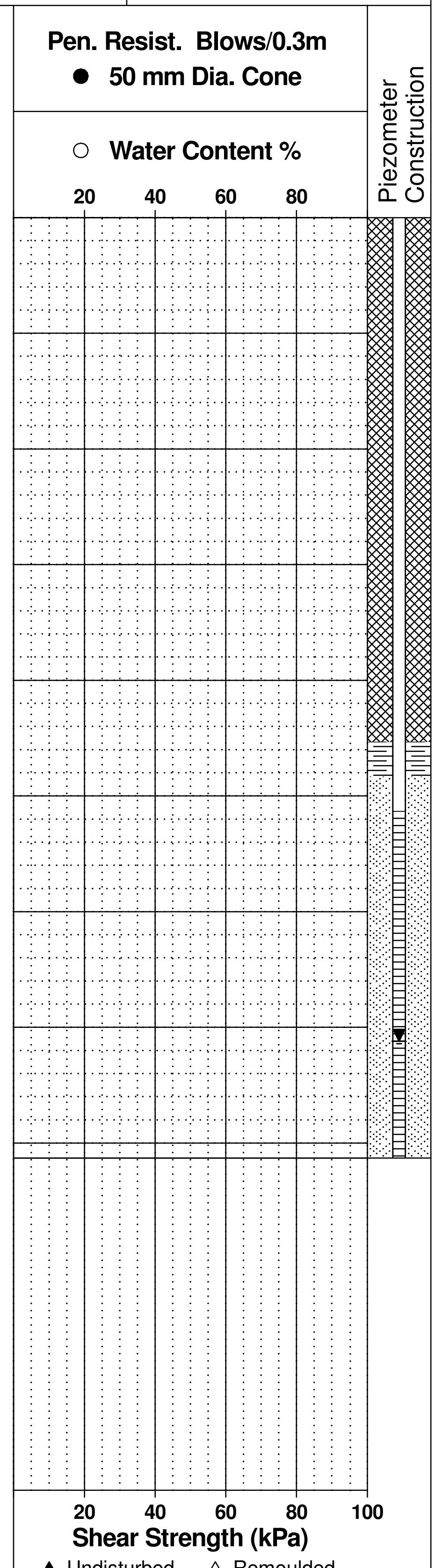
BORINGS BY Track-Mount Power Auger

DATE July 30, 2021

FILE NO. PG5901

HOLE NO. BH 4-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	64.59
TOPSOIL 0.20							
FILL: Brown silty sand with gravel 0.76		AU	1				
		SS	2	75	7	1	63.59
		SS	3	33	32	2	62.59
		SS	4	83	16		
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace to some clay		SS	5	21	29	3	61.59
- clay content decreasing with depth		SS	6	75	23	4	60.59
		SS	7	17	29	5	59.59
- grey by 5.2m depth		SS	8	67	21		
		SS	9	25	46	6	58.59
						7	57.59
8.13		SS	10	70	80	8	56.59
End of Borehole							
(GWL @ 7.12m - August 11, 2021)							



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5901

REMARKS

HOLE NO. BH 5-21

BORINGS BY Track-Mount Power Auger **DATE** July 30, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	63.84		
TOPSOIL 0.20									
FILL: Brown silty sand with gravel and cobbles, trace clay and topsoil		AU	1						
		SS	2	21	15	1	62.84		
1.45									
GLACIAL TILL: Compact to very dense, brown silty sand with gravel, cobbles and boulders, some to trace clay		SS	3	25	19	2	61.84		
		SS	4	50	50+	3	60.84		
						4	59.84		
		SS	5	42	82	5	58.84		
		SS	6	56	50+	6	57.84		
6.55									
End of Borehole									
(BH dry - August 11, 2021)									

