Site Servicing and Stormwater Management Brief – 851 Richmond Road, Ottawa, ON

File: 160401329/83

Prepared for:
Homestead Land Holdings Ltd.

Prepared by:
Stantec Consulting Ltd.

October 6, 2017

Revision Record							
Revision	Description	Prepared By		Checked By		Approved By	
0	1st submission	A. Paerez	10/03/2017	S. Gillis	10/04/2017	A. Paerez	10/06/2017

Sign-off Sheet

This document entitled Site Servicing and Stormwater Management Brief – 851 Richmond Road, Ottawa, ON was prepared by Stantec Consulting Ltd. ("Stantec") for the account of Homestead Land Holdings Ltd. (the "Client"). Any reliance on this document by any third party is strictly prohibited. The material in it reflects Stantec's professional judgment in light of the scope, schedule and other limitations stated in the document and in the contract between Stantec and the Client. The opinions in the document are based on conditions and information existing at the time the document was published and do not take into account any subsequent changes. In preparing the document, Stantec did not verify information supplied to it by others. Any use which a third party makes of this document is the responsibility of such third party. Such third party agrees that Stantec shall not be responsible for costs or damages of any kind, if any, suffered by it or any other third party as a result of decisions made or actions taken based on this document.

Approved by ______________________________

(signature)

Ana M. Paerez, P. Eng.

LICENSED PROFESSIONAL ENGINEER
A.M. PAEREZ FERNANDEZ
100105130
2017-10-11
PROVINCE OF ONTARIO

Table of Contents

LIST OF TABLES

LIST OF FIGURES

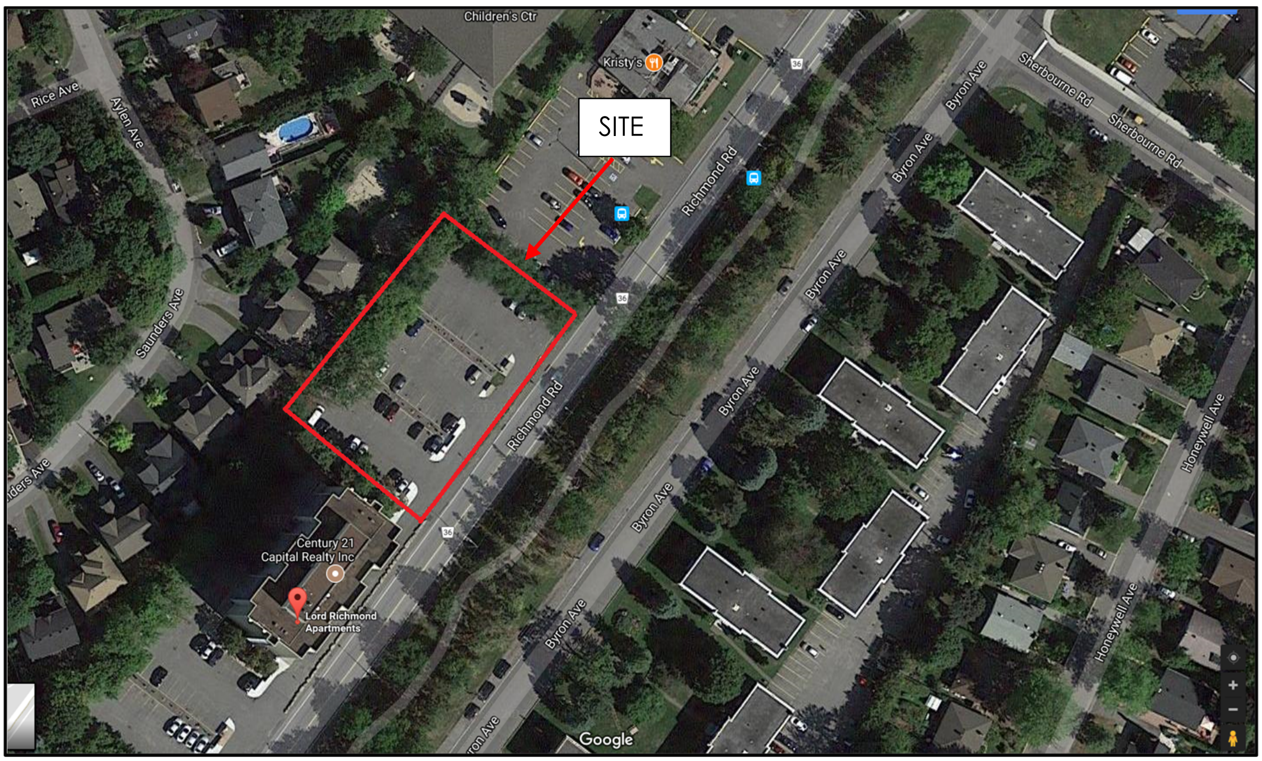
LIST OF APPENDICES

1.0 INTRODUCTION AND OBJECTIVE

Stantec Consulting Ltd. has been retained by Homestead Lands Holding Ltd. to prepare the following site servicing and stormwater management (SWM) brief to satisfy the City of Ottawa Site Plan Control Application process. The site is located on 851 Richmond Road, north-west of the intersection of Byron Avenue and Sherbourne Road and south-west of the intersection of Richmond Road and Cleary Avenue in the city of Ottawa (see **Figure 1** below).

The 0.31 ha site is currently occupied by parking areas and a small vegetated strip. The proposed development consists of an eleven-storey residential building with 132 units, underground parking and associated access and servicing infrastructure.

Figure 1: Site Location



1.1 OBJECTIVE

This site servicing and SWM brief has been prepared to present a servicing scheme that is free of conflicts and which utilizes the existing infrastructure as obtained from available as-built drawings and in consultation with City of Ottawa staff. Infrastructure requirements for water supply, sanitary 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:

- Prepare a preliminary grading plan in accordance with the proposed site plan and existing grades.
- Storm Sewer Servicing
  - Define major and minor conveyance systems in conjunction with the preliminary grade control plan
  - Determine the stormwater management storage requirements to meet the allowable release rate for the site
  - Coordinate with mechanical engineer and architect to provide an underground cistern and sump pump system to meet SWM requirements
  - Coordinate with mechanical engineer to convey external storm runoff from the adjacent development through the proposed building plumbing system
  - Coordinate with the mechanical engineer to install an oil/grit separator (OGS) within the underground parking to provide 'Enhanced' quality treatment (80% TSS removal) of runoff from the proposed development area
  - Define and size the proposed storm sewer laterals that will be connected to the existing 375 mm diameter CSP located in the back of the site
- Wastewater Servicing
  - Define and size the sanitary service laterals which will be connected to the existing 225 mm diameter on Richmond Road
- Water Servicing
  - Estimate water demands to characterize the proposed feed for the proposed development which will be serviced from the existing 203 mm diameter watermain on Richmond Road.
  - Watermain servicing for the development is to be able to provide average day and maximum day (including peak hour) demands (i.e. non-emergency conditions) at pressures within the acceptable range of 50 to 80 psi (345 to 552 kPa)
  - Under fire flow (emergency) conditions, the water distribution system is to maintain a minimum pressure greater than 20 psi (140 kPa)

The accompanying drawings included in the back of this report illustrate the preliminary internal servicing scheme for the site.

2.0 REFERENCES

The following background studies have been referenced during the preliminary servicing design of the proposed site:

- *Assessment of Adequacy of Public Services for OCEF Corp 809 Richmond Road*, David Schaeffer Engineering Ltd., April 2016
- *City of Ottawa Design Guidelines – Water Distribution*, City of Ottawa, July 2010
- *City of Ottawa Sewer Design Guidelines*, City of Ottawa, October 2012
- *Technical Bulletin ISDTB-2014-01,* City of Ottawa, February 2014
- *Technical Bulletin PIEDTB -2016-01,* City of Ottawa, September 6, 2016
- *Geotechnical Investigation Proposed Multi-Storey Building 851 Richmond Road – Ottawa,* Paterson Group, October 3, 2017

3.0 WATER DISTRIBUTION

The proposed building is located in Pressure Zone 1W of the City of Ottawa's Water Distribution System. The proposed development will be serviced through the existing 203 mm diameter watermain on Richmond Road as shown on the Conceptual Site Plan (see **Drawing SSP-1**).

The proposed eleven-storey building is to be a high rise residential building with a mix of one-bedroom and two-bedroom apartments for a total of 132 units, and underground parking. The building is to have a total floor space of approximately 12,479 m^2 (1.25 ha) above grade.

Water demands were calculated using the City of Ottawa Water Distribution Guidelines (July, 2010) to determine the typical operating pressures to be expected at the building (see detailed calculations in **Appendix A**). A daily rate of 350 L/cap/day has been applied for the population of the proposed site. The average daily (AVDY) residential demand was estimated for an occupancy of 1.4 persons per unit for a one-bedroom apartment and 2.1 persons per unit for a two-bedroom apartment. Maximum day (MXDY) residential demand was determined by multiplying the AVDY demand by a factor of 2.5 and peak hourly (PKHR) residential demand was determined by multiplying the MXDY demand by a factor of 2.2. The estimated demands are summarized in **Table 1**.

Table 1: Estimated Water Demands

	Population	AVDY (L/s)	MXDY (L/s)	PKHR (L/s)
Residential	228	0.92	2.30	5.07

1. Residential population based on 61 two-bedroom apartments and 71 one-bedroom apartments.

The fire flow requirement was calculated in accordance with Fire Underwriters Survey (FUS) and determined to be approximately 5,000 L/min (83 L/s). This estimate is based on a non-combustible construction building with a two-hour fire separation considered between each floor per requirements for buildings over six-storeys per Ontario Building Code. Additionally, it is anticipated that all buildings will be sprinklered, with final sprinkler design to conform to NFPA 13 (see detailed calculations in **Appendix A**).

The boundary conditions listed below were provided by the City of Ottawa on June 28, 2017 for the estimated water demands shown in **Table 1**.

Minimum HGL = 108.6 m

Maximum HGL = 116.2 m

MXDY (2.3L/s) + Fire Flow (83 L/s) = 99.0 m

The desired normal operating objective pressure range as per the City of Ottawa 2010 Water Distribution Design Guidelines is 345 kPa (50 psi) to 552kPa (80 psi) and no <u>less than 276kPa (40 psi)</u> at ground elevation. Furthermore, the maximum pressure at any point in the water distribution should not exceed 100 psi as per the Ontario Building/Plumbing Code; pressure reducing measures are required to service areas where pressures *greater than 552kPa (80 psi)* are anticipated.

The ground elevation along Richmond Road where the proposed building is to be connected is approximately 65.92 m. With respect to the peak hour flow conditions, the resulting boundary condition HGL of 108.6 m corresponds to a peak hour pressure of 418kPa (61 psi). Since the proposed building is an 11-storey building, an additional 34 kPa (5 psi) for every additional storey over two storeys is required to account for the change in elevation head and additional headloss. Given that the lowest pressure is expected to be 418 kPa (61 psi) at ground level, the resultant equivalent pressure at the 11th floor will be approximately 110 kPa (16 psi) and below the City's objective pressures. As a result, a pump will be required to maintain an acceptable level of service on the higher floors.

With respect to the maximum pressure during basic day demands, the resulting boundary condition HGL of 116.2 m corresponds to a pressure of 493 kPa (71 psi). The value is within the normal operating pressure range as per MOECC and City of Ottawa design guidelines.

In regards to available fire flow, boundary conditions provided by the City confirm that a flow rate of 5,000 L/min (83 L/s) would have a residual pressure of 324kPa (47 psi). The fire flow rate should be achievable within the watermain at this proposed location while maintaining a residual pressure of 138kPa (20 psi).

In conclusion, based on the boundary conditions provided, the 203 mm diameter watermain on Richmond Road provides adequate fire flow capacity as per the Fire Underwriters Survey. In order to meet the City water supply objective that limits a single feed to 50 m^3/d during basic day demands, dual connection to the existing 203 mm diameter watermain on Richmond Road is required to service the proposed building. The service connection will be capable of providing anticipated demands to the lower storeys but will require a booster pump to maintain pressures of 276 kPa (40 psi) for floors 7 to 11.

4.0 SANITARY SEWER

As illustrated on **Drawing SSP-1**, sanitary servicing for the proposed development will be provided through a proposed 200 mm diameter service lateral connecting to the existing 225 mm diameter sanitary sewer running east on Richmond Road which ultimately discharges into an existing 1500 mm diameter sanitary trunk sewer at the intersection of Richmond Road and Sherbourne Road.

The proposed 0.31 ha re-development area will consist of 71 one-bedroom apartments, 61 two-bedroom apartments, underground parking, and associated access infrastructure. The anticipated wastewater peak flow generated from the proposed development is summarized in **Table 2** below while a sanitary sewer design sheet is included in **Appendix C**.

Table 2: Estimated Wastewater Peak Flow

Residential Units				Infiltration Flow (L/s)	Total Peak Flow (L/s)
# of Units	Population	Peak Factor	Peak Flow (L/s)		
132	228	4.0	3.7	0.1	3.8

1. Average residential flow based on 350 L/p/day
2. Peak factor for residential units calculated using Harmon's formula
3. Apartment population estimated based on 1.4 persons/unit for one-bedroom apartments and 2.1 persons/unit for two-bedroom apartments
4. Infiltration flow based on 0.28 L/s/ha.

An analysis of the existing 225 mm diameter sanitary sewer on Richmond Road was completed in DSEL's Assessment of Adequacy of Public Services – 809 Richmond Road in April 2016 to estimate the available capacity within the sewer. The analysis concluded that the existing sanitary sewer had additional capacity for 42.6 L/s, and that the proposed development on 809 Richmond Road would generate 7.44 L/s of peak wet weather flow. As a result, the residual capacity of 35.2 L/s in the existing sewer will be sufficient to accommodate the proposed development.

Detailed sanitary sewage calculations are included in **Appendix C**. A backflow preventer will be required for the proposed building in accordance with the Ottawa sewer design guide, and will be coordinated with building mechanical engineers.

All underground parking drains should be connected to the internal building plumbing. A sump pump will be required to drain the underground parking levels to the existing sanitary sewer on Richmond Road.

4.1 SANITARY SEWER DESIGN CRITERIA

As outlined in the City of Ottawa Sewer Design Guidelines and the MOECC's Design Guidelines for Sewage Works, the following criteria were used to calculate estimated wastewater flow rates and to size the sanitary sewer lateral:

- Minimum Velocity – 0.6 m/s (0.8 m/s for upstream sections)
- Maximum Velocity – 3.0 m/s
- Manning roughness coefficient for all smooth wall pipes – 0.013
- 1.4 persons/one-bedroom apartment
- 2.1 persons/two-bedroom apartment
- Harmon's Formula for Peak Factor – Max = 4.0
- Extraneous Flow Allowance – 0.28 L/s/ha (conservative value)
- Manhole Spacing – 120 m
- Minimum Cover – 2.5 m

5.0 STORMWATER MANAGEMENT

5.1 OBJECTIVES

The objective of this stormwater management plan is to determine the measures necessary to control the quantity of stormwater released from the proposed development to the required levels and to provide sufficient detail for approval and construction.

5.2 EXISTING CONDITIONS

The site is currently paved consisting of parking areas for the existing 11-storey building immediately to the south. The existing parking areas sheet drain towards three existing catchbasins connected to a storm sewer system that conveys runoff from the adjacent development to the south and discharges into an existing 375 mm diameter CSP that discharges into an existing ditch in the existing Children's Centre to the north (see **Drawing EX-1**). The existing ditch is approximately 15 m long and discharges into an existing ditch inlet catchbasin connected to a 525 mm diameter storm sewer that ultimately directs runoff to the Ottawa River. Based on visual observations during a recent site visit, there are no visible inlet controls installed in the existing catchbasins.

As part of the proposed development, it is required that runoff from the existing development to the south be pumped across to the existing 375 mm diameter storm outlet during construction and that it be conveyed through the proposed building plumbing system to the outlet after development.

5.3 SWM CRITERIA AND CONSTRAINTS

The stormwater management criteria for the proposed site are based on City of Ottawa Sewer Design Guidelines (2012) and on consultation with City of Ottawa Staff. The following summarizes the criteria used in the preparation of this stormwater management plan:

- All stormwater runoff from the proposed development up to and including the 100-year event to be stored on site and released into the minor system at a maximum rate equivalent to the 5-year storm with a runoff coefficient (C) equal to 0.5
- Maximum 100-year water depth of 0.35 m in parking and access areas
- Provide adequate emergency overflow conveyance (overland flow route) off-site
- Provide a storm outlet for the existing development to the south
- Size the storm lateral to convey the 5-year storm event, assuming only roof controls are imposed (i.e. provide capacity for system without inlet control devices installed)

- Size storm sewers using an inlet time of concentration (Tc) of 10 minutes
- Post-development runoff coefficient (C) value based on proposed impervious areas as per site plan drawing (see **Appendix B**)

5.4 STORMWATER MANAGEMENT DESIGN

The proposed 0.31 ha re-development area consists of an eleven-storey residential building, underground parking, access and landscaped areas, and associated servicing infrastructure. The overall imperviousness of the site is 70% (C = 0.69).

It is proposed to direct stormwater runoff from the proposed development and the existing development to the south to the current site outlet through the existing 375 mm diameter CSP. Runoff from the existing development to the south will be conveyed through the proposed building's plumbing system to the site outlet. A combination of roof storage, a cistern and a sump pump located in the underground parking are proposed to restrict post development peak flows from the proposed re-development area to the allowable release rate which is equivalent to the 5-year runoff with a C of 0.5. Similarly, it is proposed to install an oil grit separator within the underground parking structure to provide the required 80% TSS removal from runoff from the proposed development. A sump pump is required to discharge the foundation drain. The conceptual site plan and existing storm sewer infrastructure are shown on **Drawing SSP-1**.

5.4.1 Design Methodology

The intent of the stormwater management plan presented herein is to mitigate any negative impact that the proposed development could have on the existing storm sewer infrastructure, while providing adequate capacity to service the proposed building, parking and access areas. The proposed stormwater management plan is designed to detain runoff on the rooftop and in the subsurface to ensure that peak flows after construction from the proposed re-development area will not exceed the target release rate for the site, and to provide a stormwater outlet for the existing development to the south.

A small portion of the site fronting Richmond Road could not be graded to enter the building's internal plumbing system and as such it will sheet drain uncontrolled. Runoff from this uncontrolled area is included in the overall site discharge calculations.

5.4.2 Water Quantity Control

The Modified Rational Method was used to assess the quantity and volume of runoff generated during post development conditions. The site was subdivided into subcatchments (subareas) tributary to storm sewer inlets, as defined by the location of catchbasins / inlet grates, and used in the storm sewer design (see **Appendix D**). A summary of subareas and runoff coefficients is provided in **Appendix D**, and **Drawing SD-1** indicates the stormwater management subcatchments.

