335 ROOSEVELT AVENUE

ASSESSMENT OF ADEQUACY OF PUBLIC SERVICES

Prepared for:

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K2G 5X3

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August 4, 2020

Novatech File: 110098
Ref: R-2020-099

NOVATECH
Engineers, Planners & Landscape Architects

August 4, 2020

City of Ottawa
Planning and Growth Management Department
Development Review (Urban Services - Central) Branch
Infrastructure Approvals Division
110 Laurier Avenue West, 4th Floor
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Attention: **Steve Gauthier, MCIP, RPP**
Planner

Shawn Wessel, A.Sc.T.,rcji
Project Manager

Reference: **335 Roosevelt Avenue**
Assessment of Adequacy of Public Services
Novatech File No.: 110098

In support of the Official Plan Amendment and Rezoning applications for the above-noted site, you will find enclosed the Assessment of Adequacy of Public Services for the development at 335 Roosevelt Avenue.

This report addresses the approach to site servicing and stormwater management for the subject site, which been developed based on the requirements of the City of Ottawa and Rideau Valley Conservation Authority.

Should you have any questions, or require additional information, please contact me.

Yours truly,

NOVATECH

Bassam Bahia, M.Eng., P. Eng.
Project Manager | Land Development

/bs

cc: Eric Lalande, Rideau Valley Conservation Authority
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1.0 INTRODUCTION

1.1 Background

This report addresses the approach to site servicing for the development at 335 Roosevelt Avenue (Subject Site), which is being proposed by Uniform Urban Developments Ltd. (Developer).

The Subject Site is located to the north of the Wilmont Avenue and Winston Avenue intersection, as shown on **Figure 1.1** – Key Plan. The site is bound to the north by the transitway, to the west by Roosevelt Avenue, to the south by existing residences fronting Winston Avenue and Wilmont Avenue, and to the east by an existing apartment building.

The existing land usage consists of a vacant buildings and asphalt parking area, as shown on **Figure 1.2** – Existing Conditions Plan. The Subject Site is relatively flat.

1.2 Development Intent

The Subject Site has an area 0.72ha, and the proposed development will comprise of two condominium towers (18 and 21 storeys) having a total of 323 units, and four low rise buildings (each 3 storeys) having a total of 38 units, as shown in **Table 1.1**. The development will include two levels of underground parking that is understood to encompass the entire site, with access off Roosevelt Avenue at the west side of the site, as well as access from Wilmont Avenue at the south side of the site. The proposed site plan is shown on **Figure 1.3** – Site Plan.

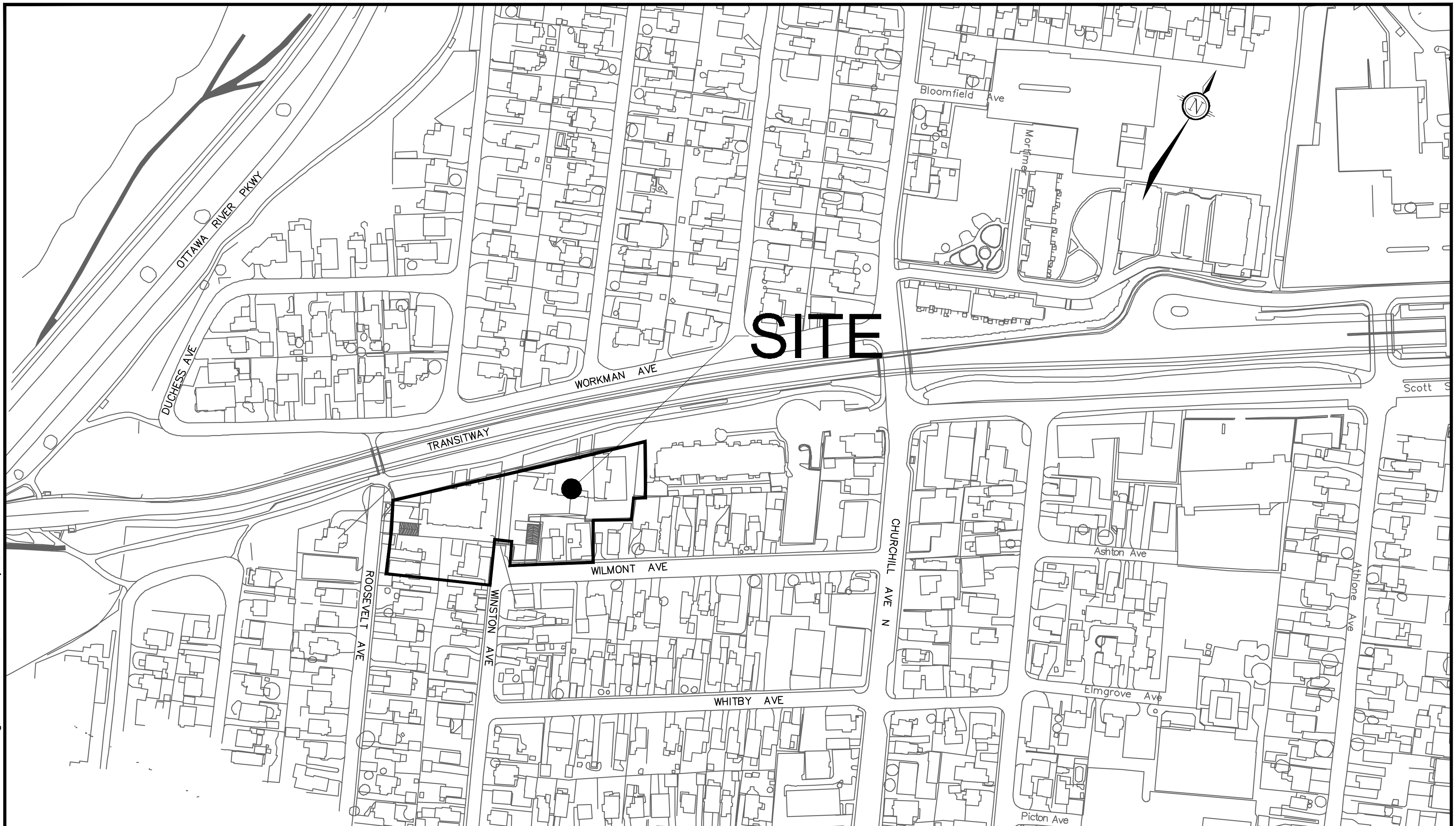
Table 1.1: Land Use, Development Potential, and Yield

Unit Type	Number of Units
Condominium Tower - Building #1 (West)	175
Condominium Tower - Building #2 (East)	148
Low Rise Building – Block A	5
Low Rise Building – Block B	9
Low Rise Building – Block C	12
Low Rise Building – Block D	12
Total	361

The Subject Site is located within the service area in the City of Ottawa Official Plan; therefore, the site has been designed with city water and sanitary sewage collection.

It should also be noted that there are Capital Works projects planned within the vicinity of the Subject Site. These include the following:

- Road and Sewer Renewal project planned for Wilmont Avenue within the next 3-5 years;
- New transit way (LRT) planned to start this season; and
- Road and Sewer Renewal project planned for Winona Avenue within the next 3-5 years.