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE August 3, 2021

FILE NO. PG5901

HOLE NO. BH 6-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	63.62
TOPSOIL	0.20						
FILL: Brown silty sand with clay, gravel, cobbles, trace topsoil	1.22	AU	1				
		SS	2	50	9	1	62.62
GLACIAL TILL: Compact to dense, brown silty sand with gravel, cobbles and boulders, trace clay - grey by 4.3m depth	6.10	SS	3	50	18	2	61.62
		SS	4	83	11	3	60.62
		SS	5	83	14		
		SS	6	50	23	4	59.62
		SS	7	83	23	5	58.62
		SS	8	83	39	6	57.62

End of Borehole

(GWL @ 5.20m - August 11, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Proposed Multi-Use Development - Parkway House
2475 Regina Street, Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Track-Mount Power Auger

DATE August 3, 2021

FILE NO. PG5901

HOLE NO. BH 7-21

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.14
TOPSOIL	0.20	AU	1				
FILL: Brown silty sand with gravel	1.37	SS	2	75	36	1	64.14
FILL: Brown silty sand with clay, gravel, some topsoil	1.83	SS	3	75	18	2	63.14
GLACIAL TILL: Compact, brown silty sand with gravel, cobbles, boulders, trace clay		SS	4	83	15	3	62.14
		SS	5	83	16	4	61.14
		SS	6	25	31	5	60.14
- grey by 4.9m depth		SS	7	58	11	6	59.14
	6.10	SS	8	25	11		

End of Borehole

(BH dry - August 11, 2021)

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone

○ Water Content %

20 40 60 80

Piezometer Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

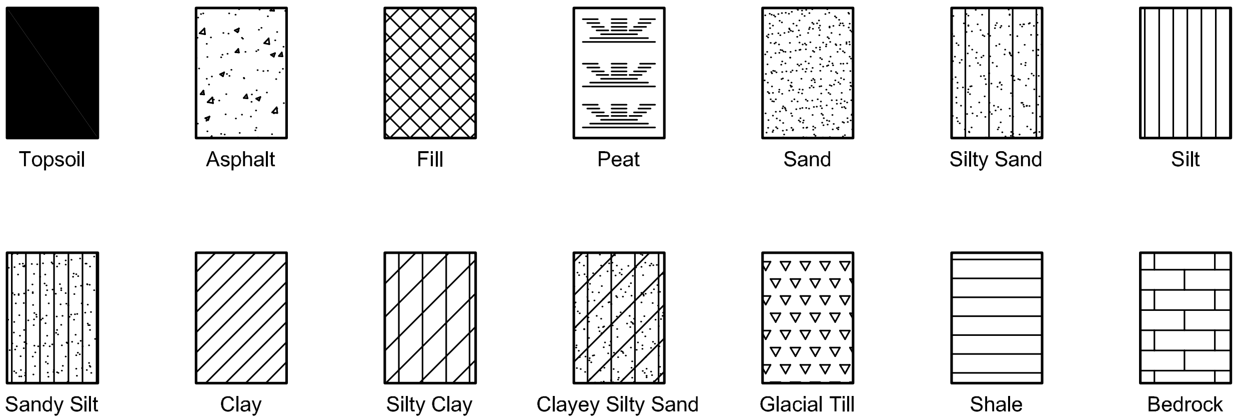
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

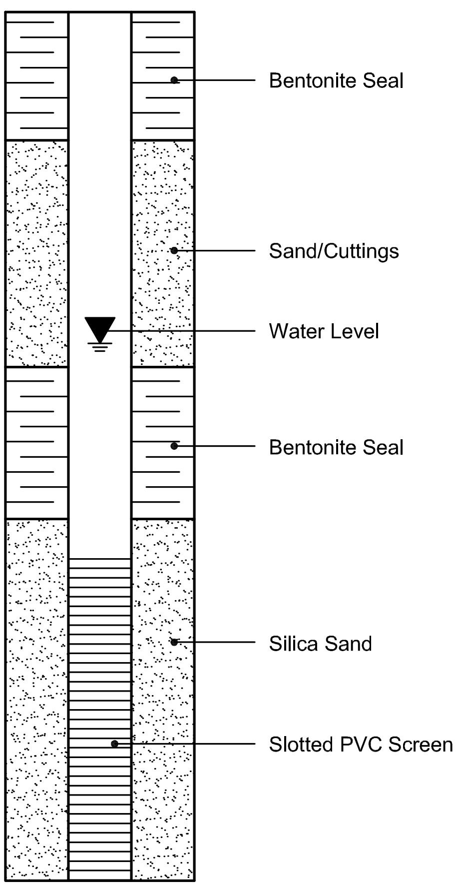
SYMBOLS AND TERMS (continued)