5.4.3 Allowable Release Rate

Site discharge rates up to the 100-year storm event are to be restricted to the 5-year storm event with a runoff coefficient, 'C' value, of 0.50 as outlined below:

Table 3: Target Release Rate

Rational Method 'C'	Area (ha)	Time of Concentration (min)	Q_{Target} (L/s)
0.50	0.31	10	44.9

5.4.4 Storage Requirements

The site requires quantity control measures to meet the stormwater release criteria. It is proposed that restricted release rooftop drains be used to reduce the peak outflow from the site. Additionally, a subsurface storage tank is proposed to reduce peak outflows from all proposed site areas connected to the internal plumbing system of the building to meet the site target discharge rate. **Drawing SD-1** indicates the design release rate from the rooftop and the underground storage system. Stormwater management calculations are provided in **Appendix D**.

5.4.4.1 Rooftop Storage

It is proposed to retain stormwater on the rooftops by installing restricted flow roof drains. The following calculations assume the proposed roof will be equipped with four standard Watts Model R1100 Accuflow Roof Drains fully open.

Watts "Accuflow" roof drain data has been used to calculate a practical roof release rate and detention storage volume for the rooftop. It should be noted that the "Accuflow" roof drain has been used as an example only and that other products may be specified for use, provided that the roof release rate is restricted to match the maximum rate of release indicated in **Table 4** and **Table 5** and that sufficient roof storage is provided to meet (or exceed) the resulting volume of detained stormwater.

Table 4 and **Table 5** provide details regarding the retention of stormwater on the proposed rooftop during the 5 and 100-year storm events. Refer to **Appendix D** for details.

Table 4: Peak Controlled (Rooftop) 5-Year Release Rate

Area ID	Area (ha)	Head (m)	$Q_{release}$ (L/s)	V_{stored} (m^3)
BLDG	0.110	0.11	5.7	16.7

Table 5: Peak Controlled (Rooftop) 100-Year Release Rate

Area ID	Area (ha)	Head (m)	$Q_{release}$ (L/s)	V_{stored} (m^3)
BLDG	0.110	0.15	7.5	37.3

5.4.4.2 Subsurface Storage

In addition to rooftop storage, it is proposed to detain stormwater within a 20 m^3 cistern below grade with a maximum controlled release rate of 29.7 L/s to the gravity service provided. The modified rational method was used to determine the peak volume requirement for the cistern. Where possible, site drainage areas are captured into the building plumbing directed to the cistern for additional control.

Table 6 and **Table 7** summarize the flow rates to the cistern.

Table 6: Peak Controlled (Tributary) 5-Year Release Rate

Area ID	Area (ha)	Runoff 'C'	$Q_{release}$ (L/s)	V_{stored} (m^3)
BLDG	0.11	0.90	5.7	16.7
L201A	0.04	0.67	7.8	0.00
L202A	0.11	0.51	16.3	0.00
RAMP	0.01	0.90	2.6	0.00
Cistern	0.27	0.71	32.3	3.6

Table 7: Peak Controlled (Tributary) 100-Year Release Rate

Area ID	Area (ha)	Runoff 'C'	$Q_{release}$ (L/s)	V_{stored} (m^3)
BLDG	0.11	1.00	7.5	37.3
L201A	0.04	0.84	15.1	0.00
L202A	0.11	0.64	34.8	0.00
RAMP	0.01	1.00	5.0	0.00
Cistern	0.27	0.88	29.7	19.6

5.4.5 Uncontrolled Area

A small portion of the site fronting Richmond Road (see area UNC-1 and UNC-2 on **Drawing SD-1**) could not be graded to enter the building's internal plumbing system and as such they will sheet drain uncontrolled. **Table 8** and **Table 9** summarize the 5 and 100-year uncontrolled release rates from the proposed development.

Table 8: Peak Uncontrolled (Non-tributary) 5-Year Release Rate

Area ID	Area (ha)	Runoff 'C'	Tc (min)	$Q_{release}$ (L/s)
UNC-1&UNC-2	0.04	0.61	10	7.1

Table 9: Peak Uncontrolled (Non-tributary) 100-Year Release Rate

Area ID	Area (ha)	Runoff 'C'	Tc (min)	$Q_{release}$ (L/s)
UNC-1&UNC-2	0.04	0.77	10	15.2

5.4.6 Results

Table 10 and **Table 11** demonstrate that the proposed stormwater management plan provides adequate attenuation storage to meet the target peak outflow for the site.

Table 10: Estimated Discharge from Site (5-Year)

Area Type	Area (ha)	V_{stored} (m^3)	$Q_{release}$ (L/s)	Target (L/s)
Controlled – Subsurface (Includes Roof area)	0.27	20	29.7	**44.9**
Uncontrolled – (UNC-1, UNC-2)	0.04	0.0	7.1	
Total	**0.31**	**20**	**36.8**	

Table 11: Estimated Discharge From Site (100-Year)

Area Type	Area (ha)	V_{stored} (m^3)	$Q_{release}$ (L/s)	Target (L/s)
Controlled – Subsurface (Includes Roof area)	0.27	57	29.7	**44.9**
Uncontrolled – (UNC-1, UNC-2)	0.04	0.0	15.2	
Total	**0.31**	**57**	**44.9**	

5.5 EXTERNAL AREA

Runoff from the existing 11-storey building and parking areas immediately south of the proposed development will be connected to the proposed building internal plumbing system and directed to the proposed site outlet. Based on observations during a recent site visit, it was concluded that the existing catchbasins in the adjacent site are not equipped with inlet control devices. Similarly, it has been assumed that the existing building is connected to the existing storm sewer system without any type of roof storage/runoff control.

Based on the above, it is estimated that approximately 55.3 L/s are generated from the existing development in the 5-year storm event and assuming a 10% capture increase in the 100-year storm event, a peak flow of approximately 61 L/s will need to be conveyed through the proposed building's internal plumbing system to the site outlet.

5.6 QUALITY CONTROL

As per correspondence with Rideau Valley Conservation Authority (RVCA) staff, runoff from the proposed development requires 'Enhanced' quality treatment (80% TSS removal) prior to discharge into the site outlet which ultimately directs runoff to the Rideau River.

As a result, it is proposed to install an oil/grit separator (OGS) unit within the underground parking structure to provide the required level of treatment of runoff from the proposed site areas. The PCSWMM for Stormceptor software has been used to provide preliminary sizing. It should be noted that the Stormceptor unit has been used as an example only and that other products may be specified for use, provided that they meet the required level of treatment.

Based on a drainage area of 0.16 ha (roof area not included) with a total imperviousness of 53% and treatment for fine particle size distribution, a Stormceptor unit STC300 will provide 88% TSS removal.

6.0 GRADING AND DRAINAGE

The proposed re-development site measures approximately 0.31 ha in area. The site currently sheet drains towards three existing catchbasins. A detailed grading plan (see **Drawing GP-1**) has been provided to satisfy the stormwater management requirements and to provide sufficient cover over top of the underground parking garage. Site grading has been established to provide emergency overland flow routes for stormwater management in accordance with City of Ottawa requirements.

The subject site maintains emergency overland flow routes to the existing property to the north as depicted on **Drawings GP-1** and **SD-1**.

7.0 UTILITIES

All utilities (Hydro Ottawa, Bell Canada, Rogers Ottawa, and Enbridge Gas) have existing plants in the area. The site will be serviced through connection to these existing services. Detailed design of the required utility services will be further investigated as part of the composite utility planning process following design circulation.

8.0 EROSION CONTROL DURING CONSTRUCTION

Erosion and sediment controls must be in place during construction. The following recommendations to the contractor will be included in contract documents.

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit extent of exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with plastic or synthetic mulches.
6. Provide sediment traps and basins during dewatering.
7. Install sediment traps (such as SiltSack® by Terrafix) between catch basins and frames.
8. Plan construction at proper time to avoid flooding.
9. Installation of a mud matt to prevent mud and debris from being transported off site.
10. Installation of a silt fence to prevent sediment runoff.

The contractor will, at every rainfall, complete inspections and guarantee proper performance. The inspection is to include:

1. Verification that water is not flowing under silt barriers.
2. Clean and change silt traps at catch basins.

Refer to **Drawing EC/DS-1** for the proposed location of silt fences, and other erosion control structures.

9.0 GEOTECHNICAL INVESTIGATION

A geotechnical report was prepared by Paterson Group October 2007 (see **Appendix E**). As stated in the geotechnical report, the subsurface profile across the site consists of 60 to 100 mm thickness of asphalt overlying a granular layer. The pavement structure lies atop a fill layer, consisting of brown to grey sand and gravel with trace to some silt and clay that extends to a depth of approximately 1.5 to 2.5 m. A native glacial till deposit was encountered underlying the above-noted fill layers, followed by grey limestone bedrock.

Groundwater levels were measured on June 8, 2017 and were found to range between 2.2 m and 3.7 m.

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. Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps. A temporary MOECC permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. A minimum of four to five months should be allocated for completion of the application and issuance of the permit by the MOECC.

Bedrock removal will be required to complete the two (2) levels of underground parking. The geotechnical report recommended line drilling and controlled blasting to remove the bedrock. The report also recommended that prior to considering blasting operations, the effects on the existing services, buildings and other structures should be addressed.

An alignment of a large diameter watermain runs within an easement along the north property boundary of the subject site. It is expected that the adjacent watermain could be subjected to potential vibrations associated with the bedrock blasting program. To ensure that no detrimental vibrations cause damage to the adjacent watermain, a vibration attenuation trench is recommended for the bedrock along the north excavation face, as well as a vibration monitoring and control program during the blasting and excavation work required for the proposed building excavation (please refer to the Geotechnical report included in **Appendix E** for details).

The geotechnical report also recommended that a perimeter foundation drainage system be provided for the proposed structures. Given that it is expected that insufficient room will be available for exterior backfill, the report suggested that the foundation drainage system could be as follows:

- Bedrock vertical surface (Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface);

- Composite drainage layer.

It was recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) extend down to the footing level. It was also recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area for mechanical evacuation.

10.0 CONCLUSIONS

10.1 WATER SERVICING

The 203 mm diameter watermain on Richmond Road provides adequate fire flow capacity as per the Fire Underwriters Survey. In order to meet the City water supply objective that limits a single feed to 50 m^3/d during basic day demands, dual connection to the existing 203 mm diameter watermain on Richmond Road is required to service the proposed building. The service connection will be capable of providing anticipated demands to the lower storeys but will require a booster pump to maintain pressures of 276 kPa (40 psi) for floors 7 to 11.

10.2 SANITARY SERVICING

The proposed sanitary sewer lateral is sufficiently sized to provide gravity drainage for the site. The proposed site will be serviced by a 200 mm diameter service lateral directing wastewater flows to the existing 225 mm dia. Richmond Road sanitary sewer. A backflow preventer will be required for the proposed building in accordance with the Ottawa sewer design guide, and will be coordinated with building mechanical engineers. The proposed sanitary drainage pattern is in accordance with direction from pre-consultation with City of Ottawa staff.

10.3 STORMWATER SERVICING

The proposed stormwater management plan is in compliance with the goals specified through consultation with the City of Ottawa, as well as local standards. Subsurface and rooftop storage is proposed to limit inflow from the site area into the minor system to the required target release rate. An underground cistern and pump will be required to direct controlled release rates from the site to the proposed gravity service connected to the existing 375 mm dia. CSP running north and ultimately discharging into the Cleary Street storm sewer. An oil grit separator will be installed within the underground parking structure to provide 80% TSS removal for runoff generated from the proposed development areas.

10.4 GRADING

Grading for the site has been designed to provide an emergency overland flow route as per City requirements. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing infrastructure. An alignment of a large diameter watermain runs within an easement along the north property boundary of the subject site. It is expected that the adjacent watermain could be subjected to potential vibrations associated with the bedrock blasting program. To ensure that no detrimental vibrations cause damage to the adjacent watermain, a vibration attenuation trench is recommended for the bedrock along the north excavation face, as well as a vibration monitoring and control program during the blasting and excavation work required for the proposed building excavation.

10.5 UTILITIES

All utilities (Hydro Ottawa, Bell Canada, Rogers Ottawa, and Enbridge Gas) have existing plants in the subject area. Exact size, location and routing of utilities will be finalized after design circulation.

10.6 APPROVAL / PERMITS

Ministry of the Environment and Climate Change (MOECC) Environmental Compliance Approvals (ECA) are not expected to be required for the subject site as the site is private and will remain under singular ownership. A Permit to Take Water may be required for pumping requirements for construction of underground parking level. No other approval requirements from other regulatory agencies are anticipated.

APPENDICES

Appendix A HYDRAULIC ANALYSIS

851 Richmond Road - Domestic Water Demand Estimates
- Based on Roderick Lahey Architect Inc Site plan June 6, 2017

Building ID	Area (m^2)	Population	Daily Rate of Demand [1]	Avg Day Demand (L/min)	Avg Day Demand (L/s)	Max Day Demand (L/min)	Max Day Demand (L/s)	Peak Hour Demand (L/min)	Peak Hour Demand (L/s)
Residential	11,424	227.5	350	55.3	0.92	138.2	2.30	304.1	5.07
Total Site :				**55.3**	**0.92**	**138.2**	**2.30**	**304.1**	**5.07**

1 Water demand criteria used to estimate peak demand rates for residential areas are as follows:
maximum day demand rate = 2.5 x average day demand rate
maximum hour demand rate = 2.2 x maximum day demand rate

Stantec

FUS Fire Flow Calculation

Calculations based on: "*Water Supply for Public Fire Protection*" by Fire Underwriters' Survey, 1999

Stantec Project #: 1604-01329
Project Name: 851 Richmond Road
Date: June 21, 2017
Data input by: Shika Rathnasooriya

Fire Flow Calculation #: 1
Building Type/Description/Name: Apartment Building

Notes:
Building Classification C, 2 hour fire seperation between each floor.

Table A: Fire Underwriters Survey Determination of Required Fire Flow - Long Method

Step	Task	Term	Options	Multiplier Associated with Option	Choose:	Value Used	Unit	Total Fire Flow (L/min)
		Framing Material						
1	Choose Frame Used for Construction of Unit	Coefficient related to type of construction (C)	Wood Frame	1.5	Non-combustible construction	0.8	-	
			Ordinary construction	1				
			Non-combustible construction	0.8				
			Fire resistive construction (> 3 hrs)	0.6				
		Floor Space Area						
2	Choose Type of Housing (if TH, Enter Number of Units Per TH Block)	Type of Housing	Single Family	1	Other (Comm, Ind, Apt etc.)	1	Units	
			Townhouse - indicate # of units	8				
			Other (Comm, Ind, Apt etc.)	1				
2.2	# of Storeys	Number of Floors/Storeys in the Unit (do not include basement):			1	1	Storeys	
3	Enter Ground Floor Area of One Unit	Average Floor Area (A) based on fire resistive building design when vertical openings are inadequately protected:			1,142 Square Metres (m2)	1,142	Area in Square Meters (m^2)	
4	Obtain Required Fire Flow without Reductions	Required Fire Flow (without reductions or increases per FUS) (F = 220 * C * √A) Round to nearest 1000L/min						6,000
5	Apply Factors Affecting Burning	**Reductions/Increases Due to Factors Affecting Burning**						
5.1	Choose Combustibility of Building Contents	Occupancy content hazard reduction or surcharge	Non-combustible	-0.25	Limited combustible	-0.15	N/A	5,100
			Limited combustible	-0.15				
			Combustible	0				
			Free burning	0.15				
			Rapid burning	0.25				
5.2	Choose Reduction Due to Presence of Sprinklers	Sprinkler reduction	Adequate Sprinkler conforms to NFPA13	-0.3	Adequate Sprinkler conforms to NFPA13	-0.3	N/A	-1,530
			None	0				
		Water Supply Credit	Water supply is standard for sprinkler and fire dept. hose line	-0.1	Water supply is standard for sprinkler and fire dept. hose line	-0.1	N/A	-510
			Water supply is not standard or N/A	0				
		Sprinkler Supervision Credit	Sprinkler system is fully supervised	-0.1	Sprinkler not fully supervised or N/A	0	N/A	0
			Sprinkler not fully supervised or N/A	0				
5.3	Choose Separation Distance Between Units	Exposure Distance Between Units	North Side	45.1m or greater	0	0.35	m	1,785
			East Side	45.1m or greater	0			
			South Side	3.1 to 10.0m	0.2			
			West Side	10.1 to 20.0m	0.15			
6	Obtain Required Fire Flow, Duration & Volume	*Total Required Fire Flow, rounded to nearest 1000 L/min, with max/min limits applied:*						**5,000**
		Total Required Fire Flow (above) in L/s:						**83**
		Required Duration of Fire Flow (hrs)						**1.75**
		Required Volume of Fire Flow (m^3)						**525**

From:	Balima, Nadege
To:	Rathnasooriya, Thakshika
Subject:	RE: Hydraulic Boundary Conditions - 851 Richmond Road
Date:	Tuesday, June 27, 2017 3:06:47 PM
Attachments:	image001.gif 851 Richmond June 2017.pdf

Hi Shika,
I have just received the results of the boundary condition request for the site in subject. Please find them below.

The following are boundary conditions, HGL, for hydraulic analysis at 851 Richmond (zone 1W) assumed to be connected to the 203mm on Richmond (see attached PDF for location).

Minimum HGL = 108.6

Maximum HGL = 116.2m

MaxDay (2.3 L/s) + FireFlow (83 L/s) = 99.0m

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.

Please refer to Guidelines and Technical bulletin ISDTB-2014-02 concerning basic day demands greater than 0.5 L/s.

Please let me know if you have questions.

Regards,

Nadège Balima, P.Eng., M.P.M., LEED Green Assoc.
Project Manager, Infrastructure Approvals
Development Review Services (West)
613.580.2424 ext. 13477

From: Rathnasooriya, Thakshika [mailto:Thakshika.Rathnasooriya@stantec.com]
Sent: Tuesday, June 27, 2017 11:33 AM
To: Balima, Nadege <Nadege.Balima@ottawa.ca>
Subject: RE: Hydraulic Boundary Conditions - 851 Richmond Road

Hi Nadege,

Is it possible to have a status update on the hydraulic boundary conditions for this site?

Thank you,

Shika Rathnasooriya
Engineering Intern
Stantec
400 - 1331 Clyde Avenue, Ottawa ON K2C 3G4
Phone: (613) 724-4081
Thakshika.Rathnasooriya@stantec.com

From: Balima, Nadege [mailto:Nadege.Balima@ottawa.ca]
Sent: Friday, June 23, 2017 8:52 AM
To: Rathnasooriya, Thakshika <Thakshika.Rathnasooriya@stantec.com>
Subject: RE: Hydraulic Boundary Conditions - 851 Richmond Road

Good morning Shika,
I have forwarded your request for processing and will get back to you as soon as I have results.
Thanks,

Nadège Balima, P.Eng., M.P.M., LEED Green Assoc.
Project Manager, Infrastructure Approvals
Development Review Services (West)
613.580.2424 ext. 13477

From: Rathnasooriya, Thakshika [mailto:Thakshika.Rathnasooriya@stantec.com]
Sent: Wednesday, June 21, 2017 1:50 PM
To: Balima, Nadege <Nadege.Balima@ottawa.ca>
Cc: Paerez, Ana <Ana.Paerez@stantec.com>
Subject: Hydraulic Boundary Conditions - 851 Richmond Road

Hello Nadege,

I am looking for watermain hydraulic boundary conditions for the proposed site at 851 Richmond Road. We anticipate connecting to the existing 200mm watermain on Richmond Road.