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CITY OF OTTAWA

335 ROOSEVELT AVE

KEY PLAN

SCALE		
NOT TO SCALE		
DATE JUL 2020	JOB 110098	FIGURE 1.1

M:\2010\110098\CAD\Design\Figures\Design Brief\110098-FIG-1.2.dwg, FIG 1.2, Jul 16, 2020 - 3:39pm, mdias

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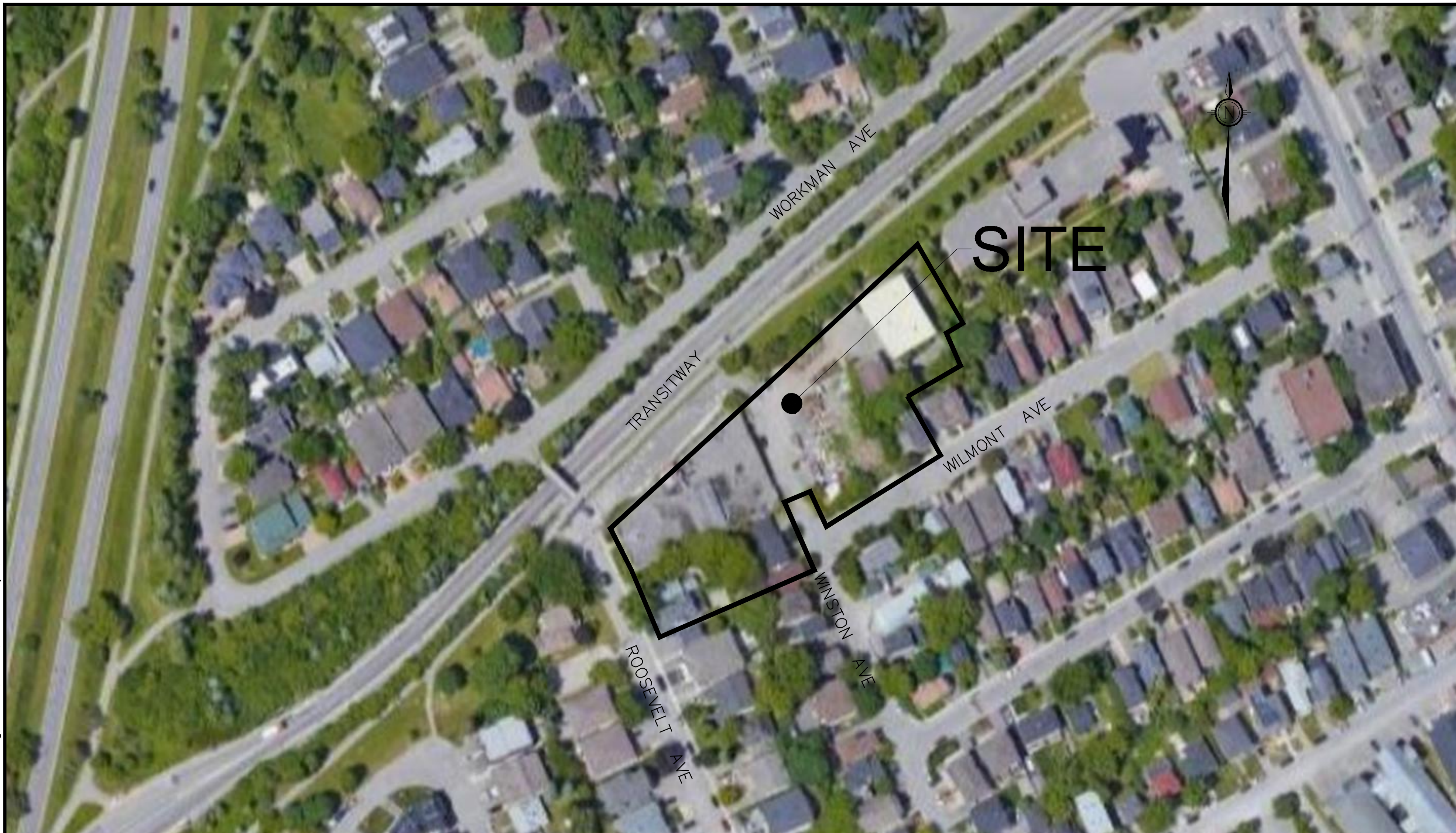

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CITY OF OTTAWA
335 ROOSEVELT AVE

EXISTING CONDITIONS

SCALE	NOT TO SCALE	
DATE JUL 2020	JOB 110098	FIGURE 1.2

SHT8X11.DWG - 216mmx279mm

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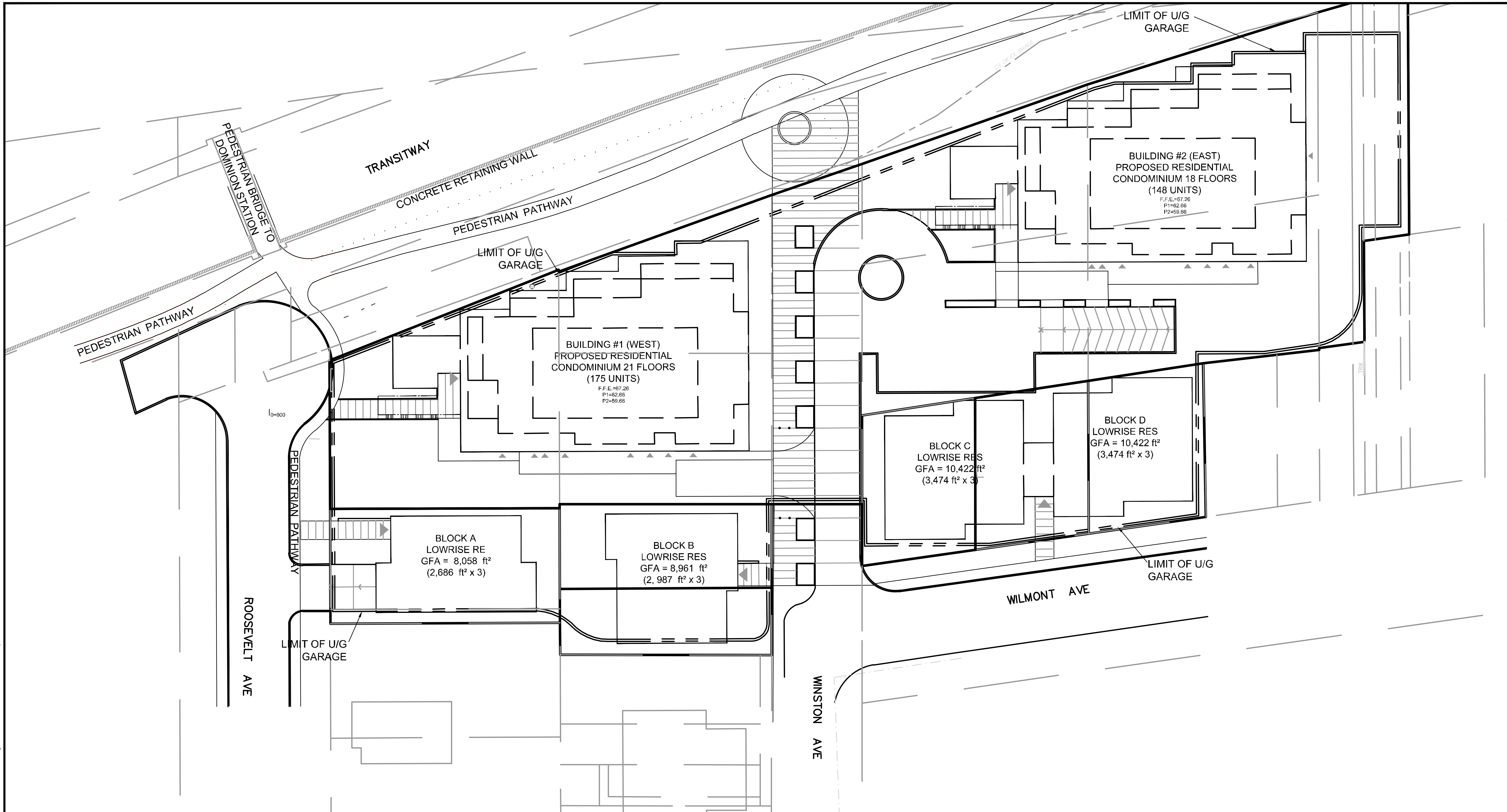

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CITY OF OTTAWA
335 ROOSEVELT AVANUE

SITE PLAN

SCALE	DATE	JOB	FIGURE
1 : 500	JULY 2020	110098	1.3

M:\2010\110098\CAD\Design\Figures\Design Brief\110098-FIG-1.3.dwg, FIG 1.3, Jul 16, 2020 - 4:22pm, bsweet

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1.3 Report Objective

This report assesses the adequacy of existing public services to support the proposed development. This report will be provided to the various agencies for approval and to obtain any applicable permits.