STRATA PLOT

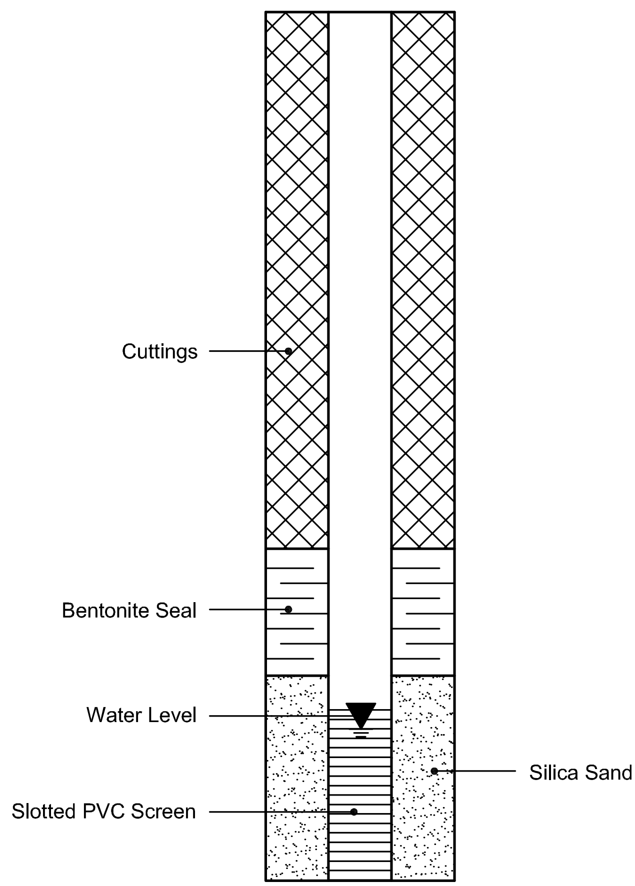


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: **Paterson Group Consulting Engineers**

Client PO: 32555

Report Date: 06-Aug-2021

Order Date: 3-Aug-2021

Project Description: PG5901

	Client ID:	BH3-21 SS9	-	-	-
	Sample Date:	30-Jul-21 09:00	-	-	-
	Sample ID:	2132173-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	93.0	-	-	-
General Inorganics					
pH	0.05 pH Units	8.06	-	-	-
Resistivity	0.10 Ohm.m	36.2	-	-	-
Anions					
Chloride	5 ug/g dry	<5	-	-	-
Sulphate	5 ug/g dry	168	-	-	-

APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 & 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG5901-1 - TEST HOLE LOCATION PLAN