Attached are the FUS calculations for the proposed building. The intended land use is residential, for a 11 storey apartment building comprising 132 units with 61 two-bedrooms units and 71 one-bedroom units.

Estimated domestic demands and fire flow requirements for the site are as follows:
Average Day Demand – 0.92L/s
Max Day Demand – 2.30L/s

Peak Hour Demand – 5.07L/s
Fire Flow Requirement per FUS - 83L/s (2 hour fire separation between each floor)

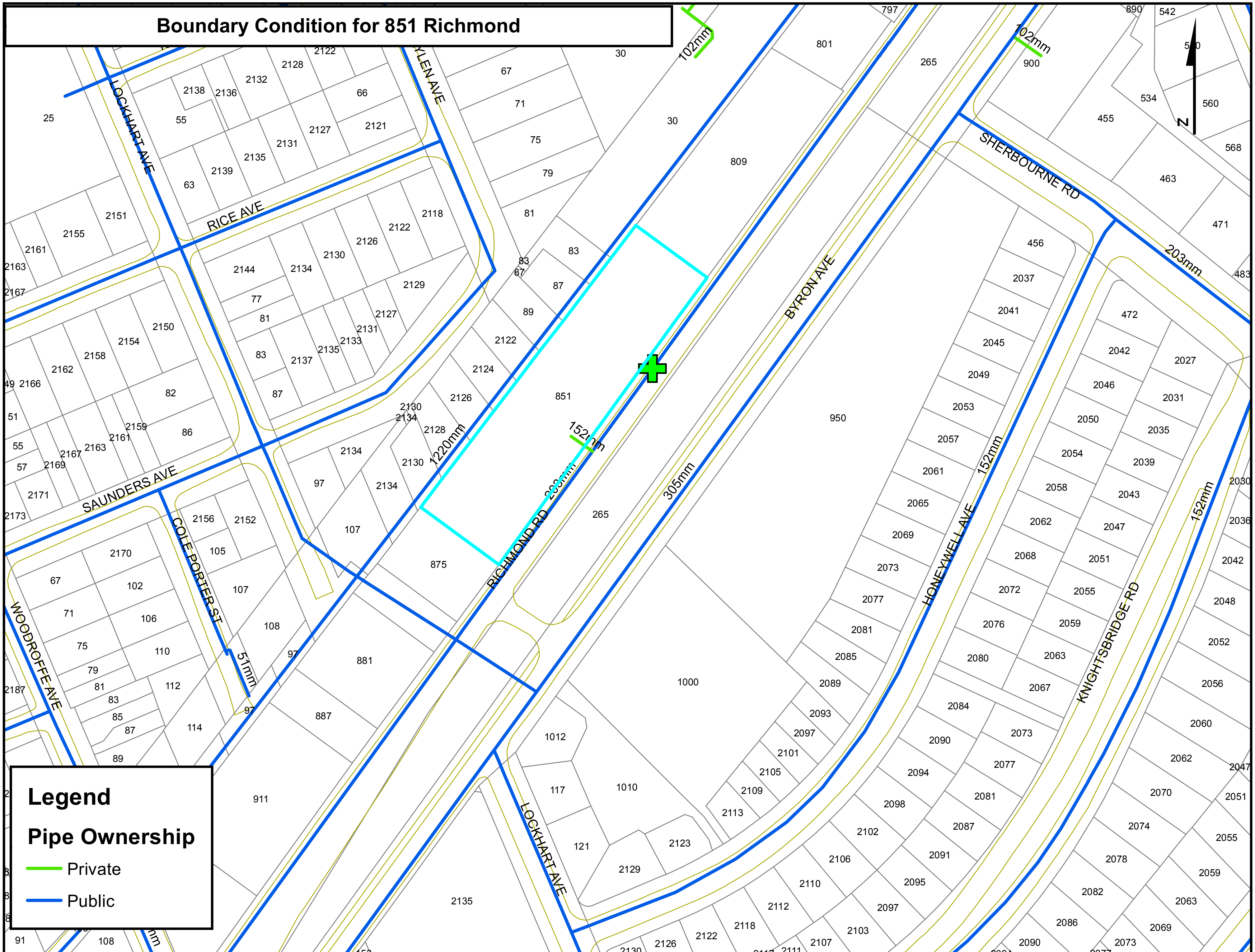
Thanks,

Shika Rathnasooriya
Engineering Intern
Stantec
400 - 1331 Clyde Avenue, Ottawa ON K2C 3G4
Phone: (613) 724-4081
Thakshika.Rathnasooriya@stantec.com

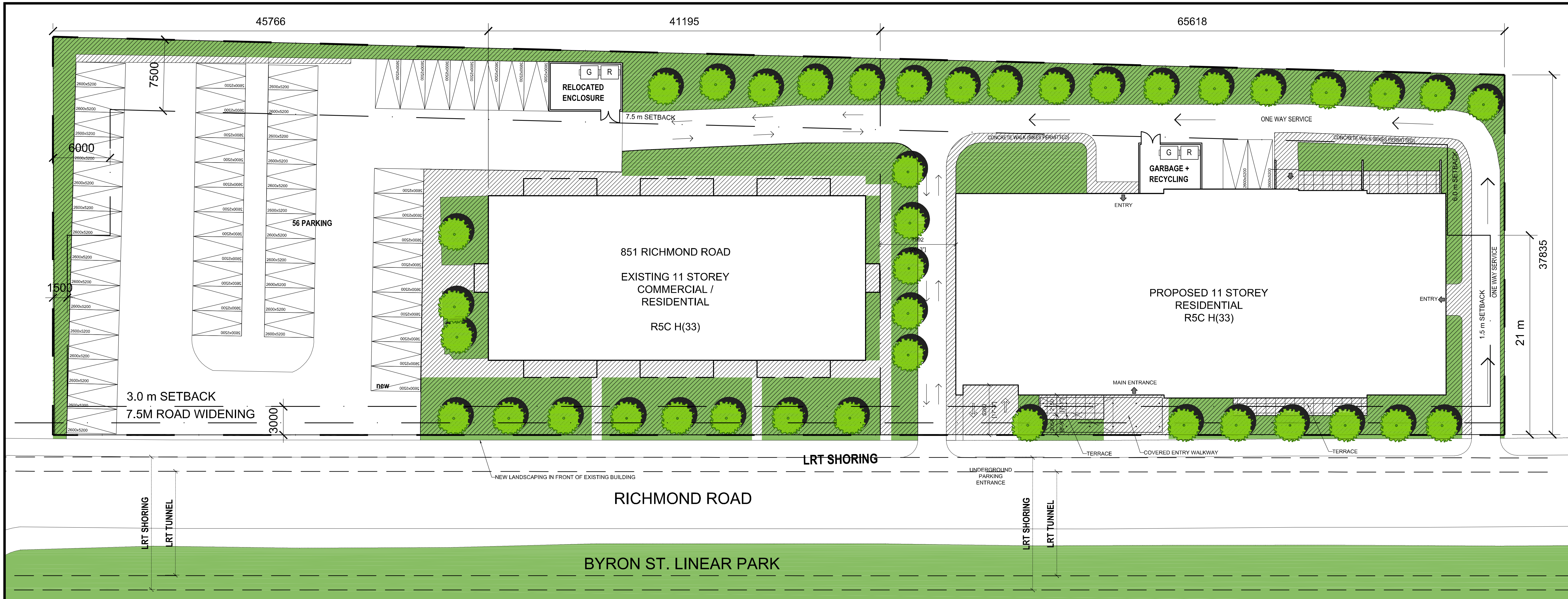
This e-mail originates from the City of Ottawa e-mail system. Any distribution, use or copying of this e-mail or the information it contains by other than the intended recipient(s) is unauthorized. Thank you.

Le présent courriel a été expédié par le système de courriels de la Ville d'Ottawa. Toute distribution, utilisation ou reproduction du courriel ou des renseignements qui s'y trouvent par une personne autre que son destinataire prévu est interdite. Je vous remercie de votre collaboration.

Boundary Condition for 851 Richmond



Appendix B PROPOSED SITE PLAN



BUILDING INFORMATION

OPEN SPACE
FULL SITE AREA: 65,474 sf
OPEN SPACE:

TOTAL 20,122 sf **30.7%**

NEW PROPOSED BUILDING SITE AREA: 27,351 sf
OPEN SPACE:

TOTAL 10,164 sf **37.16%**

OPEN GREEN SPACE

OPEN SPACE

UNIT TOTALS
GROUND FLOOR : 2 UNITS, 3.5 m height
LEVELS 2 - 11: 13 UNITS per floor (130) 2.9 m height
TOTAL: **132 UNITS**

CONSTRUCTION AREA
LEVEL 1 11,358 sf
LEVELS 2 -11 12,297 sf per floor (122,970)
TOTAL: **134,328 sf**

BLDG PARKING TOTALS
LEVEL P2: 67
LEVEL P1: 63
TOTAL: **130**

STORAGE SPACE & BIKE TOTALS
3x6 STORAGE LOCKERS (CAN BE USED FOR BIKES): 76
3X4 STORAGE LOCKERS: 66
BIKE PARKING (ground floor): 38
TOTAL BIKE STG CAPACITY: 114
TOTAL STORAGE LOCKERS: 142

AMENITY SPACE
AMENITY SMALL (MAIN FLR): 854sf
AMENITY LARGE (MAIN FLR): 2,530sf
AMENITY BALCONIES: (COMBINED LEVELS 2-11) 10,268sf

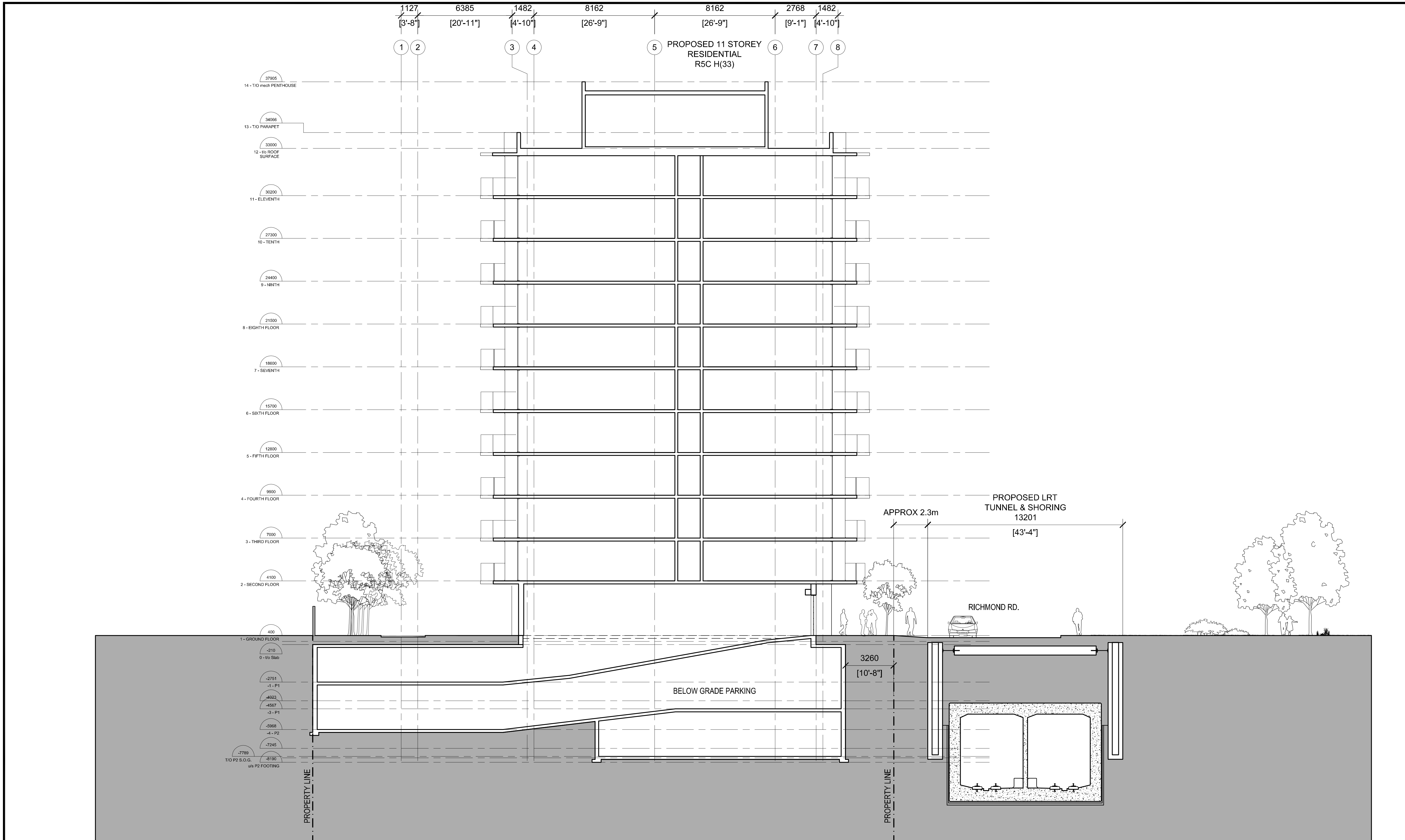
0m 5 10 15

RODERICKLAHEY
ARCHITECT INC

SITE PLAN
SCALE 1:400 date:August 11, 2017

851 RICHMOND ROAD
OTTAWA ONTARIO

SHEET #
a100
PROJ. # 1705



RODERICKLAHEY
ARCHITECT INC

TRANSVERSE BUILDING SECTION & LRT

SCALE TO FIT date:August 11, 2017

851 RICHMOND ROAD
OTTAWA ONTARIO

SHEET #
a300

PROJ. # 1705

RODERICKLAHEY ARCHITECT INC

PLAN - TYPICAL RESIDENTIAL FLOOR, LEVELS 3-11

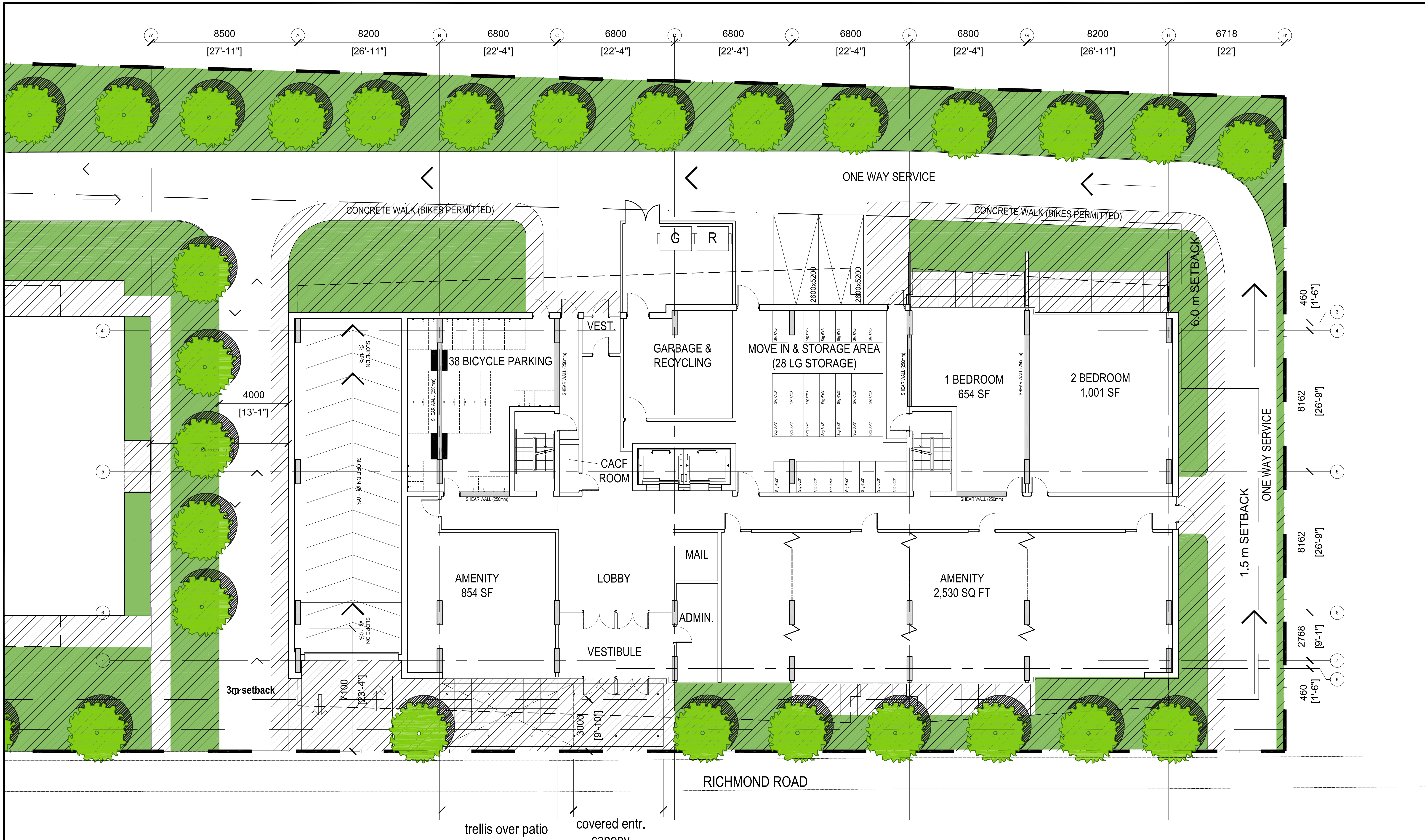
SCALE 1: 200

date:August 11, 2017

851 RICHMOND ROAD
OTTAWA ONTARIO

SHEET #
a204

PROJ. # 1705



RODERICKLAHEY
ARCHITECT INC

LEVEL 1 - MAIN/ GROUND
SCALE 1:200

851 RICHMOND ROAD
OTTAWA ONTARIO

SHEET #
4
PROJ. # 1705

Appendix C SANITARY SEWER CALCULATIONS

Stantec

SUBDIVISION:

3500 HAWTHORNE ROAD

DATE: 2017/03/24
REVISION: 1
DESIGNED BY: WAJ
CHECKED BY: AMP

SANITARY SEWER DESIGN SHEET

(City of Ottawa)

FILE NUMBER: 160401302

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	350 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	50,000 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 (COMM., INST.):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / SINGLE	3.4	INSTITUTIONAL	50,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / 1 BED APT	1.4	INFILTRATION	0.28 l/s/Ha		
PERSONS / 2 BED APT	2.1				