The City of Ottawa Applicant Study and Plan Identification List along with proof of a pre-consultation meeting is provided in **Appendix A**.

The City of Ottawa Servicing Study Guidelines for Development Applications checklist has been completed and is provided in **Appendix B**.

2.0 REFERENCES AND SUPPORTING DOCUMENTS

2.1 Guidelines and Supporting Studies

The following guidelines and supporting documents were utilized in the preparation of this report:

- **City of Ottawa Official Plan** (OP)
  City of Ottawa, adopted by Council 2003.
- **City of Ottawa Infrastructure Master Plan** (IMP)
  City of Ottawa, November 2013.
- **City of Ottawa Water Distribution Guidelines** (OWDG)
  City of Ottawa, October 2012.
- **Revisions to OWDG** (ISTB-2010-01, ISTB-2014-02, ISTB-2018-02, ISTB-2018-04)
  City of Ottawa, December 2010, May 2014, March 2018, and June 2018.
- **City of Ottawa Sewer Design Guidelines** (OSDG)
  City of Ottawa, October 2012.
- **Revisions to OSDG** (ISTB-2016-01, ISTB-2018-01)
  City of Ottawa, September 2016 and March 2018.
- **Design Guidelines for Sewage Works and Drinking Water System**
  Ontario's Ministry of the Environment, 2008.
- **Ministry of the Environment Stormwater Management Planning & Design Manual** (MOE SWM Manual)
  Ontario's Ministry of the Environment, March 2003.
- **335 Roosevelt Avenue Development Servicing Study** (Report Ref: R-2012-001)
  Novatech, May 2012.

2.2 Geotechnical Investigation

Paterson Group Inc. (Paterson) conducted a geotechnical investigation (**Appendix G**) in support of the proposed residential development:

Geotechnical Investigation – Proposed Residential Development 335 Roosevelt Avenue, Ottawa, Ontario; Report No. PG2178-1 (revision 1), Paterson Group Inc., July 26, 2011.

Based on the geotechnical study, it is not anticipated that there will be any significant geotechnical concerns with respect to servicing and developing the site. It should be noted that protection and monitoring of the existing 1200mm diameter watermain and the West Nepean Collector, running parallel to the northern property line of the Subject Site, will be required during the bedrock removal (refer to the geotechnical study for further details). A summary of the geotechnical report findings is provided in **Table 2.1** below.

Table 2.1: Summary of Geotechnical Servicing and Grading Considerations

Parameter	Summary
Sub-Soil Conditions	Silty sand, silty sand with some gravel and clay, silty clay or silt, and bedrock
Grade Raise Restriction	N/A
Groundwater Considerations	Low groundwater level (3.8m to 6.5m depths) It is recommended that basement walls and foundation drainage consider groundwater/hydrostatic pressure. Rock anchors are recommended to resist hydrostatic uplift forces.
Bedrock	Shallow bedrock encounter at boreholes (0.7m to 1m depths) Line drilling of the perimeter and rock blasting and/or hoe ramming expected.
Pipe Bedding / Backfill	Pipe Bedding 150 mm to 300 mm Granular A Pipe Cover 300 mm Granular A Backfill Native Material
Pavement Structure (Car Only Parking Areas)	50mm Wear Course (SuperPave 12.5) 150mm Base (Granular A) 300mm Subbase (Granular B Type II)
Pavement Structure (Access Lanes)	40mm Wear Course (SuperPave 12.5) 50mm Binder Course (SuperPave 19.0) 150mm Base (Granular A) 400mm Subbase (Granular B Type I or II)
Landscape Consideration	N/A

3.0 STORM SEWER SYSTEM AND STORMWATER MANAGEMENT

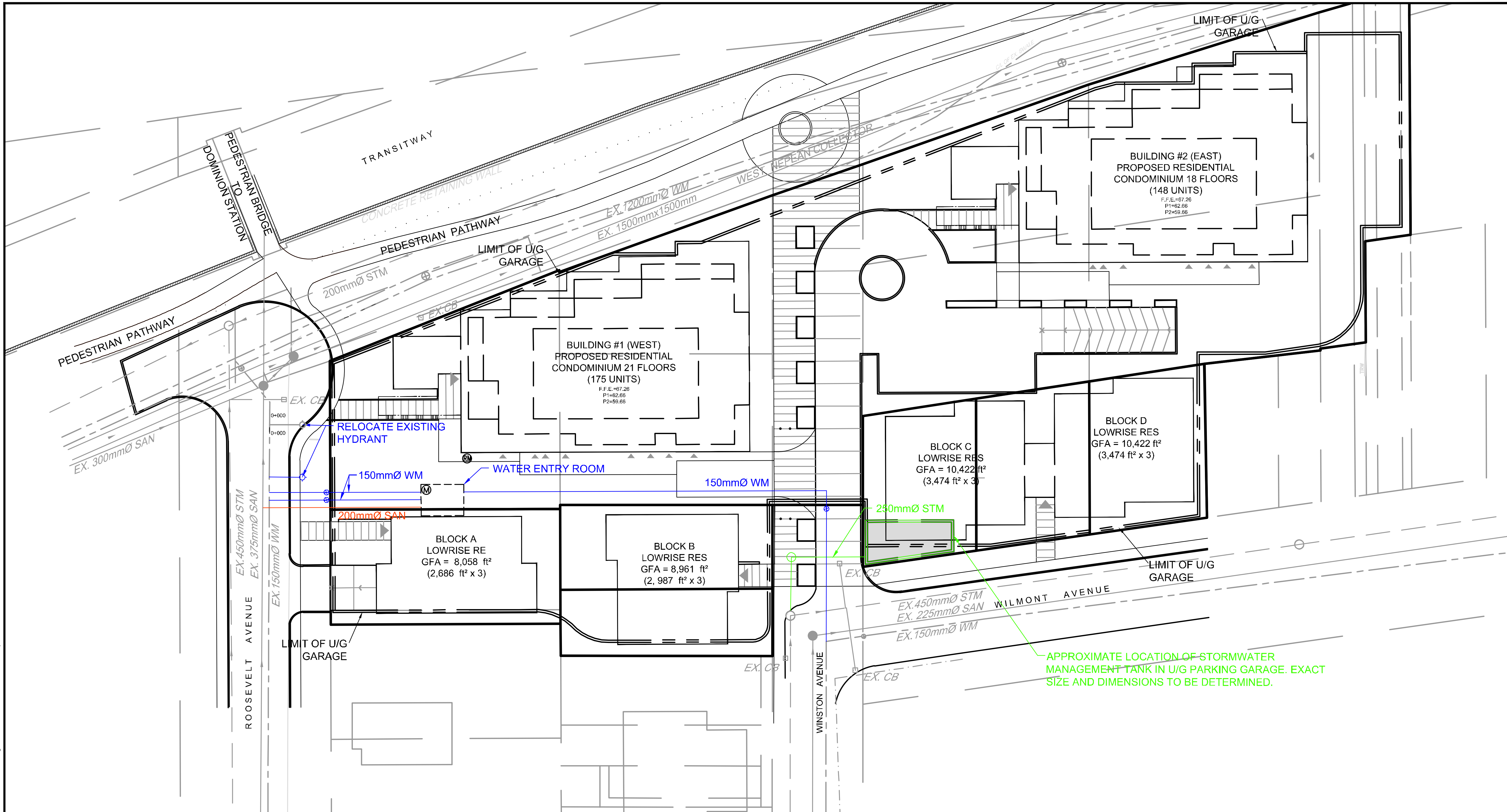
3.1 Storm Infrastructure

The proposed development will be serviced with a 250mm diameter storm service connected to the existing 450mm diameter storm sewer in Wilmont Avenue which ultimately outlets to the West Transit Storm and outlets to the Ottawa River near Onigam Street.