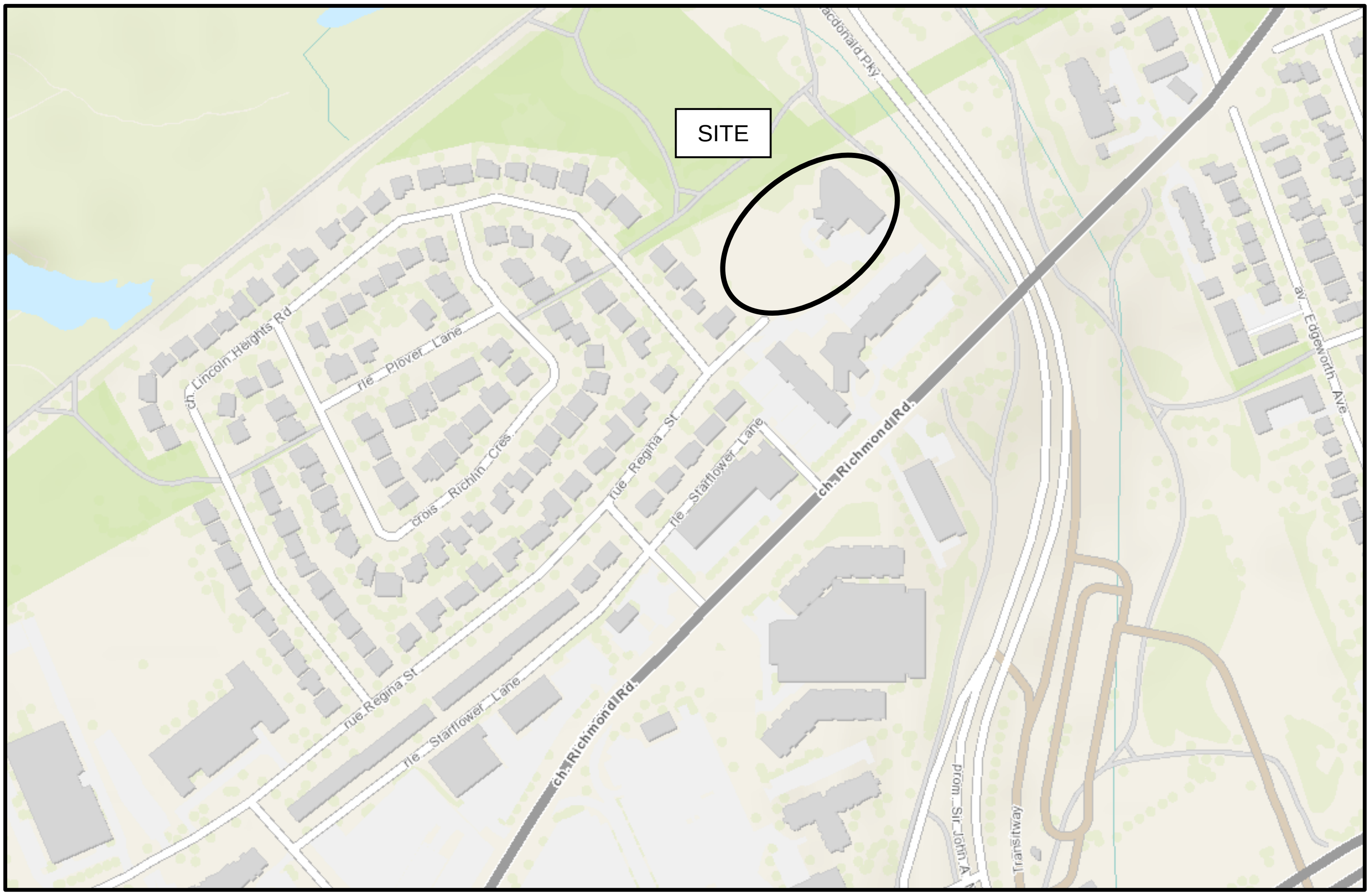


FIGURE 1

KEY PLAN

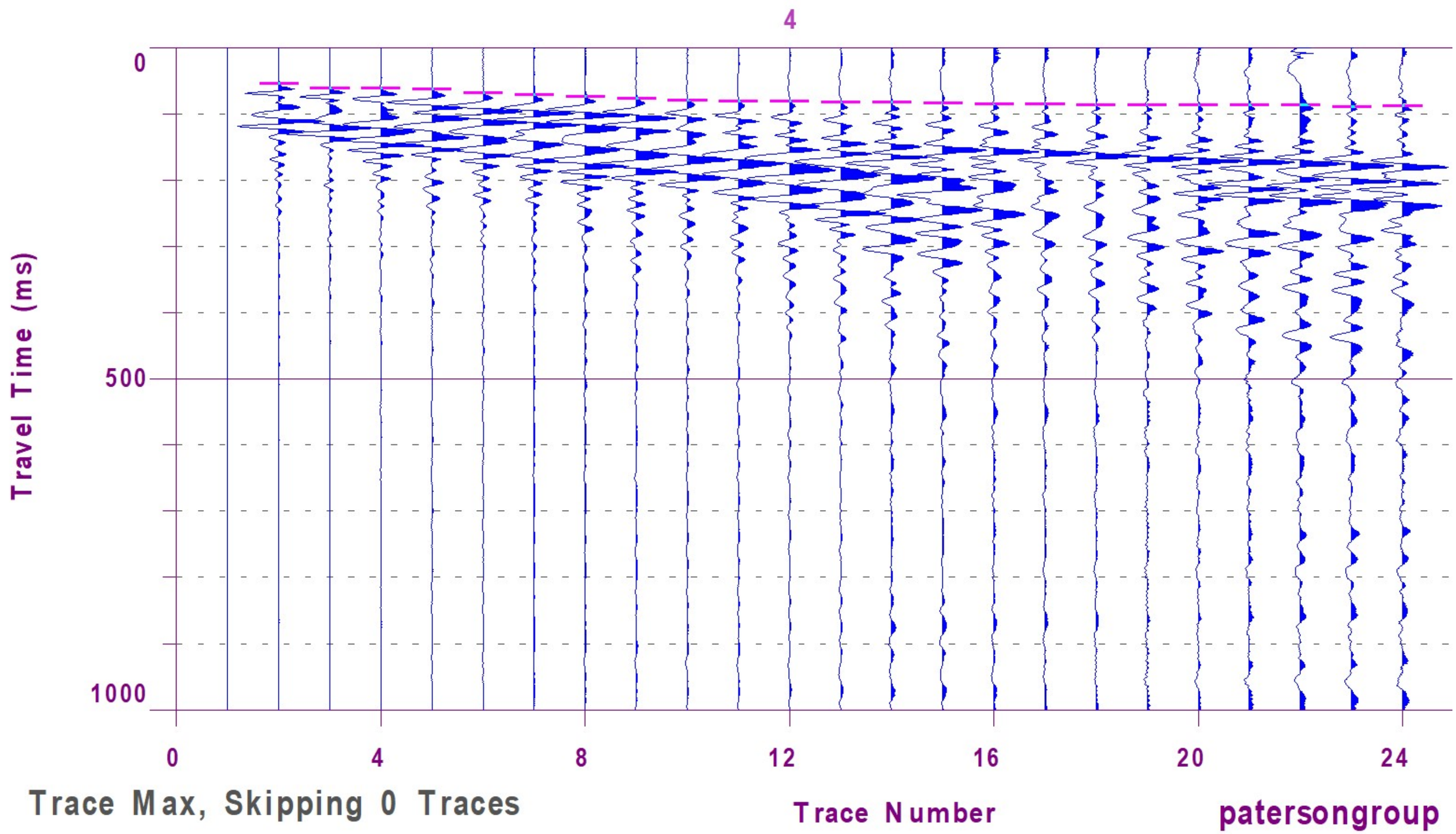


Figure 2 – Shear Wave Velocity Profile at Shot Location -15 m

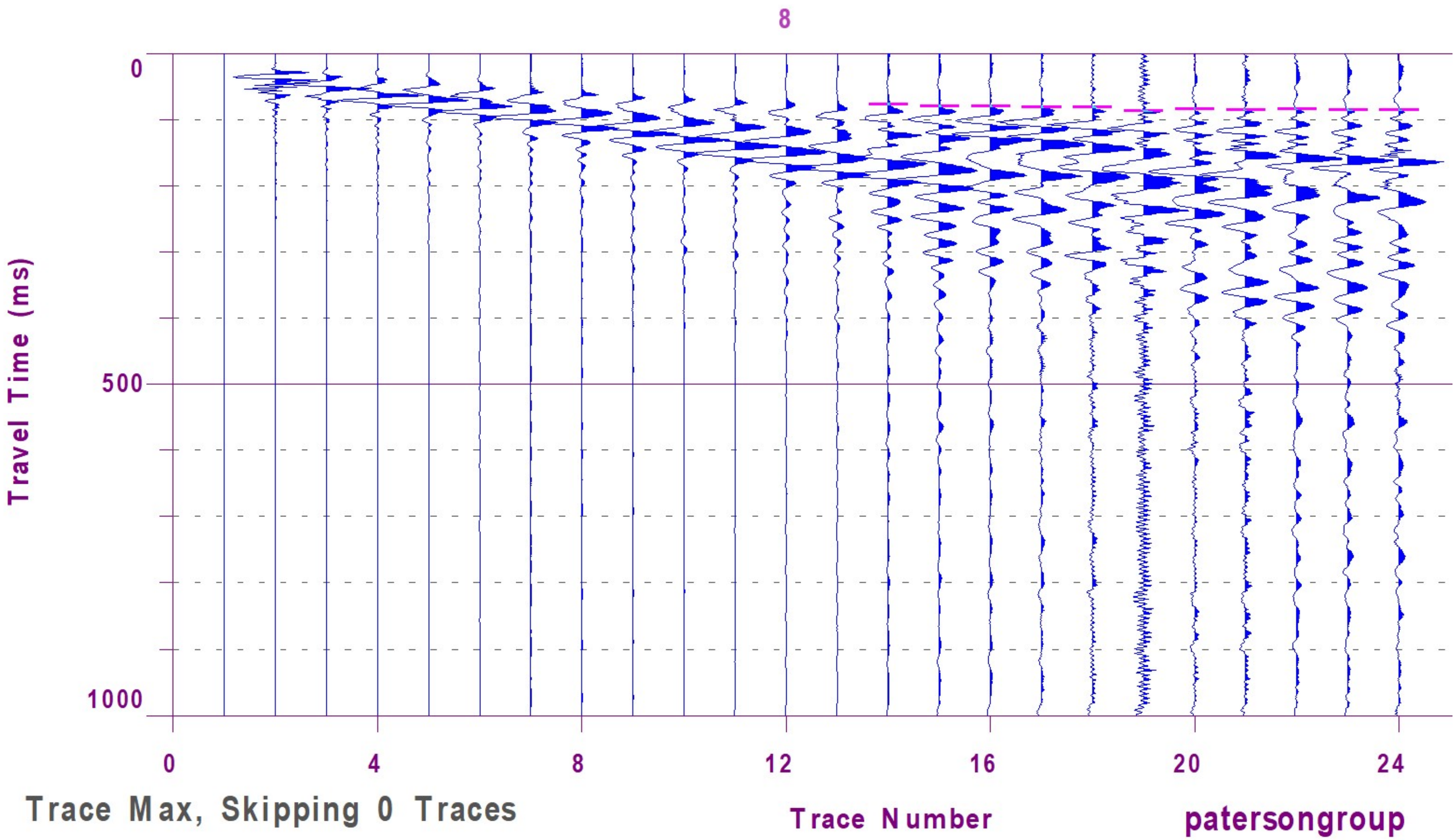