LOCATION			RESIDENTIAL AREA AND POPULATION								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	UNITS SINGLE	UNITS 1 BED APT	UNITS 2 BED APT	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)
BLDG	1	EX SAN	0.25	0	71	61	228	0.25	228	4.00	3.7

AREA ID NUMBER	COMMERCIAL AREA (ha)	COMMERCIAL ACCU. AREA (ha)	INDUSTRIAL (L) AREA (ha)	INDUSTRIAL (L) ACCU. AREA (ha)	INDUSTRIAL (H) AREA (ha)	INDUSTRIAL (H) ACCU. AREA (ha)	INSTITUTIONAL AREA (ha)	INSTITUTIONAL ACCU. AREA (ha)	GREEN / UNUSED AREA (ha)	GREEN / UNUSED ACCU. AREA (ha)	C+I+I PEAK FLOW (l/s)	INFILTRATION TOTAL AREA (ha)	INFILTRATION ACCU. AREA (ha)	INFILT. FLOW (l/s)	TOTAL FLOW (l/s)
BLDG	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.25	0.25	0.1	3.8

AREA ID NUMBER	PIPE LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
BLDG	3.5	200	PVC	SDR 35	2.00	47.3	**7.94%**	1.49	0.74
		200							

Appendix D STORMWATER MANAGEMENT CALCULATIONS

Stantec

851 RICHMOND ROAD

DATE: 2017/10/03
REVISION: 2
DESIGNED BY: WAJ
CHECKED BY: AMP

STORM SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401329

DESIGN PARAMETERS

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

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

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

LOCATION / DRAINAGE AREA

AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (5-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (100-YEAR) (ha)	AREA (ROOF) (ha)	C (2-YEAR) (-)	C (5-YEAR) (-)	C (10-YEAR) (-)	C (100-YEAR) (-)	A x C (2-YEAR) (ha)	ACCUM AxC (2YR) (ha)	A x C (5-YEAR) (ha)	ACCUM. AxC (5YR) (ha)	A x C (10-YEAR) (ha)	ACCUM. AxC (10YR) (ha)	A x C (100-YEAR) (ha)	ACCUM. AxC (100YR) (ha)	T of C (min)	$I_{2\text{-YEAR}}$ (mm/h)	$I_{5\text{-YEAR}}$ (mm/h)	$I_{10\text{-YEAR}}$ (mm/h)	$I_{100\text{-YEAR}}$ (mm/h)	$Q_{CONTROL}$ (L/s)	ACCUM. $Q_{CONTROL}$ (L/s)	Q_{ACT} (CIA/360) (L/s)
EX-CB2	EX-CB2	EX-CB1	0.00	0.07	0.00	0.00	0.00	0.00	0.84	0.00	0.00	0.000	0.000	0.059	0.059	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	17.0
EX-CB1, EX-BLDG	EX-CB1	102	0.00	0.17	0.00	0.00	0.00	0.00	0.88	0.00	0.00	0.000	0.000	0.149	0.208	0.000	0.000	0.000	0.000	10.56	74.73	101.33	118.77	173.61	0.0	0.0	58.5
	102	STUB	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.000	0.000	0.208	0.000	0.000	0.000	0.000	11.76	70.64	95.72	112.16	163.90	0.0	0.0	55.3
BLDG, L201A, L202A	BLDG	EX CSP	0.00	0.26	0.00	0.00	0.00	0.00	0.70	0.00	0.00	0.000	0.000	0.182	0.390	0.000	0.000	0.000	0.000	11.76	70.64	95.72	112.16	163.90	0.0	0.0	103.6
																				11.82							

PIPE SELECTION

AREA ID NUMBER	FROM M.H.	TO M.H.	LENGTH (m)	PIPE WIDTH OR DIAMETEI (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE %	Q_{CAP} (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	VEL. (ACT) (m/s)	TIME OF FLOW (min)
EX-CB2	EX-CB2	EX-CB1	29.0	200	200	CIRCULAR	PVC	-	0.90	31.6	**53.86%**	0.99	0.87	0.56
EX-CB1, EX-BLDG	EX-CB1	102	64.6	250	250	CIRCULAR	PVC	-	0.54	44.4	**131.79%**	0.89	0.89	1.20
	102	STUB	45.2	300	300	CIRCULAR	PVC	-	0.50	68.0	**81.27%**	0.97	0.96	0.79
BLDG, L201A, L202A	BLDG	EX CSP	3.4	375	375	CIRCULAR	CSP	-	0.38	101.6	**101.97%**	0.96	1.02	0.06
				375	375									

Stormwater Management Calculations

File No: **160401329**
Project: **851 RICHMOND ROAD**
Date: **03-Oct-17**

SWM Approach:
Post-development to 5-year equivalent with a C=0.50

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**
Roof	BLDG	Hard	0.110		0.9	0.099		
		Soft	0.000		0.2	0.000		
		Subtotal		0.11			0.099	0.90
Uncontrolled - Tributary	L201A	Hard	0.027		0.9	0.024		
		Soft	0.013		0.2	0.003		
		Subtotal		0.04			0.0268	0.67
Uncontrolled - Tributary	L202A	Hard	0.049		0.9	0.044		
		Soft	0.061		0.2	0.012		
		Subtotal		0.11			0.0561	0.51
Unontrolled - Tributary	RAMP	Hard	0.010		0.9	0.009		
		Soft	0.000		0.2	0.000		
		Subtotal		0.01			0.009	0.90
Uncontrolled - Non-Tributary	UNC-1	Hard	0.007		0.9	0.006		
		Soft	0.003		0.2	0.001		
		Subtotal		0.01			0.0068	0.68
Uncontrolled - Non-Tributary	UNC-2	Hard	0.017		0.9	0.015		
		Soft	0.013		0.2	0.003		
		Subtotal		0.03			0.0177	0.59
Total				**0.310**			**0.215**	
Overall Runoff Coefficient= C:								**0.69**

Total Roof Areas	**0.110 ha**		
Total Tributary Surface Areas (Controlled and Uncontrolled)	**0.160 ha**		
Total Tributary Area to Outlet	**0.270 ha**	**C = 0.71**	
Total Uncontrolled Areas (Non-Tributary)	**0.040 ha**		
Total Site	**0.310 ha**		
Total to Oil/Grit Separator	**0.160 ha**	**C = 0.57**	**53%**

Stormwater Management Calculations

Project #160401329, 851 RICHMOND ROAD
Modified Rational Method Calculatons for Storage

5 yr Intensity I = a/(t + b)^c
City of Ottawa

		t (min)	I (mm/hr)
a =	998.071	5	141.18
b =	6.053	10	104.19
c =	0.814	15	83.56
		20	70.25
		25	60.90
		30	53.93
		35	48.52
		40	44.18
		45	40.63
		50	37.65
		55	35.12
		60	32.94

5 YEAR Target Release from Portion of Site

Subdrainage Area: Predevelopment Tributary Area to Outlet
Area (ha): 0.3100
C: 0.50

Typical Time of Concentration

tc (min)	I (5 yr) (mm/hr)	Qtarget (L/s)
10	104.19	**44.90**

5 YEAR Modified Rational Method for Entire Site

Subdrainage Area: BLDG — Roof
Area (ha): 0.11 — 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)
5	141.18	38.86	5.65	33.21	9.96
10	104.19	28.68	5.65	23.03	13.82
15	83.56	23.00	5.65	17.35	15.62
20	70.25	19.33	5.65	13.69	16.43
25	60.90	16.76	5.65	11.11	**16.67**
30	53.93	14.84	5.65	9.20	16.55
35	48.52	13.35	5.65	7.71	16.19
40	44.18	12.16	5.65	6.51	15.64
45	40.63	11.18	5.65	5.54	14.95
50	37.65	10.36	5.65	4.72	14.15
55	35.12	9.67	5.65	4.02	13.27
60	32.94	9.07	5.65	3.42	12.32

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
5-year Water Level	111.86	0.11	5.65	16.67	38.75	0.00

Subdrainage Area: L201A — Uncontrolled - Tributary
Area (ha): 0.04
C: 0.67

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	141.18	10.52	10.52	0.00	**0.00**
10	104.19	7.76	7.76	0.00	**0.00**
15	83.56	6.23	6.23	0.00	**0.00**
20	70.25	5.23	5.23	0.00	**0.00**
25	60.90	4.54	4.54	0.00	**0.00**
30	53.93	4.02	4.02	0.00	**0.00**
35	48.52	3.61	3.61	0.00	**0.00**
40	44.18	3.29	3.29	0.00	**0.00**
45	40.63	3.03	3.03	0.00	**0.00**
50	37.65	2.81	2.81	0.00	**0.00**
55	35.12	2.62	2.62	0.00	**0.00**
60	32.94	2.45	2.45	0.00	**0.00**

Subdrainage Area: L202A — Uncontrolled - Tributary
Area (ha): 0.11
C: 0.51

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	141.18	22.02	22.02	0.00	**0.00**
10	104.19	16.25	16.25	0.00	**0.00**
15	83.56	13.03	13.03	0.00	**0.00**
20	70.25	10.96	10.96	0.00	**0.00**
25	60.90	9.50	9.50	0.00	**0.00**
30	53.93	8.41	8.41	0.00	**0.00**
35	48.52	7.57	7.57	0.00	**0.00**
40	44.18	6.89	6.89	0.00	**0.00**
45	40.63	6.34	6.34	0.00	**0.00**
50	37.65	5.87	5.87	0.00	**0.00**
55	35.12	5.48	5.48	0.00	**0.00**
60	32.94	5.14	5.14	0.00	**0.00**

Project #160401329, 851 RICHMOND ROAD
Modified Rational Method Calculatons for Storage

100 yr Intensity I = a/(t + b)^c
City of Ottawa

		t (min)	I (mm/hr)
a =	1735.688	5	242.70
b =	6.014	10	178.56
c =	0.820	15	142.89
		20	119.95
		25	103.85
		30	91.87
		35	82.58
		40	75.15
		45	69.05
		50	63.95
		55	59.62
		60	55.89

100 YEAR Modified Rational Method for Entire Site

Subdrainage Area: BLDG — Roof
Area (ha): 0.11 — 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)
10	178.56	54.60	7.46	47.15	28.29
20	119.95	36.68	7.46	29.23	35.07
30	91.87	28.09	7.46	20.64	37.15
40	75.15	22.98	7.46	15.52	**37.26**
50	63.95	19.56	7.46	12.10	36.31
60	55.89	17.09	7.46	9.64	34.69
70	49.79	15.23	7.46	7.77	32.63
80	44.99	13.76	7.46	6.30	30.25
90	41.11	12.57	7.46	5.12	27.63
100	37.90	11.59	7.46	4.14	24.81
110	35.20	10.76	7.46	3.31	21.84
120	32.89	10.06	7.46	2.60	18.75

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	147.71	0.15	7.46	37.26	38.75	0.00

Subdrainage Area: L201A — Uncontrolled - Tributary
Area (ha): 0.04
C: 0.84

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	16.63	15.12	1.51	**0.90**
20	119.95	11.17	11.17	0.00	0.00
30	91.87	8.56	8.56	0.00	0.00
40	75.15	7.00	7.00	0.00	0.00
50	63.95	5.96	5.96	0.00	0.00
60	55.89	5.21	5.21	0.00	0.00
70	49.79	4.64	4.64	0.00	0.00
80	44.99	4.19	4.19	0.00	0.00
90	41.11	3.83	3.83	0.00	0.00
100	37.90	3.53	3.53	0.00	0.00
110	35.20	3.28	3.28	0.00	0.00
120	32.89	3.06	3.06	0.00	0.00

Subdrainage Area: L202A — Uncontrolled - Tributary
Area (ha): 0.11
C: 0.64

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	34.81	34.81	0.00	0.00
20	119.95	23.38	23.38	0.00	0.00
30	91.87	17.91	17.91	0.00	0.00
40	75.15	14.65	14.65	0.00	0.00
50	63.95	12.47	12.47	0.00	0.00
60	55.89	10.90	10.90	0.00	0.00
70	49.79	9.71	9.71	0.00	0.00
80	44.99	8.77	8.77	0.00	0.00
90	41.11	8.01	8.01	0.00	0.00
100	37.90	7.39	7.39	0.00	0.00
110	35.20	6.86	6.86	0.00	0.00
120	32.89	6.41	6.41	0.00	0.00

Stormwater Management Calculations

Project #160401329, 851 RICHMOND ROAD
Modified Rational Method Calculatons for Storage

Subdrainage Area: RAMP — Unontrolled - Tributary
Area (ha): 0.01
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	141.18	3.53	3.53		
10	104.19	2.61	2.61		
15	83.56	2.09	2.09		
20	70.25	1.76	1.76		
25	60.90	1.52	1.52		
30	53.93	1.35	1.35		
35	48.52	1.21	1.21		
40	44.18	1.11	1.11		
45	40.63	1.02	1.02		
50	37.65	0.94	0.94		
55	35.12	0.88	0.88		
60	32.94	0.82	0.82		

Subdrainage Area: Site Area Tributary to Internal Cistern
Area (ha): 0.27
C: 0.71

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	141.18	41.71	**29.69**	12.02	3.61
10	104.19	32.27	29.69	2.57	1.54
15	83.56	26.99	26.99	0.00	**0.00**
20	70.25	23.59	23.59	0.00	**0.00**
25	60.90	21.20	21.20	0.00	**0.00**
30	53.93	19.42	19.42	0.00	**0.00**
35	48.52	18.04	18.04	0.00	**0.00**
40	44.18	16.93	16.93	0.00	**0.00**
45	40.63	16.03	16.03	0.00	**0.00**
50	37.65	15.27	15.27	0.00	**0.00**
55	35.12	14.62	14.62	0.00	**0.00**
60	32.94	14.06	14.06	0.00	**0.00**

Subdrainage Area: UNC-1 — Uncontrolled - Non-Tributary
Area (ha): 0.01
C: 0.68

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	141.18	2.67	2.67		
10	104.19	1.97	1.97		
15	83.56	1.58	1.58		
20	70.25	1.33	1.33		
25	60.90	1.15	1.15		
30	53.93	1.02	1.02		
35	48.52	0.92	0.92		
40	44.18	0.84	0.84		
45	40.63	0.77	0.77		
50	37.65	0.71	0.71		
55	35.12	0.66	0.66		
60	32.94	0.62	0.62		

Subdrainage Area: UNC-2 — Uncontrolled - Non-Tributary
Area (ha): 0.03
C: 0.59

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
5	141.18	6.95	6.95		
10	104.19	5.13	5.13		
15	83.56	4.11	4.11		
20	70.25	3.46	3.46		
25	60.90	3.00	3.00		
30	53.93	2.65	2.65		
35	48.52	2.39	2.39		
40	44.18	2.17	2.17		
45	40.63	2.00	2.00		
50	37.65	1.85	1.85		
55	35.12	1.73	1.73		
60	32.94	1.62	1.62		

SUMMARY TO OUTLET

		Vrequired	Vavailable*	
Tributary Area	0.27 ha			
Total 5yr Flow to Sewer	29.69 L/s	20	58 m^3	Ok
Non-Tributary Area	0.04 ha			
Total 5yr Flow Uncontrolled	7.10 L/s			
Total Area	0.31 ha			
Total 5yr Flow	36.79 L/s			
Target	44.90 L/s			

Project #160401329, 851 RICHMOND ROAD
Modified Rational Method Calculatons for Storage

Subdrainage Area: RAMP — Unontrolled - Tributary
Area (ha): 0.01
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	4.96	4.96		
20	119.95	3.33	3.33		
30	91.87	2.55	2.55		
40	75.15	2.09	2.09		
50	63.95	1.78	1.78		
60	55.89	1.55	1.55		
70	49.79	1.38	1.38		
80	44.99	1.25	1.25		
90	41.11	1.14	1.14		
100	37.90	1.05	1.05		
110	35.20	0.98	0.98		
120	32.89	0.91	0.91		

Subdrainage Area: Site Area Tributary to Internal Cistern
Area (ha): 0.27
C: 0.88

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	62.35	**29.69**	32.66	**19.59**
20	119.95	45.35	29.69	15.65	18.78
30	91.87	36.47	29.69	6.78	12.20
40	75.15	31.19	29.69	1.50	3.59
50	63.95	27.66	27.66	0.00	0.00
60	55.89	25.11	25.11	0.00	0.00
70	49.79	23.18	23.18	0.00	0.00
80	44.99	21.67	21.67	0.00	0.00
90	41.11	20.44	20.44	0.00	0.00
100	37.90	19.43	19.43	0.00	0.00
110	35.20	18.58	18.58	0.00	0.00
120	32.89	17.85	17.85	0.00	0.00

Subdrainage Area: UNC-1 — Uncontrolled - Non-Tributary
Area (ha): 0.01
C: 0.85

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	4.22	4.22		
20	119.95	2.83	2.83		
30	91.87	2.17	2.17		
40	75.15	1.78	1.78		
50	63.95	1.51	1.51		
60	55.89	1.32	1.32		
70	49.79	1.18	1.18		
80	44.99	1.06	1.06		
90	41.11	0.97	0.97		
100	37.90	0.90	0.90		
110	35.20	0.83	0.83		
120	32.89	0.78	0.78		

Subdrainage Area: UNC-2 — Uncontrolled - Non-Tributary
Area (ha): 0.03
C: 0.74

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.56	10.98	10.98		
20	119.95	7.38	7.38		
30	91.87	5.65	5.65		
40	75.15	4.62	4.62		
50	63.95	3.93	3.93		
60	55.89	3.44	3.44		
70	49.79	3.06	3.06		
80	44.99	2.77	2.77		
90	41.11	2.53	2.53		
100	37.90	2.33	2.33		
110	35.20	2.17	2.17		
120	32.89	2.02	2.02		

SUMMARY TO OUTLET

		Vrequired	Vavailable*	
Tributary Area	0.27 ha			
Total 100yr Flow to Sewer	29.69 L/s	57	58 m^3	Ok
Non-Tributary Area	0.04 ha			
Total 100yr Flow Uncontrolled	15.20 L/s			
Total Area	0.31 ha			
Total 100yr Flow	44.90 L/s			
Target	44.90 L/s			

851 RICHMOND ROAD
Roof Drain Design Sheet, Area BLDG
Standard Watts Model R1100 Accuflow Roof Drain

Rating Curve				Volume Estimation				Water Depth
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	(m)
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0013	0	0.025	22	0	0	0.025
0.050	0.0006	0.0025	1	0.050	86	1	1	0.050
0.075	0.0009	0.0038	5	0.075	194	3	5	0.075
0.100	0.0013	0.0050	11	0.100	344	7	11	0.100
0.125	0.0016	0.0063	22	0.125	538	11	22	0.125
0.150	0.0019	0.0076	39	0.150	775	16	39	0.150

Rooftop Storage Summary

Total Building Area (sq.m)		1100
Assume Available Roof Area (sq.m)	70%	775
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		4
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		39
Estimated 100 Year Drawdown Time (h)		1.8

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

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

Calculation Results

	5yr	100yr	Available
Qresult (cu.m/s)	0.006	0.007	-
Depth (m)	0.112	0.148	0.150
Volume (cu.m)	16.7	37.3	38.8
Draintime (hrs)	1.0	1.8	

Stormceptor Sizing Detailed Report

PCSWMM for Stormceptor

Project Information

Date	10/4/2017
Project Name	851 Richmond Road
Project Number	160401329
Location	Ottawa, ON

Stormwater Quality Objective

This report outlines how Stormceptor System can achieve a defined water quality objective through the removal of total suspended solids (TSS). Attached to this report is the Stormceptor Sizing Summary.