Refer to **Figure 3.1** – Proposed Servicing Layout Plan for an illustration of the proposed storm service, and existing storm sewers.

3.2 Stormwater Management Criteria

The Subject Site is located within the Ottawa River West subwatershed, which falls under the jurisdiction of the Rideau Valley Conservation Authority (RVCA). The following stormwater management criteria has been developed based on the criteria in the OSDG, subsequent Technical Bulletins, and the pre-consultation meeting discussions. As such, the City will require that on-site stormwater quantity control be implemented to control post-development stormwater discharge for any storm events greater than the 5-year, up to and including the 100-year event. No on-site stormwater quality control is required for the site.



LEGEND

- EXISTING WATERMAIN
- EXISTING HYDRANT
- PROPOSED WATERMAIN
- PROPOSED HYDRANT
- EXISTING SANITARY SEWER
- PROPOSED SANITARY SEWER
- EXISTING STORM SEWER
- PROPOSED STORM SEWER

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CITY OF OTTAWA
335 ROOSEVELT AVANUE

PROPOSED SERVICING LAYOUT PLAN

SCALE	DATE	JOB	FIGURE
1 : 500	JULY 2020	110098	3.1

3.3 Pre-Development Conditions

The Subject Site is currently occupied by three vacant buildings, an asphalt parking area, and landscaped areas. The topographical survey plan prepared by Annis O'Sullivan Vollebekk Ltd indicates that under existing conditions, the site sheet drains to the north towards the Transitway. There is currently no storm sewer system on-site, as such, the majority of the site drainage is collected in the existing low area/swale located within the Transitway property limits. Refer to **Figure 1.2** – Existing Conditions.

3.4 Allowable Release Rates

The following allowable release rates for the Subject Site have been developed based on the timing for the Wilmont Avenue Capital Works project, as the existing 450mm storm sewer in Wilmont Avenue was not designed to accommodate runoff from the entire site area.

In the event the proposed development was to proceed in advance of the Wilmont Avenue Capital Works project (Scenario 1), the allowable release rate would be restricted to 29.2 L/s, for all storms up to and including the 100-year event. This release rate is based on a rainfall intensity of 59.92 mm/hr, a runoff coefficient (C) of 0.45, and an area of 0.39 ha. Refer to **Appendix C** for the MOE Certificate of Approval, the storm sewer design sheet, and the drainage area plan for the existing 450mm storm sewer in Wilmont Avenue.

In the event the proposed development was to proceed after the Wilmont Avenue Capital Works project (Scenario 2), the allowable release rate would be restricted to 70.3 L/s, for all storms up to and including the 100-year event. This release rate is based on a time-of-concentration (Tc) of 20 minutes corresponding to a rainfall intensity of 70.25 mm/hr, a runoff coefficient (C) of 0.50, and an area of 0.72 ha. For this scenario the allowable release rate is calculated using the select criteria outlined within the OSDG and pre-consultation meeting discussions.

As the governing allowable release rate will be dependant on timing of the Wilmont Avenue Capital Works project and the development of the Subject Site, the allowable release rate will be confirmed during the detailed design stage.

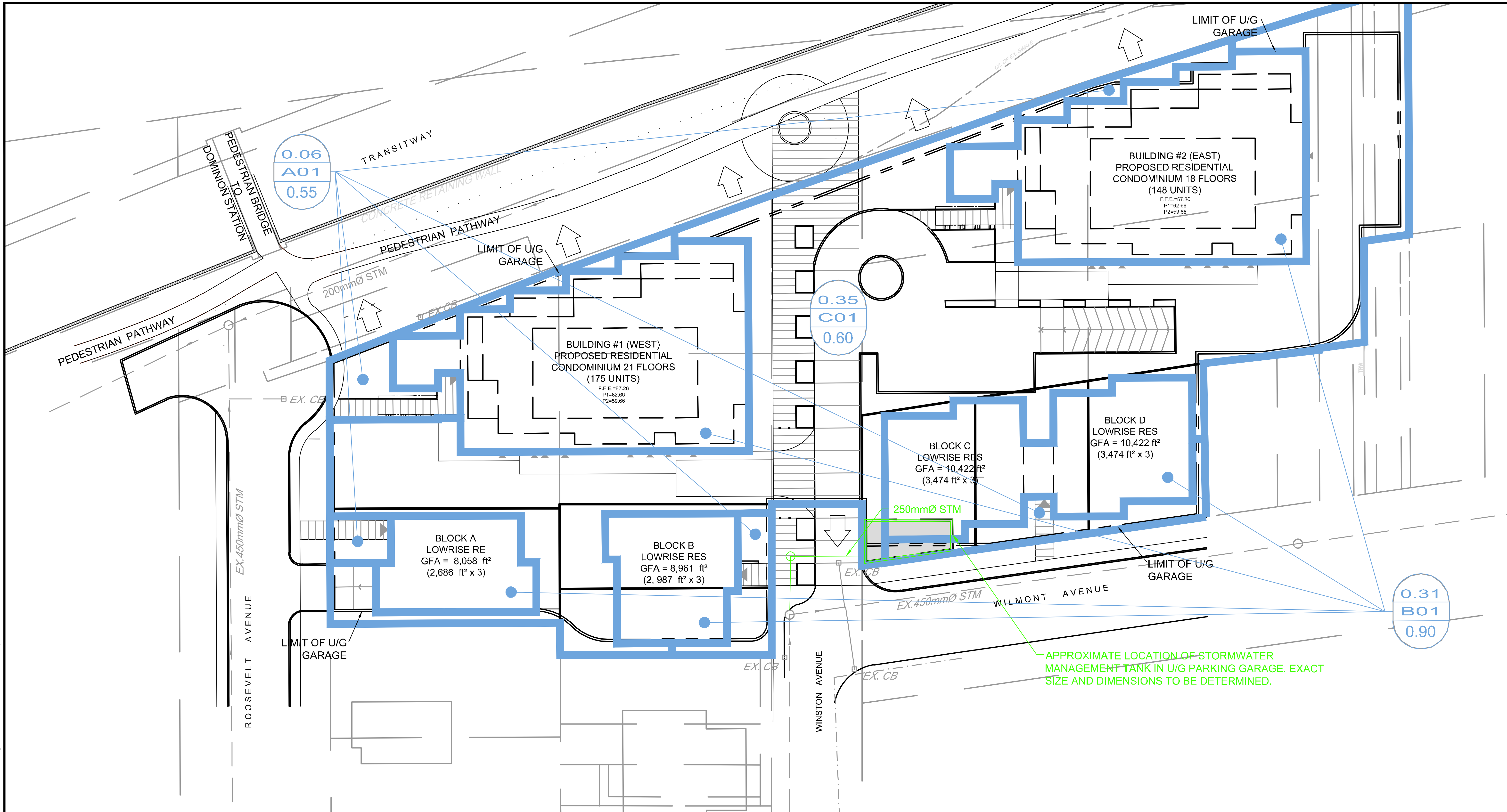
3.5 Stormwater Quantity Control

Stormwater runoff from the Subject Site will consist of both uncontrolled and controlled flows. Stormwater quantity control will be provided using underground storage.