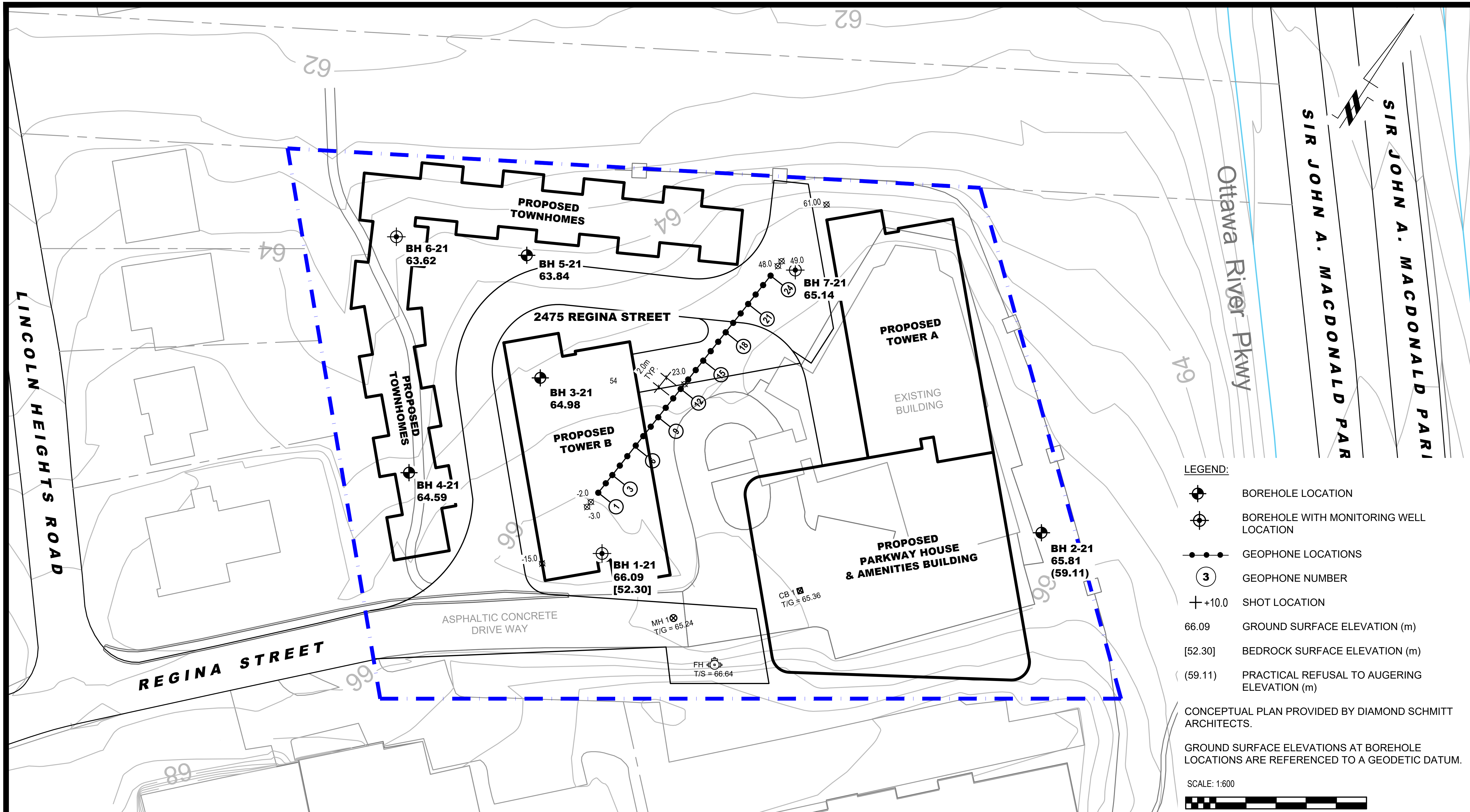


Figure 3 – Shear Wave Velocity Profile at Shot Location -2 m



patersongroup
consulting engineers

154 Colonnade Road South
Ottawa, Ontario K2E 7J5
Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL

PARKWAY HOUSE DEVELOPMENT FUND LP
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE DEVELOPMENT
2475 REGINA STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:600	Date: 08/2021
Drawn by: JM	Report No.: PG5901-1
Checked by: KP	Dwg. No.: PG5901-1
Approved by: SD	Revision No.:

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11x17

Appendix F Drawings