Stormceptor System Recommendation

The Stormceptor System model STC 300 achieves the water quality objective removing 88% TSS for a Fine (organics, silts and sand) particle size distribution.

The Stormceptor System

The Stormceptor oil and sediment separator is sized to treat stormwater runoff by removing pollutants through gravity separation and flotation. Stormceptor's patented design generates positive TSS removal for all rainfall events, including large storms. Significant levels of pollutants such as heavy metals, free oils and nutrients are prevented from entering natural water resources and the re-suspension of previously captured sediment (scour) does not occur.

Stormceptor provides a high level of TSS removal for small frequent storm events that represent the majority of annual rainfall volume and pollutant load. Positive treatment continues for large infrequent events, however, such events have little impact on the average annual TSS removal as they represent a small percentage of the total runoff volume and pollutant load.

Stormceptor is the only oil and sediment separator on the market sized to remove TSS for a wide range of particle sizes, including fine sediments (clays and silts), that are often overlooked in the design of other stormwater treatment devices.

Small storms dominate hydrologic activity, US EPA reports

"*Early efforts in stormwater management focused on flood events ranging from the 2-yr to the 100-yr storm. Increasingly stormwater professionals have come to realize that small storms (i.e. < 1 in. rainfall) dominate watershed hydrologic parameters typically associated with water quality management issues and BMP design. These small storms are responsible for most annual urban runoff and groundwater recharge. Likewise, with the exception of eroded sediment, they are responsible for most pollutant washoff from urban surfaces. Therefore, the small storms are of most concern for the stormwater management objectives of ground water recharge, water quality resource protection and thermal impacts control.*"

"*Most rainfall events are much smaller than design storms used for urban drainage models. In any given area, most frequently recurrent rainfall events are small (less than 1 in. of daily rainfall).*"

"Continuous simulation offers possibilities for designing and managing BMPs on an individual site-by-site basis that are not provided by other widely used simpler analysis methods. Therefore its application and use should be encouraged."

– US EPA Stormwater Best Management Practice Design Guide, Volume 1 – General Considerations, 2004

Design Methodology

Each Stormceptor system is sized using PCSWMM for Stormceptor, a continuous simulation model based on US EPA SWMM. The program calculates hydrology from up-to-date local historical rainfall data and specified site parameters. With US EPA SWMM's precision, every Stormceptor unit is designed to achieve a defined water quality objective.

The TSS removal data presented follows US EPA guidelines to reduce the average annual TSS load. Stormceptor's unit process for TSS removal is settling. The settling model calculates TSS removal by analyzing (summary of analysis presented in Appendix 2):

- Site parameters
- Continuous historical rainfall, including duration, distribution, peaks (Figure 1)
- Interevent periods
- Particle size distribution
- Particle settling velocities (Stokes Law, corrected for drag)
- TSS load (Figure 2)
- Detention time of the system

The Stormceptor System maintains continuous positive TSS removal for all influent flow rates. Figure 3 illustrates the continuous treatment by Stormceptor throughout the full range of storm events analyzed. It is clear that large events do not significantly impact the average annual TSS removal. There is no decline in cumulative TSS removal, indicating scour does not occur as the flow rate increases.

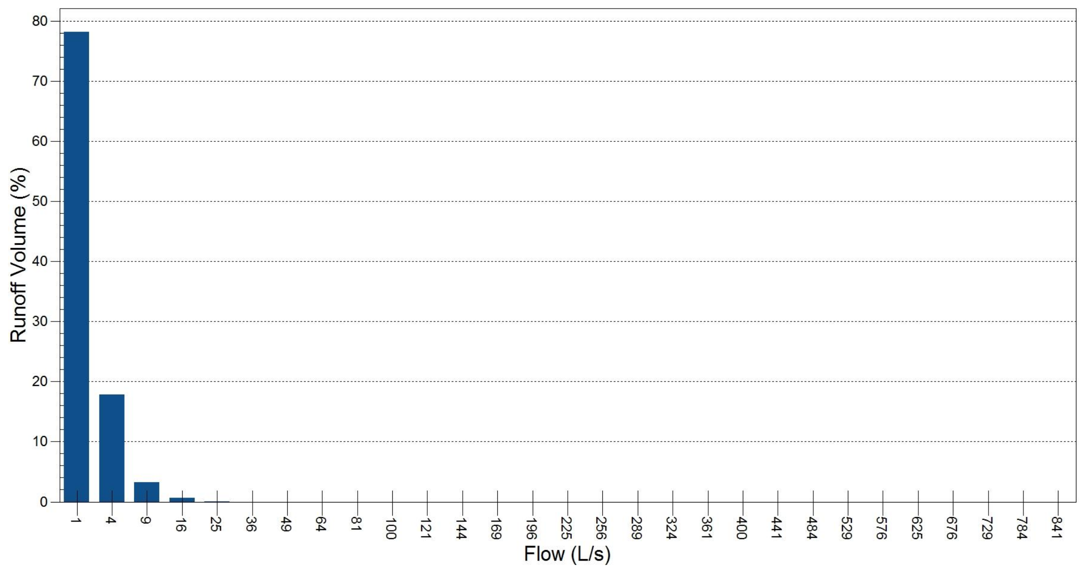


Figure 1. Runoff Volume by Flow Rate for OTTAWA MACDONALD-CARTIER INT'L A – ON 6000, 1967 to 2003 for 0.16 ha, 53% impervious. Small frequent storm events represent the majority of annual rainfall volume. Large infrequent events have little impact on the average annual TSS removal, as they represent a small percentage of the total annual volume of runoff.

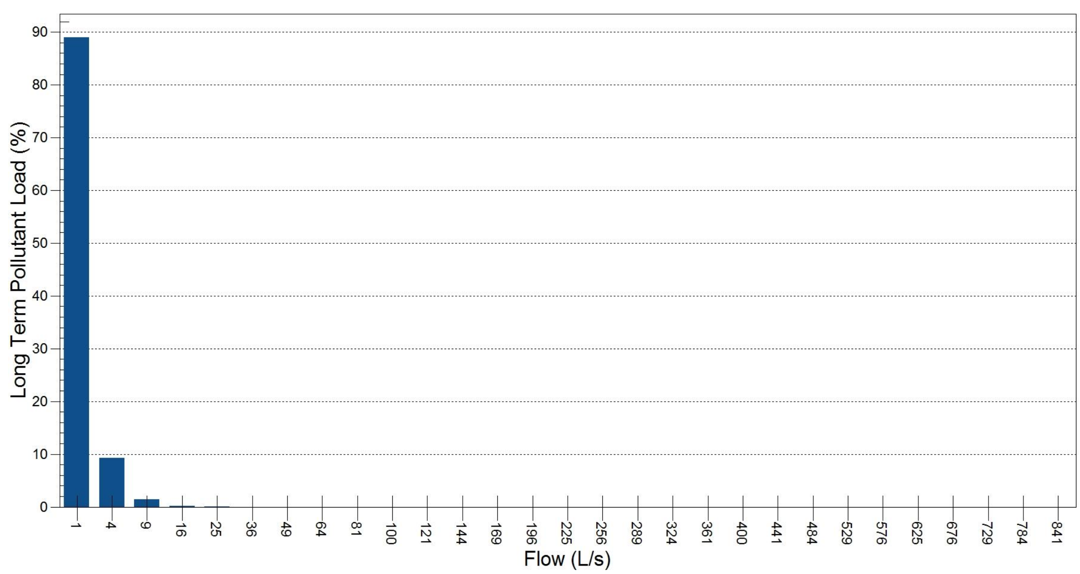
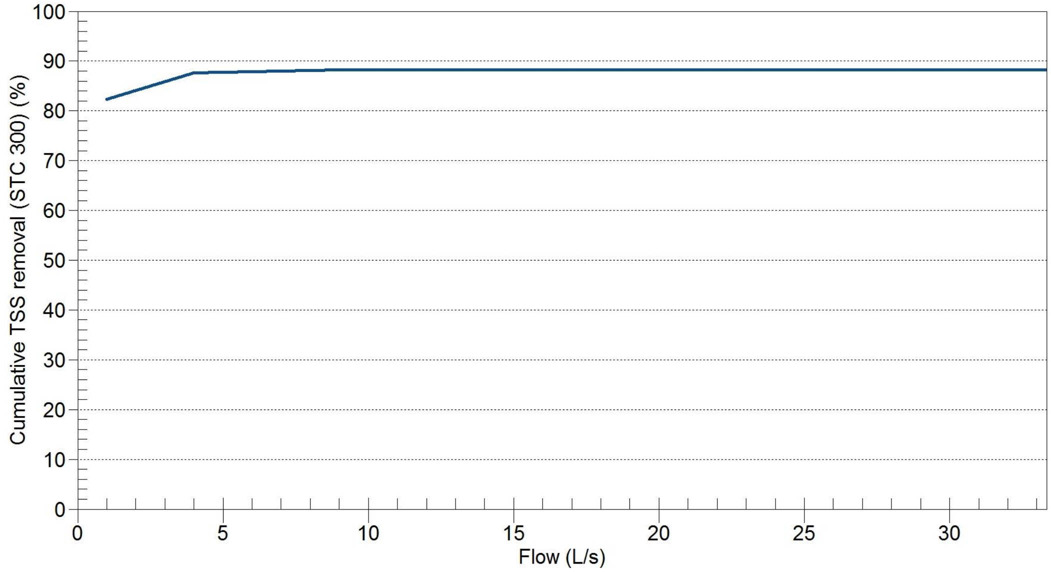


Figure 2. Long Term Pollutant Load by Flow Rate for OTTAWA MACDONALD-CARTIER INT'L A – 6000, 1967 to 2003 for 0.16 ha, 53% impervious. The majority of the annual pollutant load is transported by small frequent storm events. Conversely, large infrequent events carry an insignificant percentage of the total annual pollutant load.



Stormceptor Model	STC 300	Drainage Area (ha)	0.16
TSS Removal (%)	88	Impervious (%)	53

Figure 3. Cumulative TSS Removal by Flow Rate for OTTAWA MACDONALD-CARTIER INT'L A – 6000, 1967 to 2003. Stormceptor continuously removes TSS throughout the full range of storm events analyzed. Note that large events do not significantly impact the average annual TSS removal. Therefore no decline in cumulative TSS removal indicates scour does not occur as the flow rate increases.

Stormceptor®

Appendix 1
Stormceptor Design Summary

Project Information

Date	10/4/2017
Project Name	851 Richmond Road
Project Number	160401329
Location	Ottawa, ON

Designer Information

Company	Stantec Consulting Ltd.
Contact	Ana M. Paerez

Notes

N/A

Drainage Area

Total Area (ha)	0.16
Imperviousness (%)	53

The Stormceptor System model STC 300 achieves the water quality objective removing 88% TSS for a Fine (organics, silts and sand) particle size distribution.

Rainfall

Name	OTTAWA MACDONALD-CARTIER INT'L A
State	ON
ID	6000
Years of Records	1967 to 2003
Latitude	45°19'N
Longitude	75°40'W

Water Quality Objective

TSS Removal (%)	80

Upstream Storage

Storage (ha-m)	Discharge (L/s)
0	0

Stormceptor Sizing Summary

Stormceptor Model	TSS Removal %
STC 300	**88**
STC 750	92
STC 1000	93
STC 1500	93
STC 2000	95
STC 3000	96
STC 4000	97
STC 5000	97
STC 6000	98
STC 9000	98
STC 10000	98
STC 14000	99

Stormceptor®

Particle Size Distribution

Removing silt particles from runoff ensures that the majority of the pollutants, such as hydrocarbons and heavy metals that adhere to fine particles, are not discharged into our natural water courses. The table below lists the particle size distribution used to define the annual TSS removal.

Fine (organics, silts and sand)								
Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s		Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
20	20	1.3	0.0004					
60	20	1.8	0.0016					
150	20	2.2	0.0108					
400	20	2.65	0.0647					
2000	20	2.65	0.2870					

Stormceptor Design Notes

- Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor version 1.0
- Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal.
- Only the STC 300 is adaptable to function with a catch basin inlet and/or inline pipes.
- Only the Stormceptor models STC 750 to STC 6000 may accommodate multiple inlet pipes.
- Inlet and outlet invert elevation differences are as follows:

Inlet and Outlet Pipe Invert Elevations Differences

Inlet Pipe Configuration	STC 300	STC 750 to STC 6000	STC 9000 to STC 14000
Single inlet pipe	75 mm	25 mm	75 mm
Multiple inlet pipes	75 mm	75 mm	Only one inlet pipe.

- Design estimates are based on stable site conditions only, after construction is completed.
- Design estimates assume that the storm drain is not submerged during zero flows. For submerged applications, please contact your local Stormceptor representative.
- Design estimates may be modified for specific spills controls. Please contact your local Stormceptor representative for further assistance.
- For pricing inquiries or assistance, please contact Imbrium Systems Inc., 1-800-565-4801.

Stormceptor®

Appendix 2
Summary of Design Assumptions

SITE DETAILS

Site Drainage Area

Total Area (ha)	0.16	Imperviousness (%)	53

Surface Characteristics

Width (m)	80
Slope (%)	2
Impervious Depression Storage (mm)	0.508
Pervious Depression Storage (mm)	5.08
Impervious Manning's n	0.015
Pervious Manning's n	0.25

Infiltration Parameters

Horton's equation is used to estimate infiltration

Max. Infiltration Rate (mm/h)	61.98
Min. Infiltration Rate (mm/h)	10.16
Decay Rate (s^{-1})	0.00055
Regeneration Rate (s^{-1})	0.01

Maintenance Frequency

Sediment build-up reduces the storage volume for sedimentation. Frequency of maintenance is assumed for TSS removal calculations.

Maintenance Frequency (months)	12

Evaporation

Daily Evaporation Rate (mm/day)	2.54

Dry Weather Flow

Dry Weather Flow (L/s)	No

Upstream Attenuation

Stage-storage and stage-discharge relationship used to model attenuation upstream of the Stormceptor System is identified in the table below.

Storage ha-m	Discharge L/s
0	0

PARTICLE SIZE DISTRIBUTION

Particle Size Distribution

Removing fine particles from runoff ensures the majority of pollutants, such as heavy metals, hydrocarbons, free oils and nutrients are not discharged into natural water resources. The table below identifies the particle size distribution selected to define TSS removal for the design of the Stormceptor System.

Fine (organics, silts and sand)								
Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s		Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
20	20	1.3	0.0004					
60	20	1.8	0.0016					
150	20	2.2	0.0108					
400	20	2.65	0.0647					
2000	20	2.65	0.2870					

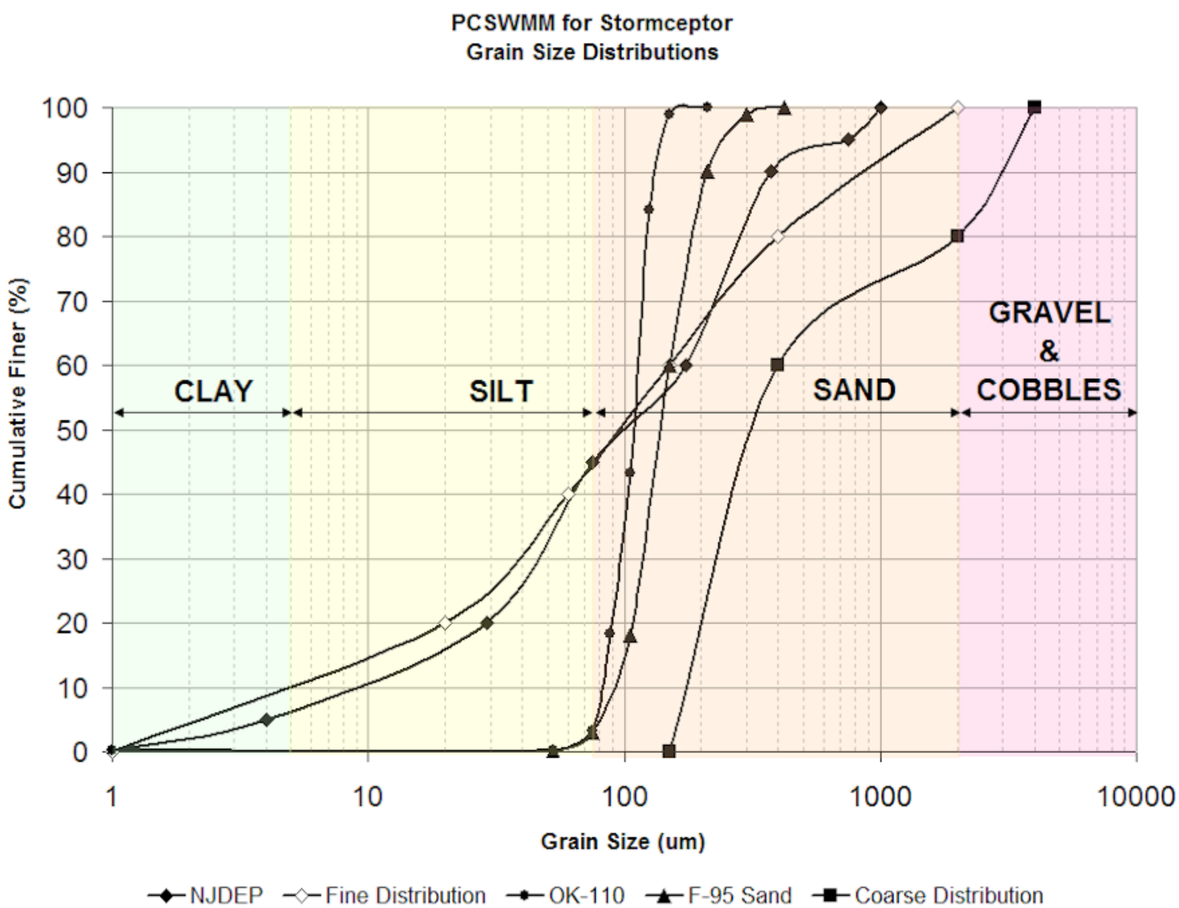


Figure 1. PCSWMM for Stormceptor standard design grain size distributions.