Refer to **Figure 4.1** – Post-Development Storm Drainage Area Plan for details on the drainage areas. A description of each area is as follows:

A-01: Areas A-01 consists of landscape areas along the site boundary. These areas will remain uncontrolled and drain to the existing catch basins within Roosevelt Avenue, Wilmont Avenue, and the Transitway where stormwater will outlet into the Roosevelt Avenue and Wilmont Avenue storm sewers per existing conditions. The calculated post-development flows are a significant decrease compared to the entire existing site sheet draining uncontrolled to the landscaped area, thus the small uncontrolled release rate should not adversely affect the downstream public sewers.

B-01: Areas B-01 consists of the rooftop areas within the site boundary. These areas will be controlled using roof drains and scuppers.



LEGEND

0.06 / A01 / 0.25

- DRAINAGE AREA (hectares)
- MANHOLE TO MANHOLE
- RUN-OFF COEFFICIENT
- STORM DRAINAGE AREA
- EMERGENCY OVERLAND FLOW
- *EXISTING STORM SEWER*
- PROPOSED STORM SEWER

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POST-DEVELOPMENT STORM DRAINAGE AREA PLAN

SCALE	DATE	JOB	FIGURE
1 : 500 (0, 5m, 10m, 20m)	JULY 2020	110098	4.1

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C-01: Areas C-01 consists of the parking lot and landscape areas within the site boundary. These areas will be controlled using area drains and an underground storage system complete with an inlet control device (ICD).

Table 3.1 below summarizes the total post-development flow (uncontrolled + controlled) from the Subject Site for the 5-year and 100-year design events, and storage required for each catchment area for Scenario 1. In this scenario; flows from Areas B-01 and C-01 will be left uncontrolled and will outlet to the underground storage system. Flow from the storage system will then be pumped to the 250mm diameter storm service connected to the Wilmont Avenue sewer. A “stand-by” pump will be provided for emergency and/or maintenance purposes. An emergency power supply will also be provided.

Table 3.1: Scenario 1 (29.2 L/s) - Stormwater Management Summary

Area ID	Area (ha)	1:5 Year Weighted Cw	5-Year Storm Event		100-Year Storm Event	
			Release (L/s)	Req'd Vol (cu.m)	Release (L/s)	Req'd Vol (cu.m)
A-01	0.060	0.55	6.4	N/A	12.5	N/A
B-01/C-01	0.660	0.74	16.7	104.08	16.7	248.54
Total Flow to Wilmont Ave			**23.1**		**29.2**	
Allowable Flow to Wilmont Ave			**29.2**		**29.2**	

Table 3.2 below summarizes the total post-development flow (uncontrolled + controlled) from the Subject Site for the 5-year and 100-year design events, and storage required for each catchment area for Scenario 2. In this scenario; flows from Areas B-01 will be controlled to optimize rooftop storage, before outletting downstream of the underground storage system, to the 250mm diameter storm service connected to the Wilmont Avenue sewer, flows from Areas C-01 will be controlled to optimize surface storage within the parking lot and landscape areas, before outletting to the underground storage system. Flow from the storage system will then be pumped to the 250mm diameter storm service connected to the Wilmont Avenue sewer. A “stand-by” pump will be provided for emergency and/or maintenance purposes. An emergency power supply will also be provided.

Table 3.2: Scenario 2 (70.3 L/s) - Stormwater Management Summary

Area ID	Area (ha)	1:5 Year Weighted Cw	5-Year Storm Event		100-Year Storm Event	
			Release (L/s)	Req'd Vol (cu.m)	Release (L/s)	Req'd Vol (cu.m)
A-01	0.060	0.55	6.4	N/A	12.5	N/A
B-01	0.310	0.90	12.6	52.54	17.6	113.18
C-01	0.350	0.60	40.2	12.38	40.2	48.73
Total Flow to Wilmont Ave			**59.2**		**70.3**	
Allowable Flow to Wilmont Ave			**70.3**		**70.3**	

Refer to **Appendix B** for preliminary Rational Method and Modified Rational Method calculations. Note that during detailed design, dynamic modelling will be provided along with additional details on the pumping of the underground storage system, to account for head fluctuations and ensure the allowable release rate is met.

During the detailed design stage, the following additional information will also be provided:

- A plan detailing roof drain and scupper locations, product name and specifications on drains, drain type and weir openings (if controlled), flow rates, and the 5-year and 100-year ponding limits; and
- A plan detailing area drain locations, product name and specifications on drains, drain type, flow rates, and the 5-year and 100-year ponding limits;
- A plan detailing the underground storage system including product name and model, number of chambers, chamber configuration, dimensions (i.e. length, width, and height), elevations (i.e. inverts, obverts, top of ground, major and minor water levels, etc.), required cover over system, interior bottom slope for self-cleansing, system volume provided during major and minor events, entry/maintenance ports, etc.;
- Details on the foundation drain connections, including whether the drains will be independently connected to sewers or if the flows will be pumped).

3.6 Site Grading & Emergency Overland Flow

As described above the existing site is currently graded to direct runoff north towards the low area/swale within the landscape area between the proposed site and Transitway. The proposed design intent for the site is to contain and direct all stormwater runoff to the on-site area drains while minimizing uncontrolled direct runoff from the site.p The site has two accesses to the underground garage, one from Roosevelt Avenue and one from Wilmont Avenue. Elevations along the existing edge of roadways will be matched into, thus minimizing any disturbances to the surrounding roadways.

In the case of a major rainfall event exceeding the design storms provided for, the stormwater collected on-site will pond to a maximum depth of 0.30m before cascading towards the landscaped area to the north and towards Wilmont/Winston Avenue to the south. The emergency overland flow route is demonstrated on **Figure 4.1** – Post-Development Storm Drainage Area Plan.

During the detailed design stage, a grading plan will be provided to detail the proposed site grading, grading tie-ins, spill elevations, and the emergency overland flow route.

3.7 Assessment of Storm Infrastructure

As outlined in the above sections, all post-development runoff in excess of the allowable will be stored and controlled on-site prior to being released into the Wilmont Avenue sewer. This will be done using roof drains and scuppers, area drains, and an underground storage system located in the second level of the underground parking garage adjacent to Wilmont Avenue.

As mentioned previously, the governing allowable release rate will be dependant on timing of the Wilmont Avenue Capital Works project and the development of the Subject Site, the allowable release rate will be confirmed during the detailed design stage. As the allowable release rate will directly impact the required on-site stormwater quantity control, this will also be further detailed during the detailed design stage.

4.0 SANITARY SEWER SYSTEM

4.1 Sanitary Infrastructure

The proposed development will be serviced with a 200mm diameter sanitary service connected to the existing 375mm diameter sanitary sewer in Roosevelt Avenue which ultimately outlets to the West Nepean Collector via the existing 450mm sewer from City of Ottawa manhole 47044.

As mentioned above, it is important to note that protection of the existing West Nepean Collector, which runs parallel to the Subject Site's northern property line, will be required during removal of bedrock.

Refer to **Figure 3.1** – Proposed Servicing Layout Plan for an illustration of the proposed sanitary service, and existing sanitary sewers.

4.2 Sanitary Design Parameters

The peak design flow parameters in **Table 4.1** has been used in the sewer capacity analysis. Unit and population densities and all other design parameters are specified in the OSDG.

Table 4.1: Sanitary Sewer Design Parameters

Design Component	Design Parameter
Unit Population: Single Family Semi-detached/Row Townhome Average Apartment	 3.4 people/unit (used for existing) 2.7 people/unit (used for existing) 1.8 people/unit
Residential Flow Rate: Design	 280 L/cap/day
Residential Peaking Factor	Harmon Equation (min=2.0, max=4.0)
Harmon Correction Factor: Design	 0.8
Extraneous Flow Rate: Design	 0.33 L/s/ha
Minimum Pipe Size	200 mm (Res)
Minimum Velocity[1]	0.6 m/s
Maximum Velocity	3.0 m/s
Minimum Pipe Cover	2.5 m (Unless frost protection provided)

4.3 Assessment of Sanitary Infrastructure

Existing sanitary flows upstream of City of Ottawa sanitary manhole 47044 were analyzed to determine available capacity for additional flows from the proposed development. Two existing sanitary sewers enter manhole 47044, the 375mm diameter sewer from Roosevelt Avenue and the 300mm diameter sewer from Berkley Avenue. The analysis includes Roosevelt Street north of Byron Avenue, Danforth Avenue, Berkley Avenue, Dominion Avenue, Tay Street and a portion of Richmond Road. Based on the City of Ottawa as-built drawings and field investigation, it was confirmed that sanitary flows from Roosevelt Avenue south of Byron were directed into the Byron

Avenue sewer and they are not included in the analysis. Refer to the Sanitary Drainage Area Plan enclosed in **Appendix D** for reference.

Based on the above parameters, sanitary flows from the proposed development are calculated to be 6.83 L/s. Sanitary flows from the existing areas upstream of the 375mm diameter Roosevelt sewer are calculated to be 2.33 L/s. The total sanitary flows in the Roosevelt sewer will be approximately 9.16 L/s, while the capacity of this sewer (at 0.17% slope) is 75.40 L/s.

The existing sanitary flow upstream of the 300mm diameter sewer from Berkley Avenue was calculated using the above parameters for properties collected within the Tay Street, Dominion Avenue and Berkley Avenue sewers. Sanitary flows from Richmond road were taken from the 2003 Richmond Road rehabilitation projects design sheet. The combined calculated flow is 88.36 L/s. Refer to **Appendix D** for detailed calculations and Richmond Road design sheets.

Combining the wastewater flow entering City of Ottawa manhole 47044, the total flow conveyed through the existing 450mm diameter sewer connecting to the West Nepean Collector will be 95.19 L/s (6.83 L/s + 88.36 L/s). As-built information for the existing 450mm diameter sewer was not available at the City of Ottawa, particularly the downstream invert of the 450mm diameter sewer at West Nepean Collector. However, if concluded that the sewer is built at the minimum design slope of 0.2% the capacity of the sewer is 131.34 L/s, which exceeds the projected total flows (existing and proposed) to the 450mm diameter sewer discharging to the West Nepean Collector. Refer to **Appendix D** for detailed calculations.

5.0 WATER SUPPLY SYSTEM

5.1 Water Infrastructure

The proposed development will be serviced with two 150mm diameter watermains connected to the existing 150mm diameter watermain in Roosevelt Avenue, and a third 150mm diameter watermain connected to the existing 150mm diameter watermain in Wilmont Avenue (for redundancy).

As mentioned above, it is important to note that protection of the existing 1200mm diameter trunk watermain, which runs parallel to the Subject Site’s northern property line, will be required during removal of bedrock.

Refer to **Figure 3.1** – Proposed Servicing Layout Plan for an illustration of the proposed watermain services, and existing watermains.

5.2 Watermain Design Parameters and Demands

The domestic and fire fighting demand design paramters, and system pressure design criteria are outlined in **Table 5.1** below. Unit and population densities and all other design parameters and system pressure design criteria are specified in the OWDG. The system pressure design criteria are based on a conservative approach that considers three possible scenarios.

Table 5.1: Watermain Design Parameters and Criteria

Domestic Demand Design Parameters	Design Parameters
Unit Population: Average Apartment	 1.8 people/unit
Average Day Residential Demand (AVDY)	280 L/c/d
Maximum Day Demand (MXDY)	2.5 x AVDY
Peak Hour Demand (PKHR)	2.2 x MXDY
Fire Demand Design	**Design Flows**
Fire Demand (FF)	67 and 83 L/s per FUS / OWDG TB-2014
System Pressure Criteria Design Parameters	**Criteria**
Maximum Pressure (AVDY) Condition	< 80 psi occupied areas < 100 psi unoccupied areas
Minimum Pressure (PKHR) Condition	> 40 psi
Minimum Pressure (MXDY + FF) Condition	> 20 psi

5.2.1 Domestic Demands

Based on the above parameters, the theoretical water demands from the proposed development were calculated and are as follows:

- Population = 650 persons
- Average Day Demand = 2.11 L/s
- Maximum Day Demand = 5.26 L/s
- Peak Hour Demand = 11.58 L/s

Refer to **Appendix E** for water demand calculations.

5.2.2 Fire Demands

The required fire demand for the Subject Site was calculated using the Fire Underwriters Survey (FUS). The fire flow supply required was calculated to be 67 L/s, 83 L/s, and 67 L/s for Building #1, Building #2, and the low-rise buildings (worst case scenario), respectively.

Refer to **Appendix E** for a copy of the FUS fire flow calculations.

5.3 Assessment of Water Infrastructure

This water demand information was submitted to the City and boundary conditions provided from the City's water model. The boundary conditions were used to complete a simple hydraulic analysis to confirm the existing water infrastructure has capacity for the proposed development. The hydraulic analysis was completed to confirm that the existing water infrastructure will meet the required pressures in the average day and peak hour conditions under domestic use. Refer to **Table 5.2** for the results of the hydraulic analysis for the domestic demands.

Table 5.2 Domestic Demand Water Analysis Results Summary

Condition	Service Connection Location	Demand (L/s)	Min/Max Allowable Operating Pressures (psi)	Limits of Design Operating Pressures (psi)
Maximum Pressure (AVDY)	Roosevelt	2.11	80psi (Max)	58.5
Minimum Pressure (PKHR)	Roosevelt	11.58	40psi (Min)	67.9

Therefore, the existing watermain along Roosevelt Avenue can provide adequate pressures for domestic demands. Note that due to the size of the buildings, booster pumps will be required to provide adequate service pressure on the upper floor levels.

For fire fighting purposes, the proposed development is to be sprinklered with Siamese connections (locations of the connections will be determined during the detailed design stage). In addition to the Siamese connections, there are three existing fire hydrants in the vicinity of the site; located at 335 Roosevelt Avenue, 349 Winston Ave, and 364 Wilmont Avenue. Boundary conditions were requested for fire protection from the existing 150mm diameter watermain along Roosevelt Avenue and Winston Avenue. The City indicated that there is 167 L/s of available flow at Roosevelt Avenue and 65 L/s available flow at a pressure of 20 psi at Winston Avenue.

The fire flow required for the proposed development, as indicated previously, is 67 L/s and 83 L/s, depending on the asset. As such, the aggregate fire flow of all available fire hydrants within 150m of the site will be greater than the required fire flow.

Therefore, based on the boundary condition information provided by the City, the existing watermain infrastructure can provide adequate flow and pressure for domestic demand and fire protection for the proposed development. Refer to **Appendix E** for water demands, fire flow calculations, and boundary conditions.

Note that during detailed design, further coordination with the Mechanical or Fire Protection Engineer regarding the buildings internal water system will be required. At this time another request to the City for boundary conditions may be made.

6.0 UTILITIES

The development will be serviced by Hydro Ottawa, Bell Canada, Rogers Communications, and Enbridge Gas Distribution Inc. The utility servicing approach will be coordinated with local utility companies during the detailed design stage.

7.0 EROSION AND SEDIMENT CONTROL AND DEWATERING MEASURES

Temporary erosion and sediment control measures will be implemented during construction in accordance with the "Guidelines on Erosion and Sediment Control for Urban Construction Sites" (Government of Ontario, May 1987). Details will be provided on an Erosion and Sediment Control Plan, prepared during the detailed design stage. Erosion and sediment control measures may include:

- Placement of filter fabric under all catch basin and maintenance hatches;
- Tree protection fence around the trees to be maintained

- Silt fence around the area under construction placed as per OPSS 577 / OPSD 219.110
- Light duty straw bale check dam per OPSD 219.180

The erosion and sediment control measures will need to be installed to the satisfaction of the engineer, the City, the Ontario Ministry of Environment Conservation and Parks (MECP), and the RVCA, prior to construction and will remain in place during construction until vegetation is established. The erosion and sediment control measure will also be subject to regular inspection to ensure that measures are operational.