TSS LOADING

TSS Loading Parameters

TSS Loading Function	Buildup / Washoff

Parameters

Target Event Mean Concentration (EMC) (mg/L)	125
Exponential Buildup Power	0.4
Exponential Washoff Exponential	0.2

HYDROLOGY ANALYSIS

PCSWMM for Stormceptor calculates annual hydrology with the US EPA SWMM and local continuous historical rainfall data. Performance calculations of the Stormceptor System are based on the average annual removal of TSS for the selected site parameters. The Stormceptor System is engineered to capture fine particles (silts and sands) by focusing on average annual runoff volume ensuring positive removal efficiency is maintained during all rainfall events, while preventing the opportunity for negative removal efficiency (scour).

Smaller recurring storms account for the majority of rainfall events and average annual runoff volume, as observed in the historical rainfall data analyses presented in this section.

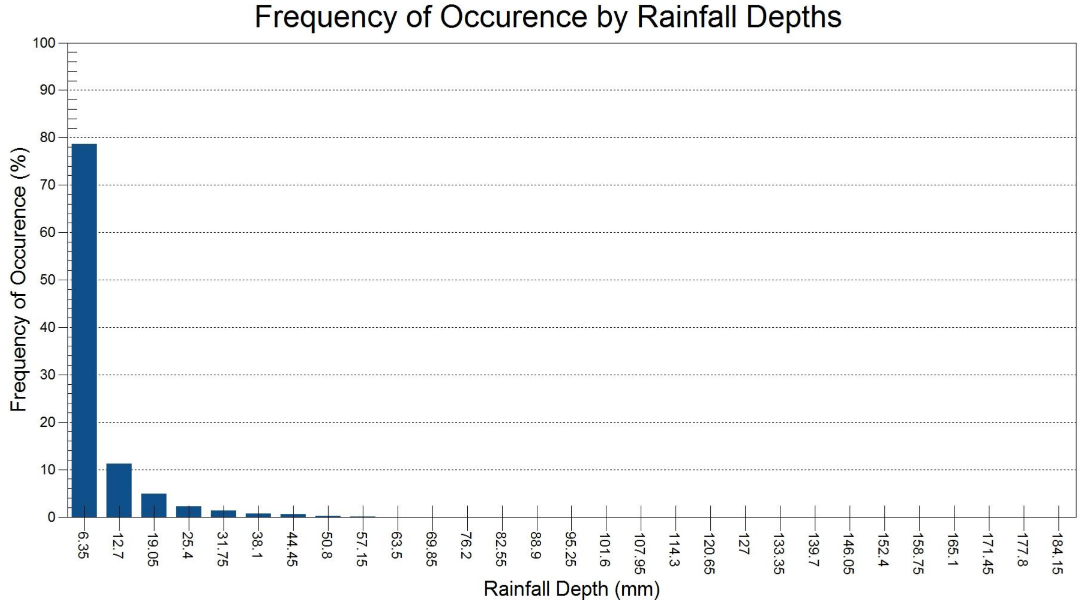
Rainfall Station

Rainfall Station	OTTAWA MACDONALD-CARTIER INT'L A		
Rainfall File Name	ON6000.NDC	Total Number of Events	4537
Latitude	45°19'N	Total Rainfall (mm)	20978.1
Longitude	75°40'W	Average Annual Rainfall (mm)	567.0
Elevation (m)	371	Total Evaporation (mm)	951.7
Rainfall Period of Record (y)	37	Total Infiltration (mm)	9831.7
Total Rainfall Period (y)	37	Percentage of Rainfall that is Runoff (%)	49.1

Rainfall Event Analysis

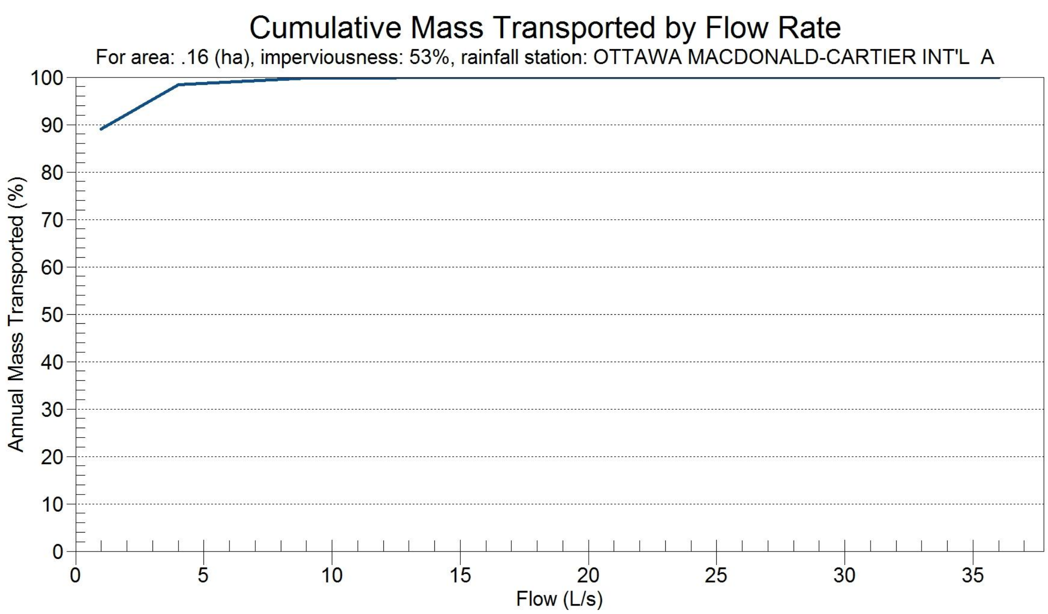
Rainfall Depth mm	No. of Events	Percentage of Total Events %	Total Volume mm	Percentage of Annual Volume %
6.35	3564	78.6	5671	27.0
12.70	508	11.2	4533	21.6
19.05	223	4.9	3434	16.4
25.40	102	2.2	2244	10.7
31.75	60	1.3	1704	8.1
38.10	33	0.7	1145	5.5
44.45	28	0.6	1165	5.6
50.80	9	0.2	416	2.0
57.15	5	0.1	272	1.3
63.50	1	0.0	63	0.3
69.85	1	0.0	64	0.3
76.20	1	0.0	76	0.4
82.55	0	0.0	0	0.0
88.90	1	0.0	84	0.4
95.25	0	0.0	0	0.0
101.60	0	0.0	0	0.0
107.95	0	0.0	0	0.0
114.30	1	0.0	109	0.5
120.65	0	0.0	0	0.0
127.00	0	0.0	0	0.0
133.35	0	0.0	0	0.0
139.70	0	0.0	0	0.0
146.05	0	0.0	0	0.0
152.40	0	0.0	0	0.0
158.75	0	0.0	0	0.0
165.10	0	0.0	0	0.0
171.45	0	0.0	0	0.0
177.80	0	0.0	0	0.0
184.15	0	0.0	0	0.0
190.50	0	0.0	0	0.0
196.85	0	0.0	0	0.0
203.20	0	0.0	0	0.0
209.55	0	0.0	0	0.0
>209.55	0	0.0	0	0.0

Frequency of Occurence by Rainfall Depths



Pollutograph

Flow Rate L/s	Cumulative Mass %
1	89.1
4	98.4
9	99.8
16	100.0
25	100.0
36	100.0
49	100.0
64	100.0
81	100.0
100	100.0
121	100.0
144	100.0
169	100.0
196	100.0
225	100.0
256	100.0
289	100.0
324	100.0
361	100.0
400	100.0
441	100.0
484	100.0
529	100.0
576	100.0
625	100.0
676	100.0
729	100.0
784	100.0
841	100.0
900	100.0



Appendix E GEOTECHNICAL REPORT

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Archaeological Services

patersongroup

Geotechnical Investigation

Proposed Multi-Storey Building
851 Richmond Road - Ottawa

Prepared For

Homestead Land Holdings Ltd.

October 3, 2017

Report: PG4163-1 Revision 1

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

Table of Contents

Page

Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Homestead Land Holdings Ltd. (Homestead) to conduct a geotechnical investigation for the proposed multi-storey building to be located at 851 Richmond Road in the City of Ottawa (refer to Figure 1 - Key Plan presented in Appendix 2).

The objective of the investigation was to:

❏ Determine the subsoil and groundwater conditions at this site by means of boreholes.

❏ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect its design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation. A report addressing environmental issues for the subject site was prepared under separate cover.

2.0 Proposed Project

It is our understanding that the proposed project consists of a multi-storey building with two underground parking levels encompassing the majority of the subject site.

3.0 Method of Investigation

3.1 Field Investigation

The field program for our geotechnical investigation was carried out on June 1, 2017. At that time, a total of six (6) boreholes were advanced to a maximum depth of 7.0 m. The borehole locations were determined in the field by Paterson personnel taking into consideration site features and underground services. The locations of the boreholes are shown on Drawing PG4163-1 - Test Hole Location Plan included in Appendix 2.

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

Sampling and In Situ Testing

Soil samples were collected 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 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.

The 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.

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

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

Groundwater

Monitoring wells and flexible standpipes were installed in the boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program.

Sample Storage

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

3.2 Field Survey

The borehole locations were determined by Paterson personnel taking into consideration the presence of underground and aboveground services. The location and ground surface elevation at each borehole location was surveyed by Paterson personnel. The ground surface elevation at the borehole locations were surveyed with respect to a temporary benchmark (TBM), consisting of the top of catch basin located within the northeast corner the existing site. A geodetic elevation of 65.24 m was provided for the TBM by Homestead. The borehole locations and ground surface elevation at each borehole location are presented on Drawing PG4163-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples and rock cores recovered from the subject site were examined in our laboratory to review the results of the field logging.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by at-grade parking for the adjacent multi-storey residential building to the west. The site is bordered to the north by an easement, which contains a large diameter watermain, followed by residential buildings, to the south by Richmond Road and to the east by at grade parking area. The ground surface across the site is relatively flat and at grade with the neighbouring properties.

4.2 Subsurface Profile

Generally, the subsurface profile encountered at the borehole locations consists of 60 to 100 mm thickness of asphalt overlying a granular layer, consisting of crushed stone with silt and sand with maximum thickness of 230 mm. The pavement structure lies atop a fill layer, consisting of loose to compact, brown to grey sand and gravel with trace to some silt and clay which extends to a depth of approximately 1.5 to 2.5 m. A native glacial till deposit was encountered underlying the abovenoted fill layers followed by a grey limestone bedrock. Generally, the bedrock quality consists of poor quality within the upper 0.5 to 1 m and fair to excellent quality at depth based on the RQD values. The upper portion of the bedrock was noted to consist of a weathered, poor quality bedrock. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

Based on available geological mapping, the bedrock in this area mostly consists of limestone with some shaly partings of the Ottawa formation with an overburden drift thickness of less than 5 m depth.

4.3 Groundwater

The measured groundwater levels in the monitoring wells and piezometers at the borehole locations are presented in Table 1. It should be further noted that the groundwater level could vary at the time of construction.

Table 1 - Summary of Groundwater Level Readings

Test Hole Number	Ground Elevation (m)	Groundwater Levels (m)		Recording Date
		Depth	Elevation	
BH 1	66.03	2.93	63.10	June 8, 2017
BH 2	65.69	2.31	63.38	June 8, 2017
BH 3	65.44	3.72	61.72	June 8, 2017
BH 4	66.05	2.19	63.86	June 8, 2017
BH 5	65.79	3.20	62.59	June 8, 2017
BH 6	65.56	3.35	62.21	June 8, 2017

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey building. The proposed building is expected to be founded on conventional footings placed on clean, surface sounded bedrock.

Bedrock removal will be required to complete the two (2) levels of underground parking. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

An alignment of a large diameter watermain runs within an easement along the north property boundary of the subject site. It is expected that the adjacent watermain could be subjected to potential vibrations associated with the bedrock blasting program. To ensure that no detrimental vibrations cause damage to the adjacent watermain, a vibration attenuation trench is recommended for the bedrock along the north excavation face, as well as a vibration monitoring and control program during the blasting and excavation work required for the proposed building excavation.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage levels.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

As a general guideline, peak particle velocity (measured at the structures) should not exceed 25 mm/s during the blasting program to reduce the risks of damage to the existing structures.

The blasting operations should be planned and conducted under the supervision of a licensed professional engineer who is an experienced blasting consultant.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

A vibration attenuation trench is recommended to be completed within the bedrock along the north property boundary. The construction of the vibration attenuation trench would require line drilling in a tight pattern on both sides of the proposed 1 m wide trench alignment and within the interior portion of the trench to the design underside of footing elevation. A hoe ram operation would be used to break up the bedrock and remove it from the trench. It is expected that the coreholes for the bedrock blasting program may not be possible within 1 to 2 m of the attenuation trench due to the presence of the drilled holes within the attenuation trench, which can cause an energy loss and blow-out during blasting if connected to the blast source by potential fractures within the bedrock. Therefore, a hoe ramming operation will most likely be required to complete the bedrock removal within the area adjacent to the attenuation trench.

Vibration Considerations

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

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require these pieces of equipments. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

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

Vibration Monitoring and Control Plan

To ensure that no disturbance to the existing watermain occurs, a vibration monitoring and control plan (VMCP) is recommended during the excavation program. The purpose of the vibration monitoring and control plan is to provide measures to be implemented by the contractor to manage excavation operations and any other vibration sources during the construction for the proposed development. The VMCP will also provide a guideline for assessing results against the relevant vibration impact assessment criteria and recommendations to meet the required limits.

The monitoring program will incorporate real time results at the existing watermain segment adjacent to the subject site. The monitoring equipment should consist of a tri-axial seismograph, capable of measuring vibration intensities up to 254 mm/s at a frequency response of 2 to 250 Hz. At least two vibration monitoring devices should be placed adjacent to the existing watermain. It is recommended that the vibration monitoring devices be installed at invert level of the existing watermain and periodically inspected during the construction program.

A copy of the geotechnical report, which includes the VMCP should be provided to all parties involved with the construction for review. A meeting between Paterson and site contractor should be conducted prior to any excavation or construction of the subject site to review the following:

- ❑ Review the pre-condition/pre-construction survey;
- ❑ Control measures (i.e vibrations, noise);
- ❑ Monitoring locations;
- ❑ Tracking and reporting of excavation progress, and;
- ❑ Review procedure for exceedances (i.e vibrations, noise), complaints, evaluation and corrective measures.

When an event is triggered, Paterson will review the results and provide any necessary feedback. Otherwise, the vibration results will be summarized in the weekly report. The following table outlines the vibration limits for the adjacent watermain segment.

Table 2 - Structure Vibration Limits for adjacent Watermain Segment

Dominant Frequency Range (Hz)	Peak Particle Velocity (mm/s)	Event	Description of Event
<10	all	none	no action required
<40	>10	trigger level	Warning e-mail sent to contractor.
<40	≥15	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.
>40	>15	trigger level	Warning e-mail sent to contractor.
>40	≥20	exceedance level	Exceedance e-mail and phone call to the contractor. All operations are ceased to review on-site activities.

The monitoring protocol should include the following information:

Trigger Level Event

- ❏ Paterson will review all vibrations over the established warning level, and;
- ❏ Paterson will notify the contractor if any vibration occur due to construction activities and are close to exceedance level.

Exceedance Level Event

- ❏ Paterson will notify all the relevant stakeholders via email;
- ❏ Ensure monitors are functioning, and;
- ❏ Issue the vibration exceedance result.

Fill Placement

Fill used for grading beneath the building areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the proposed building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa** incorporating a geotechnical resistance factor of 0.5.

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

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Settlement

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

5.4 Design for Earthquakes

A site specific shear wave velocity test was completed by Paterson to accurately determine the applicable seismic site classification for foundation design of the proposed building as presented in Table 4.1.8.4.A of the Ontario Building Code 2012. Two (2) shear wave velocity profiles from our on-site testing are presented in Appendix 2.

Field Program

The location of the seismic array was chosen to provide adequate coverage of the area. The seismic array testing location is presented in Drawing PG4163-1 - Test Hole Location Plan in Appendix 2.

At the seismic array location, Paterson field personnel placed 18 horizontal 4.5 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 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 five to ten times at each shot location to improve signal to noise ratio. The shot locations are also completed in forward and reverse directions (i.e.- striking both sides of the I-Beam seated parallel to the geophone array). The shot locations are located at 3,4.5 and 13.5 m away from the first, 3, 4.5, and 14 m away from the last geophone, and at the center of the seismic array.

The methods of testing completed by Paterson are guided by the standard testing procedures used by the expert seismologists at Carleton University and Geological Survey of Canada (GSC).

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, Vs_{30}, of the upper 30 m profile, immediately below the building's foundation.

Based on the test results, the average overburden seismic shear wave velocity is 248 m/s. Through interpretation, the bedrock has a shear wave velocity of 2,256 m/s. The Vs_{30} was calculated using the standard equation for average shear wave velocity from the Ontario Building Code (OBC) 2012.

The Vs_{30} was calculated using the standard equation for average shear wave velocity calculation from the Ontario Building Code (OBC) 2012, as presented below.

$$V_{s30} = \frac{Depth_{OfInterest}(m)}{\sum\left(\frac{(Depth_i(m)}{Vs_i(m/s)}\right)}$$

$$V_{s30} = \frac{30m}{\left(\frac{0.0m}{248m/s} + \frac{30.0m}{2{,}256m/s}\right)}$$

$$V_{s30} = 2{,}256m/s$$

Based on the results of the seismic testing, the average shear wave velocity, Vs_{30}, beneath the foundation is 2,256 m/s. Therefore, a **Site Class A** is applicable for 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

All overburden soil will be removed for the proposed building and the basement floor slab will be founded on a bedrock medium. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. It is recommended that the upper 200 mm of sub-slab fill consists of a 19 mm clear crushed stone.

In consideration of the groundwater conditions encountered during the investigation, a subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone backfill under the lower basement floor.

5.6 Basement Wall

It is expected that a portion of the basement walls are to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a dry unit weight of 23.5 kN/m^3 (effective unit weight of 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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

Two distinct conditions, static and seismic, should be reviewed for design calculations. The parameters for design calculations for the two conditions are presented below.

Static Conditions

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

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

An additional pressure with 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 Conditions

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}) could be calculated using $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

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

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

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

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

5.7 Pavement Structure

For design purposes, the pavement structure presented in the following tables could be used for the design of car parking areas and access lanes.

Table 3 - 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 - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Table 4 - Recommended Pavement Structure - Access Lanes

Thickness (mm)	Material Description
40	**Wear Course -** HL3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
400	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Either fill, in situ soil or OPSS Granular B Type I or II material placed over in situ soil or fill	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated to a competent layer and replaced with OPSS Granular B Type II material. Weak subgrade conditions may be experienced over service trench fill materials. This may require the use of a geotextile, such as Terratrack 200 or equivalent, thicker subbase or other measures that can be recommended at the time of construction as part of the field observation program.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment, noting that excessive compaction can result in subgrade softening.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structures. It is expected that insufficient room is available for exterior backfill. It is suggested that this system could be as follows:

- ❏ Bedrock vertical surface (Hoe ram any irregularities and prepare bedrock surface. Shotcrete areas to fill in cavities and smooth out angular features at the bedrock surface);
- ❏ composite drainage layer

It is recommended that the composite drainage system (such as Miradrain G100N, Delta Drain 6000 or equivalent) extend down to the footing level. It is recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration. For preliminary design purposes, we recommend that 100 or 150 mm in perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

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

6.2 Protection Against Frost Action

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

A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other exterior unheated footings.

6.3 Excavation Side Slopes

Unsupported Excavations

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. 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 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

The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer 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. Furthermore, the design of the temporary shoring system should take into consideration, a full hydrostatic condition which can occur during significant precipitation events.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, the shoring systems should be provided with tie-back rock anchors to ensure the stability. The shoring system is recommended to be adequately supported to resist toe failure, if required, by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is the preferred method.
The earth pressures acting on the shoring system may be calculated with the following parameters.

Table 5 - Soil Parameters

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

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

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

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

6.4 Pipe Bedding and Backfill

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

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Infiltration levels are anticipated to be low through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

A temporary MOE permit to take water (PTTW) will be required for this project if more than 50,000 L/day are to be pumped during the construction phase. A minimum of four to five months should be allocated for completion of the application and issuance of the permit by the MOE.

Long-term Groundwater Control

Our recommendations for the proposed building's long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building's perimeter or sub-slab drainage system will be directed to the proposed building's cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 50,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. It is anticipated that the groundwater flow will be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on our observations, a local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within native glacial till and/or directly over a bedrock bearing surface. No issues are expected with respect to groundwater lowering that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

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

Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, 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.

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

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

6.7 Corrosion Potential and Sulphate

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

7.0 Recommendations

It is recommended that the following be carried out once the master plan and site development are determined:

- ❏ Review master grading plan from a geotechnical perspective, once available.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- ❏ Observation of all subgrades prior to placement of backfilling materials.
- ❏ 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 request, following the completion of a satisfactory material testing and observation program by the geotechnical consultant.

8.0 Statement of Limitations

The recommendations made in this report are in accordance with our present understanding of the project. We request permission to review the grading plan once available. Also, our recommendations should be reviewed when the drawings and specifications are complete.

The client should be aware that any information pertaining to soils and all test hole logs are furnished as a matter of general information only and test hole descriptions or logs are not to be interpreted as descriptive of conditions at locations other than those of the test holes.

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, we request that we be notified immediately in order to permit reassessment of our recommendations.

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 Homestead Land Developments or their agent(s) is not authorized without review by this firm for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Oct. 11-2017
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Nathan Christie, P.Eng. David J. Gilbert, P.Eng.

Report Distribution:

- ❏ Homestead Land Holdings Ltd. (3 copies)
- ❏ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 851 Richmond Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (refer to Dwg. PG4163-1). Geodetic elevation = 65.24m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 1, 2017

FILE NO. PG4163

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.03
Asphaltic concrete	0.08						
FILL: Brown sand and gravel	0.23						
FILL: Brown sand and gravel, some silt	2.49	SS	1	42	21		
		SS	2	33	11	1	65.03
		SS	3	36	50+	2	64.03
Grey fractured limestone **BEDROCK**	3.02	SS	4	71	50+	3	63.03
BEDROCK: Fair to excellent quality, grey limestone	5.92	RC	1	85	69	4	62.03
		RC	2	100	100	5	61.03
End of Borehole							
(GWL @ 2.93m - June 8, 2017)							

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

○ Water Content %

20 40 60 80

Monitoring Well Construction

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

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 851 Richmond Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (refer to Dwg. PG4163-1). Geodetic elevation = 65.24m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 1, 2017

FILE NO. PG4163

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.69
Asphaltic concrete	0.10						
FILL: Grey-brown sand, some silt	2.29	SS	1	62	11		
		SS	2	25	10	1	64.69
		SS	3	42	5	2	63.69
Grey fractured limestone **BEDROCK**	2.44	SS	4	100	50+		

End of Borehole

Practical refusal to augering at 2.44m depth

(GWL @ 2.31m - June 8, 2017)

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
Prop. Multi-Storey Building - 851 Richmond Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (refer to Dwg. PG4163-1). Geodetic elevation = 65.24m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 1, 2017

FILE NO. PG4163

HOLE NO. BH 3

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	65.44
Asphaltic concrete	0.09						
FILL: Grey-brown sand, trace silt		SS	1	58	21		
		SS	2	33	35	1	64.44
		SS	3	67	18	2	63.44
	2.36	SS	4	88	50+		
GLACIAL TILL: Brown silty clay with sand, gravel, fractured rock and boulders		RC	1	94		3	62.44
		RC	2	67			
	3.99	SS	5	100	50+	4	61.44
BEDROCK: Poor to excellent quality, grey limestone		RC	3	80	60	5	60.44
		RC	4	100	96	6	59.44
	6.98						
End of Borehole							
(GWL @ 3.72m - June 8, 2017)							

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

○ Water Content %

20 40 60 80

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 851 Richmond Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (refer to Dwg. PG4163-1). Geodetic elevation = 65.24m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 1, 2017

FILE NO. PG4163

HOLE NO. BH 4

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.05
Asphaltic concrete 0.09							
FILL: Grey-brown sand, trace silt 0.76		SS	1	75	20		
FILL: Brown silty sand, some clay, trace gravel 1.52		SS	2	83	8	1	65.05
GLACIAL TILL: Brown sandy silt, trace clay and gravel		SS	3	75	24	2	64.05
2.39		SS	4	100	50+		
End of Borehole							
Practical refusal to augering at 2.39m depth							
(GWL @ 2.19m - June 8, 2017)							

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
Prop. Multi-Storey Building - 851 Richmond Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (refer to Dwg. PG4163-1). Geodetic elevation = 65.24m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 1, 2017

FILE NO. PG4163

HOLE NO. BH 5

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.06				
FILL: Brown sand with gravel, trace silt and clay	1.52	SS	1	46	57
		SS	2	42	11
GLACIAL TILL: Brown sand with gravel, trace silt	2.06	SS	3	67	39
BEDROCK: Very poor to fair quality, grey limestone	5.72	RC	1	81	21
		RC	2	64	40
		RC	3	100	100
End of Borehole					
(GWL @ 3.20m - June 8, 2017)					

DEPTH (m)	ELEV. (m)
0	65.79
1	64.79
2	63.79
3	62.79
4	61.79
5	60.79

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

○ Water Content %

20 40 60 80

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

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 851 Richmond Road
Ottawa, Ontario

DATUM TBM - Top of grate of catch basin (refer to Dwg. PG4163-1). Geodetic elevation = 65.24m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 1, 2017

FILE NO. PG4163

HOLE NO. BH 6

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	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	65.56		
Asphaltic concrete 0.08									
FILL: Brown sand and gravel, trace silt		SS	1	58	18				
		SS	2	50	45	1	64.56		
		SS	3	42	17				
2.29						2	63.56		
GLACIAL TILL: Brown silty sand with clay and gravel		SS	4	58	13				
						3	62.56		
		SS	5	100	27				
						4	61.56		
		SS	6	100	52				
4.60									

End of Borehole

Practical refusal to augering at 4.60m depth

(GWL @ 3.35m - June 8, 2017)

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

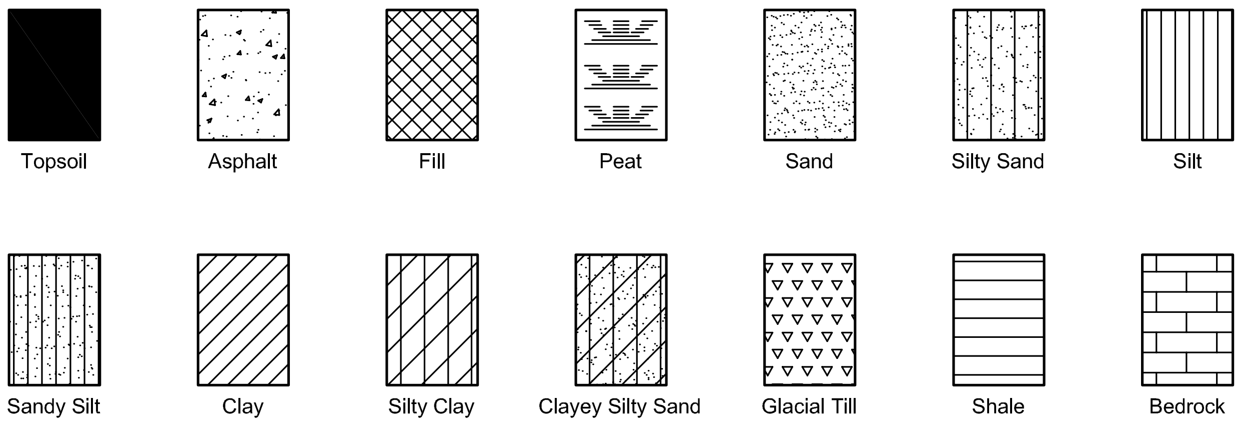
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void 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.

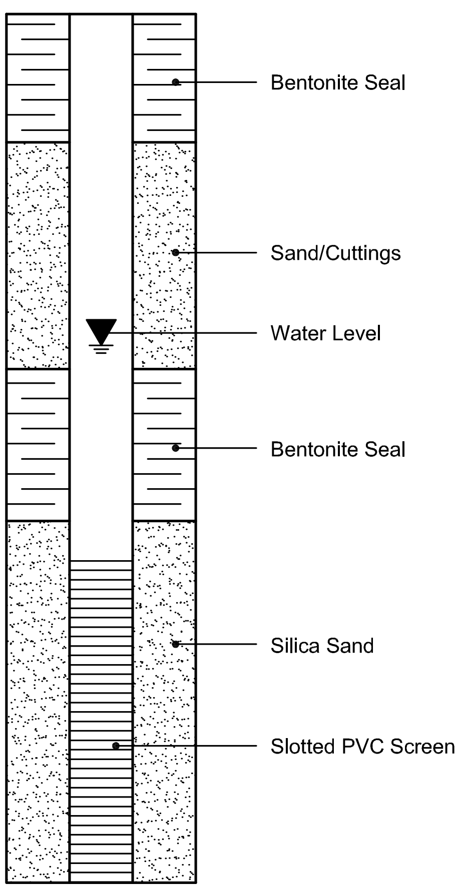
SYMBOLS AND TERMS (continued)