Prior to construction, a Permit-To-Take-Water (PTTW) or Environmental Activity and Sector Registry (EASR) application will be submitted to the Ministry of the Environment, Conservation and Parks (MECP). The permit will outline the water taking quantity, and location/quality of the discharge.

8.0 MISSISSIPPI-RIDEAU SOURCE PROTECTION PLAN

The Mississippi-Rideau Source Protection Plan has been implemented in order to oversee the source protection program in the Mississippi-Rideau Source Protection Region, in which the proposed development is located. Please refer to the Source Protection figures provided in **Appendix F** and the Source Protection policy screening correspondence provided in **Appendix A**. Although the location of the Subject Site is within the Surface Water Intake Protection Zone for the Ottawa River (Britannia) Intake and the Highly Vulnerable Aquifer source protection areas, the proposed development is not considered to cause a significant drinking water threat.

9.0 NEXT STEPS, COORDINATION, AND APPROVALS

The proposed municipal infrastructure may be subject, but not limited to the following approvals:

- Site Plan Control Application. Submitted to: City of Ottawa. Proponent: Developer.
- MECP PTTW/EASR. Submitted to: MECP. Proponent: Developer.
- Road Cut Permit. Submitted to: City of Ottawa. Proponent: Developer, or its contractor/agent.

10.0 SUMMARY AND CONCLUSIONS

This report demonstrates that the proposed development can be adequately serviced with storm and sanitary sewers and watermain. The report is summarized below:

Stormwater Management:

- The proposed development will be serviced with a 250mm diameter storm service connected to the existing 450mm diameter storm sewer in Wilmont Avenue. The existing storm sewers have adequate capacity to service the proposed development.
- Stormwater management will be provided to adhere to the allowable release rates. Quantity control will be achieved via rooftop storage, surface storage, and an underground storage system. Quality control is not required.

Sanitary and Wastewater Collection System:

- The proposed development will be serviced with a 200mm diameter sanitary service connected to the existing 375mm diameter sanitary sewer in Roosevelt Avenue. The existing sanitary sewers have adequate capacity to service the proposed development.

Water Supply System

- The proposed development will be serviced with two 150mm diameter watermains connected to the existing 150mm diameter watermain in Roosevelt Avenue, and a third 150mm diameter watermain connected to the existing 150mm diameter watermain in Wilmont Avenue (for redundancy).
- The existing water supply system has adequate capacity to meet system pressure for the Subject Site's domestic and fire demands.
- Fire fighting protection will be achieved by proximity to existing fire hydrants, an automated sprinkler system, and the Siamese connections.

Erosion and Sediment Control

- Temporary erosion and sediment control measures will be implemented both prior to commencement and during construction in accordance with the "Guidelines on Erosion and Sediment Control for Urban Construction Sites" (Government of Ontario, May 1987).

11.0 CLOSURE

This report is respectfully submitted for review and subsequent approval. Please contact the undersigned should you have questions or require additional information.

NOVATECH

Prepared by:

Ben Sweet, P.Eng.
Project Coordinator I Land Development

Reviewed by:

Bassam Bahia, M.Eng., P.Eng.
Project Manager | Land Development

Appendix A
Correspondence

Pre-Consul Meeting Notes to the File Lead - Steve Gauthier July 9, 2020
Re: 335 Roosevelt Avenue
Ward 15 - Kitchesippi, Councillor Jeff Leiper

For **Zoning applications**, please provide Adequacy of Servicing for the site, demonstrating that the site can be appropriately serviced and is able to achieve SWM requirements, as per City Guidelines as well as City Policies, Standard Detail Drawings and By-Laws and note the following for SPC applications:

Infrastructure:

Roosevelt Ave.

A 152mm dia. UCI Watermain (c. 1931) is available.

A 450 mm dia. Conc. Sanitary Sewer (c. 1930) is available which drains to Scott St. Trunk and connects to the Interceptor Sewer.

A 450 mm dia. Conc. Storm Sewer (c. ?) is available which drains to Dominion Storm and Outlets to the Ottawa River at Sir John A. MacDonald Pkwy.

Wilmont Ave.

A 152mm dia. UCI Watermain (c. 1931) is available.

A 225mm dia. Conc. Sanitary Sewer (c. 1932) is available which drains to Scott St. Trunk and connects to the Interceptor Sewer.

A 450mm dia. Conc. Storm Sewer (c. 1989) is available which drains to the West Transit Storm and Outlets to the Ottawa River near Onigam St.

The following apply to this site and any development within a separated sewer area:

- Total (San & Stm) allowable release rate will be 5-year pre-development rate.
- Coefficient (C) of runoff will need to be determined **as per existing conditions** but in no case more than 0.5
- TC = 20 minutes or can be calculated
  TC should be not be less than 10 minutes, since IDF curves become unrealistic at less than 10 min.

- Any storm events greater than 5 year, up to 100 year, and including 100-year storm event must be detained on site.
- Two separate sewer laterals (one for sanitary and other for storm) will be required.

Please note:
Foundation drains are to be independently connected to sewermain (separated or combined) unless being pumped with appropriate back up power, sufficient sized pump and back flow prevention.

Roof drains are to be connected downstream of any incorporated ICD within the SWM system. Provide Roof plan showing roof drain and scupper locations, flow rates, drain type and weir opening if controlled. Provide Manufacturer Specifications on drains and also provide 5- and 100-year ponding limits on plan.

Boundary Conditions will be provided at request of consultant after providing Average Daily Demands, Peak Hour Demands & Max Day + Fire Flow Demands

If window wells are proposed, they are to be indirectly connected to the footing drains.
A detail of window well with indirect connection is required, as is a note at window well location speaking to indirect connection.

Note:

If applicable, existing buildings require a CCTV inspection and report to ensure existing services to be re-used are in good working order and meet current minimum size requirements.
Located services to be placed on site servicing plans.

CCTV Scan Guideline.pdf

Other:

Environmental Noise Study is required due proximity of Transitway.

Stationary Noise Study – consultant to speak to this in their report as per City NCG and NPC 300 Guidelines. May be required after Mechanical Design completed and prior to building permit issuance.

When greater than 9 metres in height, a Shadow Study required for all buildings/dwellings.

When greater than 9 metres in height Wind Study for all buildings/dwellings.

Capital Works:
Road and Sewer Renewal project planned for Wilmont St. within the next 3-5 years.
New transit way (LRT) planned to start this season.
Road and Sewer Renewal project planned for Winona Ave. within the next 3-5 years.

Water Supply Redundancy – Fire Flow:

> Applicant to ensure that a second service with an inline valve chamber be provided where the average daily demand exceeds 50 m³ / day (0.5787 l/s per day)
> FUS Fire Flow Criteria to be used unless a low-rise building, where OBC requirements may be applicable.

Vibration monitoring will be required for all backbone watermains (1220 mm dia.) and trunk sewers (1500 mm dia.) in proximity of site.

CCTV sewer inspection required for pre and post construction conditions to ensure no damage to City Assets surrounding site. See Transit Way, Roosevelt and Wilmont Avenues.

Pre-Construction (Piling/Hoe Ramming) and/or Pre-Blasting (if applicable) Survey required for any occupants of buildings/dwellings in proximity of 75m of site and circulation of notice of vibration/noise to residents within 150 m of site.

Source Protection Policy Screening (SPPS):
SPPS will be provided to applicant by City Risk Mgmt. Officer within Asset Mgmt. Dept.