STRATA PLOT

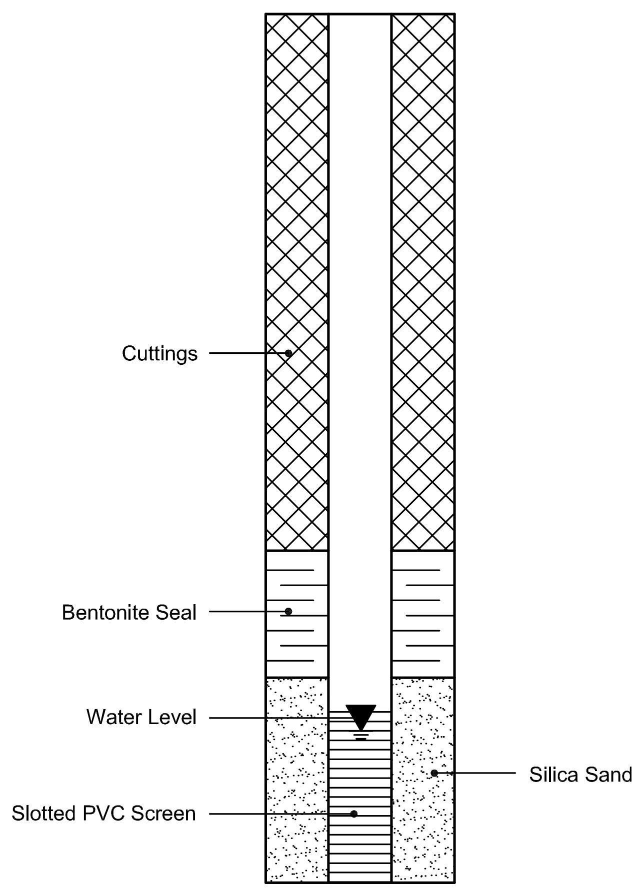


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



APPENDIX 2

FIGURE 1 - KEY PLAN

FIGURES 2 AND 3 - SEISMIC SHEAR WAVE VELOCITY PROFILES

DRAWING PG4163-1 - TEST HOLE LOCATION PLAN

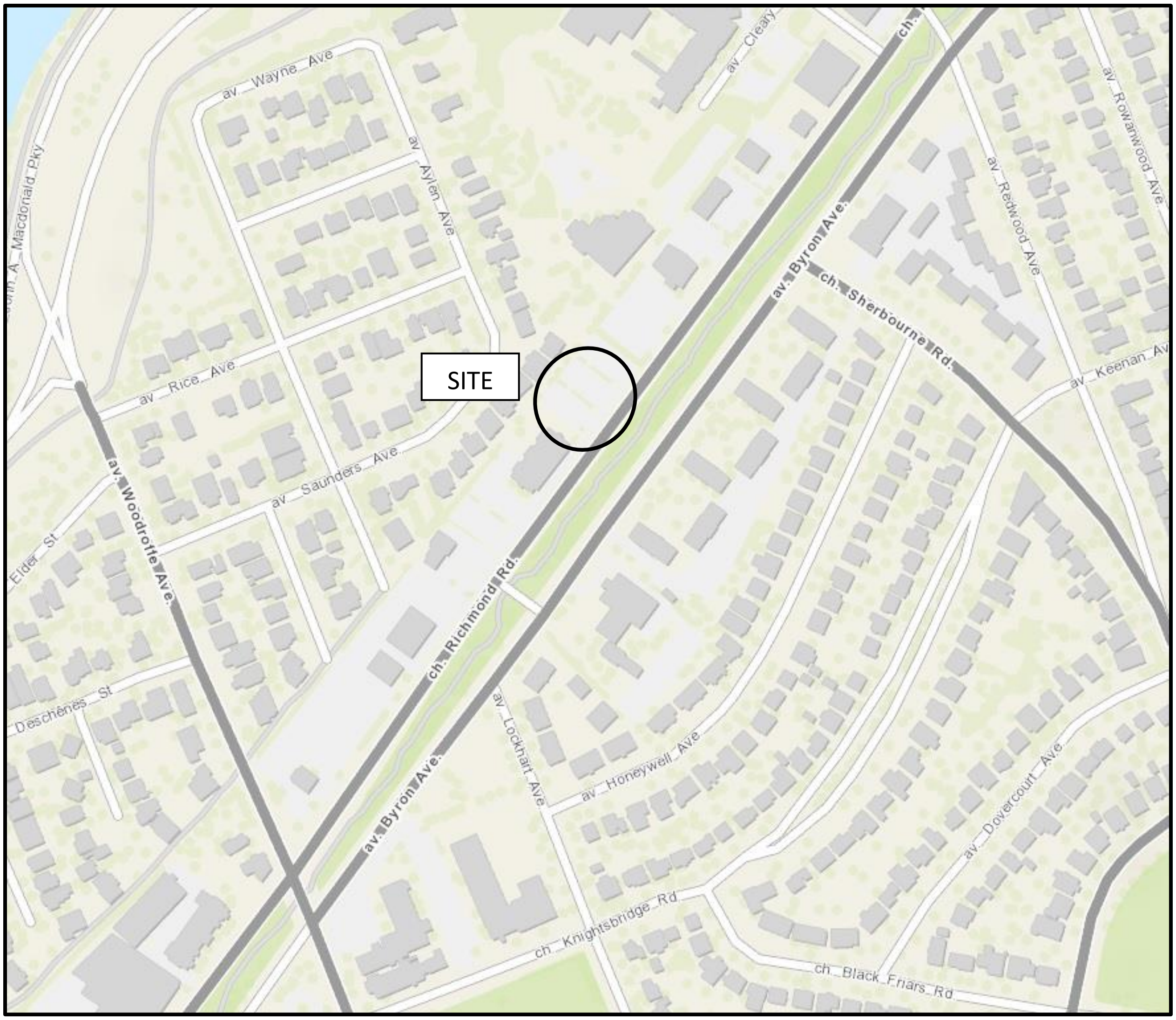


FIGURE 1

KEY PLAN

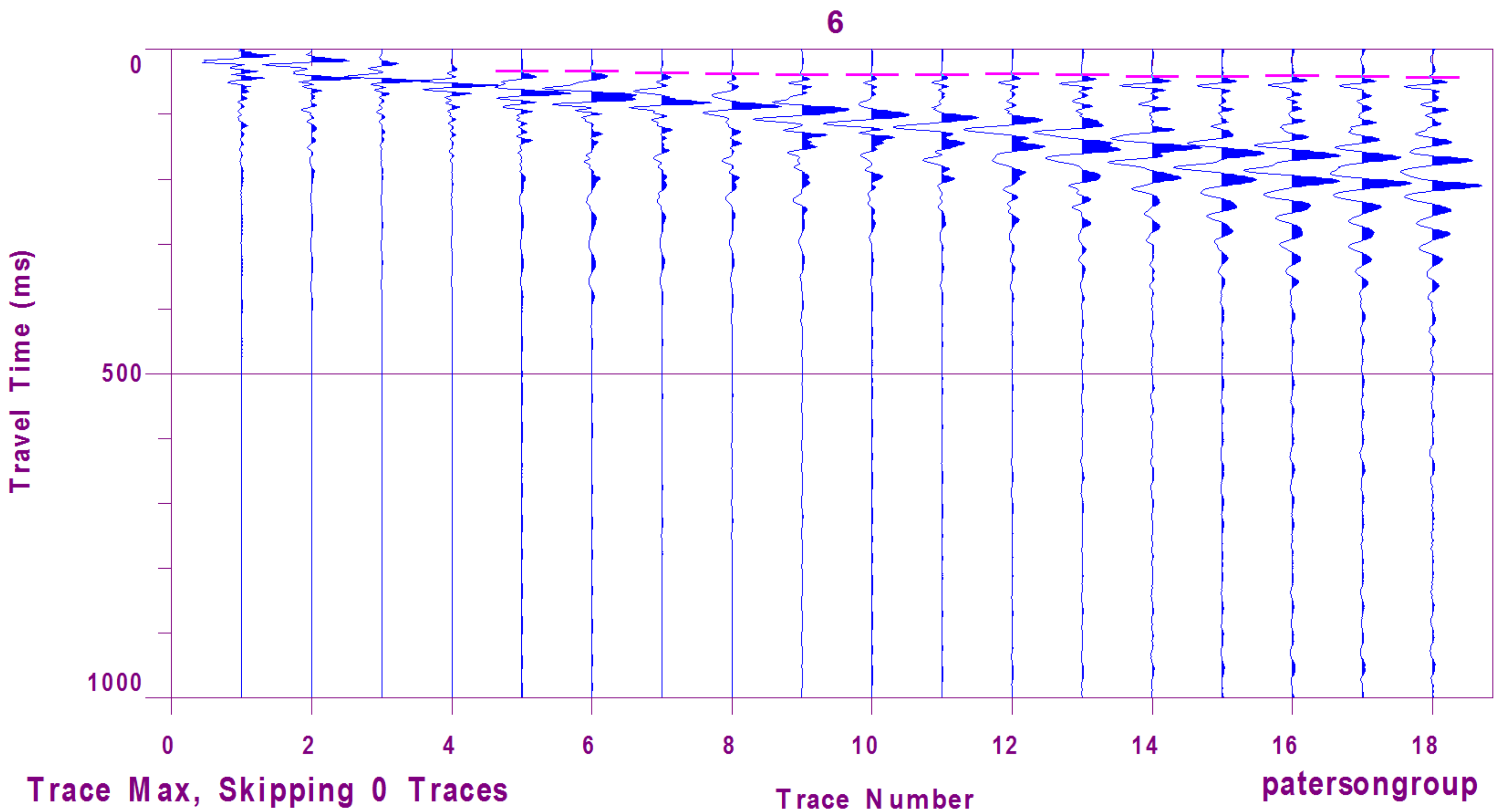


Figure 2 – Shear Wave Velocity Profile at Shot Location -3 m

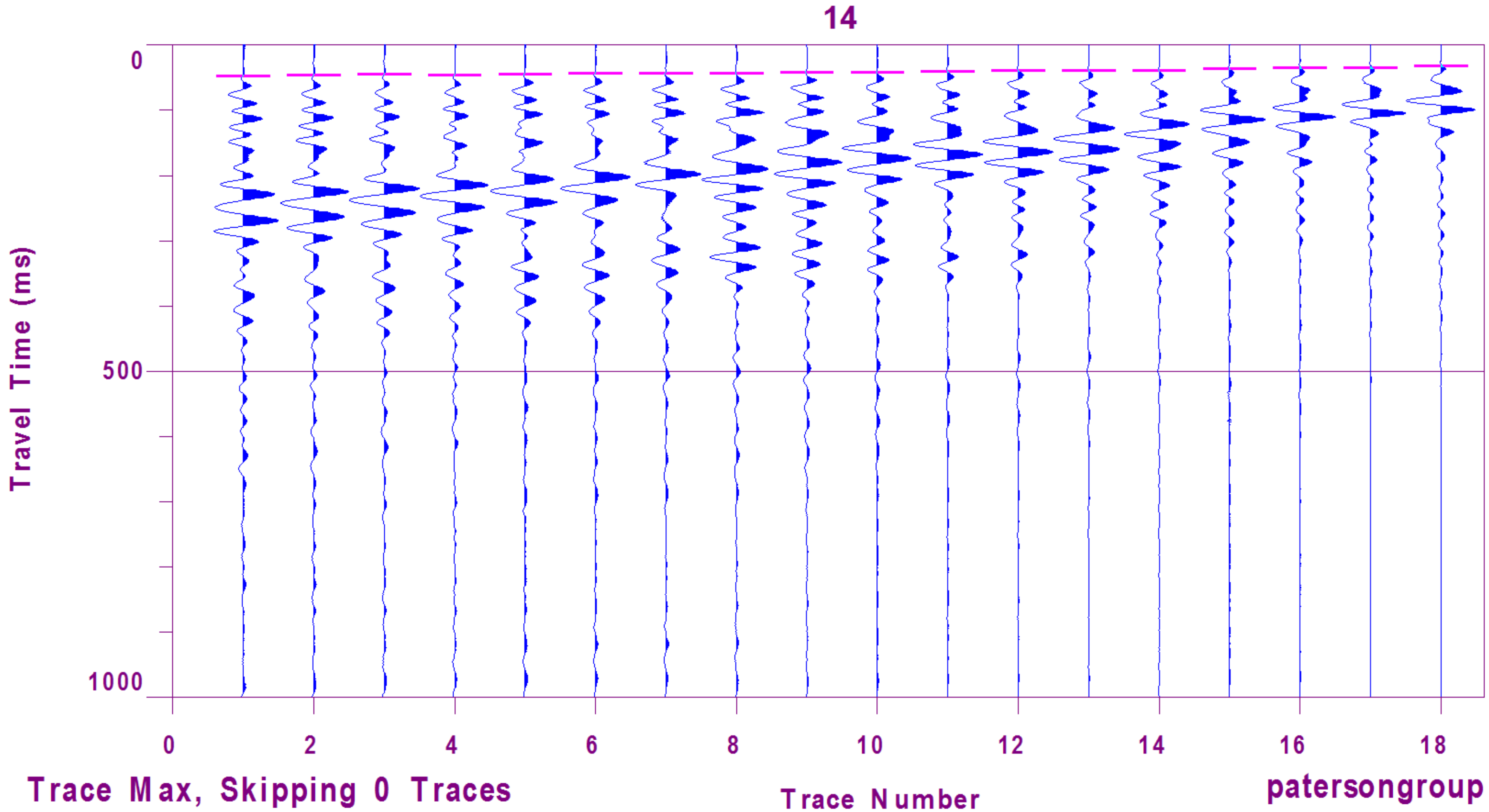
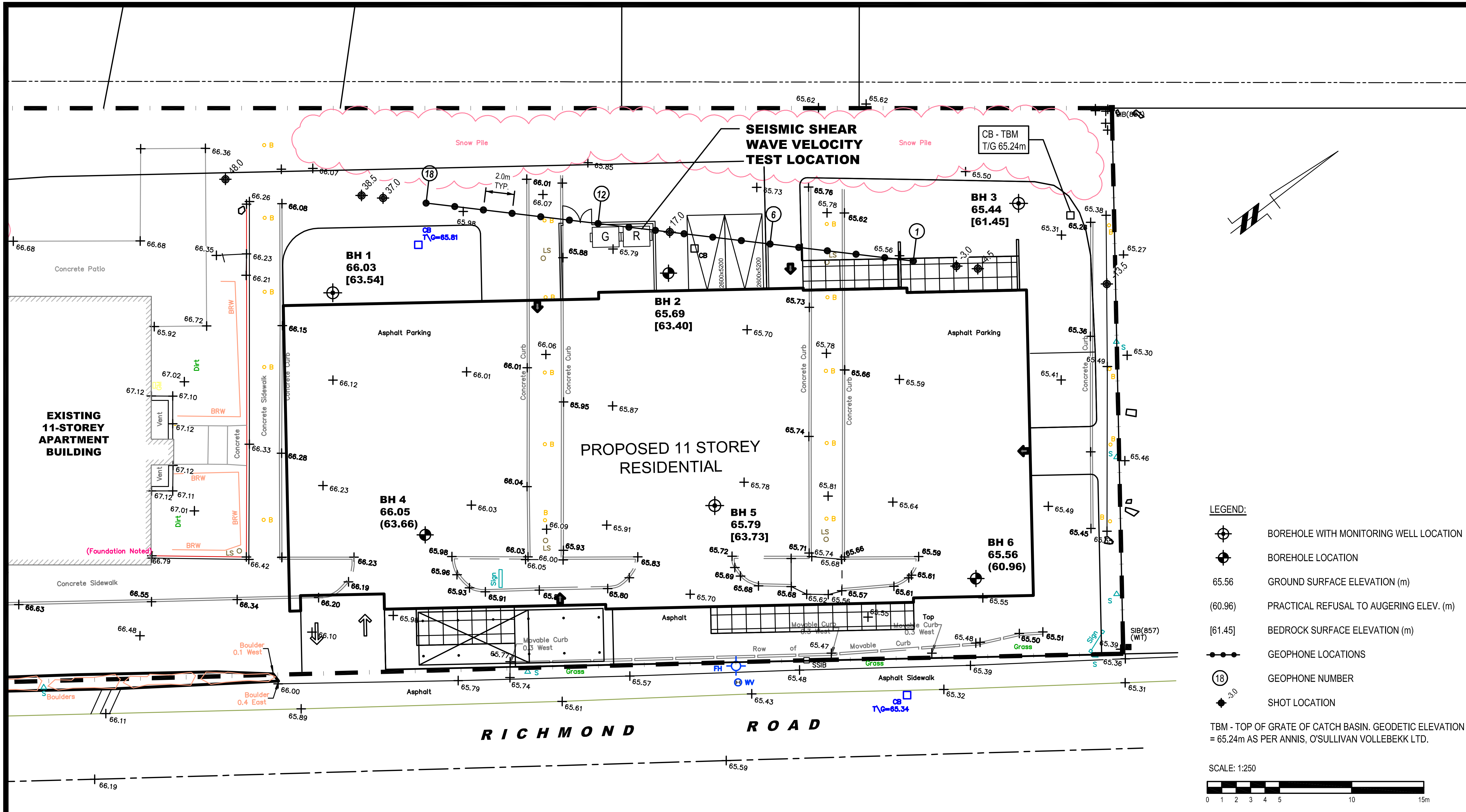


Figure 3 – Shear Wave Velocity Profile at Shot Location 48 m



patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
0			

HOMESTEAD LAND HOLDINGS

GEOTECHNICAL INVESTIGATION
PROP. MULTI-STOREY BUILDING - 851 RICHMOND ROAD

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 06/2017
Drawn by: MPG	Report No.: PG4163-1
Checked by: NC	Dwg. No.: PG4163-1
Approved by: DJG	Revision No.: 0

p:\autocad drawings\geotechnical\pg41xx\pg4163\pg4163-1.dwg

Appendix F CITY OF OTTAWA SERVICING STUDY CHECKLIST

Stantec

Development Servicing Study Checklist

Job#: 160401329

4.1 General Content	Addressed (Y/N/NA)	Section	Comments
Executive Summary (for larger reports only).	N/A	-	Introduction
Date and revision number of the report.	Y	-	
Location map and plan showing municipal address, boundary, and layout of proposed development.	Y	1.0	
Plan showing the site and location of all existing services.	Y		Existing Condtions Plan
Development statistics, land use, density, adherence to zoning and official plan, and reference to applicable subwatershed and watershed plans that provide context to which individual developments must adhere.	Y		Appendix B
Summary of Pre-consultation Meetings with City and other approval agencies.	N/A		
Reference and confirm conformance to higher level studies and reports (Master Servicing Studies, Environmental Assessments, Community Design Plans), or in the case where it is not in conformance, the proponent must provide justification and develop a defendable design criteria.	N/A		
Statement of objectives and servicing criteria.	Y		In each section
Identification of existing and proposed infrastructure available in the immediate area.	Y		In each section
Identification of Environmentally Significant Areas, watercourses and Municipal Drains potentially impacted by the proposed development (Reference can be made to the Natural Heritage Studies, if available).	N/A		
Concept level master grading plan to confirm existing and proposed grades in the development. This is required to confirm the feasibility of proposed stormwater management and drainage, soil removal and fill constraints, and potential impacts to neighbouring properties. This is also required to confirm that the proposed grading will not impede existing major system flow paths.	N/A		
Identification of potential impacts of proposed piped services on private services (such as wells and septic fields on adjacent lands) and mitigation required to addresspotential impacts.	N/A		
Proposed phasing of the development, if applicable.	N/A		
Reference to geotechnical studies and recommendations concerning servicing.		9.0	Report and Appendix
All preliminary and formal site plan submissions should have the following information:			
Metric scale	Y		Appendix G Drawings
North arrow (including construction North)	N/A		Appendix G Drawings
Key plan	Y		Appendix G Drawings
Name and contact information of applicant and property owner	Y		Appendix G Drawings
Property limits including bearings and dimensions	Y		Appendix G Drawings
Existing and proposed structures and parking areas	Y		Appendix G Drawings
Easements, road widening and rights-of-way	Y		Appendix G Drawings
Adjacent street names	Y		Appendix G Drawings

4.2 Water	Addressed (Y/N/NA)	Section	Comments
Confirm consistency with Master Servicing Study, if available	N/A	3.0	
Availability of public infrastructure to service proposed development	Y	3.0	
Identification of system constraints	Y	3.0	
Identify boundary conditions	Y	3.0	
Confirmation of adequate domestic supply and pressure	Y	3.0	
Confirmation of adequate fire flow protection and confirmation that fire flow is calculated as per the Fire Underwriter's Survey. Output should show available fire flow at locations throughout the development.		3.0	Appendix A
Provide a check of high pressures. If pressure is found to be high, an assessment is required to confirm the application of pressure reducing valves.	Y	3.0	
Definition of phasing constraints. Hydraulic modeling is required to confirm servicing for all defined phases of the project including the ultimate design.	N/A		
Address reliability requirements such as appropriate location of shut-off valves	N/A		
Check on the necessity of a pressure zone boundary modification.	N/A		
Reference to water supply analysis to show that major infrastructure is capable of delivering sufficient water for the proposed land use. This includes data that shows that the expected demands under average day, peak hour and fire flow conditions provide water within the required pressure range		3.0	
Description of the proposed water distribution network, including locations of proposed connections to the existing system, provisions for necessary looping, and appurtenances (valves, pressure reducing valves, valve chambers, and fire hydrants) including special metering provisions.	Y	3.0	
Description of off-site required feedermains, booster pumping stations, and other water infrastructure that will be ultimately required to service proposed development, including financing, interim facilities, and timing of implementation.	Y	3.0	
Confirmation that water demands are calculated based on the City of Ottawa Design Guidelines.	Y	3.0	
Provision of a model schematic showing the boundary conditions locations, streets, parcels, and building locations for reference.	N/A		

4.3 Wastewater	Addressed (Y/N/NA)	Section	Comments
Summary of proposed design criteria (Note: Wet-weather flow criteria should not deviate from the City of Ottawa Sewer Design Guidelines. Monitored flow data from relatively new infrastructure cannot be used to justify capacity requirements for proposed infrastructure).	Y	4.0	
Confirm consistency with Master Servicing Study and/or justifications for deviations.	N/A		
Consideration of local conditions that may contribute to extraneous flows that are higher than the recommended flows in the guidelines. This includes groundwater and soil conditions, and age and condition of sewers.	N/A		
Description of existing sanitary sewer available for discharge of wastewater from proposed development.	Y	4.0	
Verify available capacity in downstream sanitary sewer and/or identification of upgrades necessary to service the proposed development. (Reference can be made to previously completed Master Servicing Study if applicable)	Y	4.0	Appendix C
Calculations related to dry-weather and wet-weather flow rates from the development in standard MOE sanitary sewer design table (Appendix 'C') format.	Y	4.0	Appendix C
Description of proposed sewer network including sewers, pumping stations, and forcemains.	Y	4.0	
Discussion of previously identified environmental constraints and impact on servicing (environmental constraints are related to limitations imposed on the development in order to preserve the physical condition of watercourses, vegetation, soil cover, as well as protecting against water quantity and quality).	N/A		
Pumping stations: impacts of proposed development on existing pumping stations or requirements for new pumping station to service development.	N/A		
Forcemain capacity in terms of operational redundancy, surge pressure and maximum flow velocity.	N/A		
Identification and implementation of the emergency overflow from sanitary pumping stations in relation to the hydraulic grade line to protect against basement flooding.	N/A		
Special considerations such as contamination, corrosive environment etc.	N		

4.4 Stormwater	Addressed (Y/N/NA)	Section	Comments
Description of drainage outlets and downstream constraints including legality of outlets (i.e. municipal drain, right-of-way, watercourse, or private property)	Y	5.0	
Analysis of available capacity in existing public infrastructure.	N		
A drawing showing the subject lands, its surroundings, the receiving watercourse, existing drainage patterns, and proposed drainage pattern.	Y		Existing Conditions Plan
Water quantity control objective (e.g. controlling post-development peak flows to pre-development level for storm events ranging from the 2 or 5 year event (dependent on the receiving sewer design) to 100 year return period); if other objectives are being applied, a rationale must be included with reference to hydrologic analyses of the potentially affected subwatersheds, taking into account long-term cumulative effects.	Y	5.0	Appendix D
Water Quality control objective (basic, normal or enhanced level of protection based on the sensitivities of the receiving watercourse) and storage requirements.	Y	5.0	Appendix D
Description of the stormwater management concept with facility locations and descriptions with references and supporting information.	Y	5.0	Appendix D
Set-back from private sewage disposal systems.	N/A		
Watercourse and hazard lands setbacks.	N/A		
Record of pre-consultation with the Ontario Ministry of Environment and the Conservation Authority that has jurisdiction on the affected watershed.	N		
Confirm consistency with sub-watershed and Master Servicing Study, if applicable study exists.	N/A		
Storage requirements (complete with calculations) and conveyance capacity for minor events (1:5 year return period) and major events (1:100 year return period).	Y	5.0	Appendix D
Identification of watercourses within the proposed development and how watercourses will be protected, or, if necessary, altered by the proposed development with applicable approvals.	N		
Calculate pre and post development peak flow rates including a description of existing site conditions and proposed impervious areas and drainage catchments in comparison to existing conditions.	Y	5.0	Appendix D
Any proposed diversion of drainage catchment areas from one outlet to another.	N/A		
Proposed minor and major systems including locations and sizes of stormwater trunk sewers, and stormwater management facilities.	N/A		
If quantity control is not proposed, demonstration that downstream system has adequate capacity for the post-development flows up to and including the 100-year return period storm event.	N/A		
Identification of potential impacts to receiving watercourses	N/A		
Identification of municipal drains and related approval requirements.	N/A		
Descriptions of how the conveyance and storage capacity will be achieved for the development.	Y	5.0	Appendix D
100 year flood levels and major flow routing to protect proposed development from flooding for establishing minimum building elevations (MBE) and overall grading.	N		
Inclusion of hydraulic analysis including hydraulic grade line elevations.	N		
Description of approach to erosion and sediment control during construction for the protection of receiving watercourse or drainage corridors.	Y	5.0	
Identification of floodplains – proponent to obtain relevant floodplain information from the appropriate Conservation Authority. The proponent may be required to delineate floodplain elevations to the satisfaction of the Conservation Authority if such information is not available or if information does not match current conditions.	N/A		
Identification of fill constraints related to floodplain and geotechnical investigation.	N/A		

4.5 Approval and Permit Requirements	Addressed (Y/N/NA)	Section	Comments
Conservation Authority as the designated approval agency for modification of floodplain, potential impact on fish habitat, proposed works in or adjacent to a watercourse, cut/fill permits and Approval under Lakes and Rivers Improvement Act. The Conservation Authority is not the approval authority for the Lakes and Rivers Improvement Act. Where there are Conservation Authority regulations in place, approval under the Lakes and Rivers Improvement Act is not required, except in cases of dams as defined in the Act.	N/A		
Application for Certificate of Approval (CofA) under the Ontario Water Resources Act.	N/A		
Changes to Municipal Drains.	N/A		
Other permits (National Capital Commission, Parks Canada, Public Works and Government Services Canada, Ministry of Transportation etc.)	N/A		

4.6 Conclusion	Addressed (Y/N/NA)	Section	Comments
Clearly stated conclusions and recommendations	Y	10.0	
Comments received from review agencies including the City of Ottawa and information on how the comments were addressed. Final sign-off from the responsible reviewing agency.	N		
All draft and final reports shall be signed and stamped by a professional Engineer registered in Ontario	Y		

Appendix G DRAWINGS