Due to proximity of site to Transit Way and Dominion Station, applicant to contact City LRT Group in regard to required building offset from transitway. Noise study to review vibration conditions within 75m of Transitway. See Rail Guidelines and CPCS Report as well as OP Annex 17 – Zone of Influence.

2013_05_29_Guidelines_NewDevelopment_E

CPCS Report Appendix_F.pdf

Applicant to contact Rideau Valley Conservation Authority (RVCA) for possible restrictions due to quality control. Provide correspondence in Report.

Where underground storage (UG) and surface ponding are being considered:

Show all ponding for 5- and 100-year events

Above and below ground storage is permitted although uses ½ Peak Flow Rate or is modeled. Please confirm that this has been accounted for and/or revise.

Rationale:

The Modified Rational Method for storage computation in the Sewer Design Guidelines was originally intended to be used for above ground storage (i.e. parking lot) where the change in head over the orifice varied from 1.5 m to 1.2 m (assuming a 1.2 m deep CB and a max ponding depth of 0.3 m). This change in head was small and hence the release rate fluctuated little, therefore there was no need to use an average release rate.

When underground storage is used, the release rate fluctuates from a maximum peak flow based on maximum head down to a release rate of zero. This difference is large and has a significant impact on storage requirements. We therefore require that an average release rate be used to estimate the required volume. Alternatively, the consultant may choose to use a submersible pump in the design to ensure a constant release rate.

In the event that there is a disagreement from the designer regarding the required storage, The City will require that the designer demonstrate their rationale utilizing dynamic modelling, that will then be reviewed by City modellers in the Water Resources Group.

Note that the above will added to upcoming revised Sewer Design Guidelines to account for underground storage, which is now widely used.

Further to above, what will be the actual underground storage provided during the major (100 year) and minor (2 year) storm events?

Please provide information on UG storage pipe. Provide required cover over pipe and details, chart of storage values, capacity etc. How will this pipe be cleaned of sediment and debris?

Note - There must be at least 15cm of vertical clearance between the spill elevation and the ground elevation at the building envelope that is in proximity of the flow route or ponding area. The exception in this case would be at reverse sloped loading dock locations. At these locations, a minimum of 15cm of vertical clearance must be provided below loading dock openings. Ensure to provide discussion in report and ensure grading plan matches if applicable.

Provide information on type of underground storage system including product name and model, number of chambers, chamber configuration, confirm invert of chamber system, top of chamber system, required cover over system and details, interior bottom slope (for self-cleansing), chart of storage values, length, width and height, capacity, entry ports (maintenance) etc.

Provide a cross section of underground chamber system showing invert and obvert/top, major and minor HWLs, top of ground, system volume provided during major and minor events. UG storage to provide actual 2- and 100-year event storage requirements.

In regard to all proposed UG storage, ground water levels (and in particular HGW levels) will need to be reviewed to ensure that the proposed system does not become surcharged and thereby ineffective.

Modeling can be provided to ensure capacity for both storm and sanitary sewers for the proposed development by City's Water Distribution Dept. – Modeling Group, through PM and upon request.

For proposed depressed driveways or developments with private lanes, parking areas or with entrances etc. lower than roadway…

S18.pdf   S18.1.pdf

Rear yard on grade parking to be permeable pavement. Refer to City Standard Detail Drawings SC26 (maintenance/temp parking areas), SC27 or permeable asphalt materials. No gravel or stone dust parking areas permitted.

Note:

"Provided Info to applicant":

Please be advised that it is the responsibility of the applicant and their representatives/consultants to verify information provided by the City of Ottawa.

Please contact City View and Release Info Centre at Ext. 44455

Environmental Source Information:

Due to more sensitive use, a Record of Site Condition (RSC) is required. Ensure Phase I, and if applicable, Phase II ESA's speak to required RSC.

Please also note that in the event soil and/or groundwater contamination is identified on this site and the proposal is for a more sensitive land use, the MECP will require approximately 1-1.5 years to review the RSC.
PIED will apply appropriate conditions, based on Environmental Protection Act (Section 168.3.1 (1)) and O.Reg. 153/04 (Parts IV and V) regarding requirements for RSC prior to building permit issuance. Dependent on the levels/types of contamination, timelines for building permit issuance may be longer than expected and we recommend applicant speak to Building Code Services, at the earliest convenience, so as to discuss these timelines in more detail, if deemed applicable.

RSC is required prior to building permit issuance, not occupancy. No exceptions.

City of Ottawa - Historical Land Use Inventory (HLUI) - Required

Rationale:
The HLUI database is currently undergoing an update. The updated HLUI will include additional sources beyond those included in the current database, making the inclusion of this record search even more important.
Although a municipal historic land use database is not specifically listed as required environmental record in O. Reg 153/04, Schedule D, Part II states the following:

The following are the specific objectives of a records review:

1. To obtain and review records that relate to the Phase I (One) property and to the current and past uses of and activities at or affecting the Phase I (One) property in order to determine if an area of potential environmental concern exists and to interpret any area of potential environmental concern.

2. To obtain and review records that relate to properties in the Phase I (One) study area other than the Phase I (One) property, in order to determine if an area of potential environmental concern exists and to interpret any area of potential environmental concern.

It is therefore reasonable to request that the HLUI search be included in the Phase I ESA to meet the above objectives.
Please submit.

All existing reports and plans will need to be revised if older than 2 years and must reflect current City Standards, Guidelines, By-laws and Policies.

Please refer to City of Ottawa website portal **for "Guide to preparing Studies and Plans"** at https://ottawa.ca/en/city-hall/planning-and-development/information-developers/development-application-review-process/development-application-submission/guide-preparing-studies-and-plans.

Please ensure you are using the current guidelines, bylaws and standards including materials of construction, disinfection and all relevant reference to OPSS/D and AWWA guidelines - all current and as amended, such as:

City of Ottawa Sewer Design Guidelines (**CoOSDG**) complete with ISTDB 2012-01, 2014-01, 2016-01, 2018-01 & 2019-02 technical bulletin updates as well as current Sewer, Landscape & Road Standard Detail Drawings as well as Material Specifications (MS Docs).
Sewer Connection (2003-513) & Sewer Use (2003-514) By-Laws.

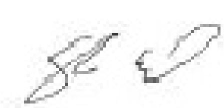
City of Ottawa Water Distribution Design Guidelines (**CoOWDDG**) complete with ISTDB 2010-02, 2014-02 & 2018-02 technical bulletin updates as well as current Watermain/ Services Material Specifications (MS Docs) as well as Water and Road Standard Detail Drawings.
FUS Fire Flow standards
Water (2018-167) By-Law

Ensure to include version date and add **"(as amended)"** when referencing all standards, detail drwaings, by-Laws and guidelines.

Fourth (4th) Review Charge:
Please be advised that additional charges for each review, after the 3rd review, will be applicable to each file. There will be no exceptions.

Contact me by e-mail shawn.wessel@ottawa.ca if you have any questions.

Sincerely,



Shawn Wessel, A.Sc.T., rcji
Project Manager
Development Review, Central Branch

Ben Sweet

From:	Gauthier, Steve <Steve.Gauthier@ottawa.ca>
Sent:	Tuesday, July 14, 2020 12:00 PM
To:	Jacob Bolduc
Subject:	FW: 335 Roosevelt AVe

FYI

From: Wessel, Shawn <shawn.wessel@ottawa.ca>
Sent: July 14, 2020 9:52 AM
To: Gauthier, Steve <Steve.Gauthier@ottawa.ca>
Subject: FW: 335 Roosevelt AVe

For the applicant
Thanks

If you require additional information or clarification, please do not hesitate to contact me anytime.

Thank you

Regards,

Shawn Wessel, A.Sc.T.,rcji
Project Manager - Infrastructure Approvals
Gestionnaire de projet – Approbation des demandes d'infrastructures

Development Review Central Branch | Direction de l'examen des projets d'aménagement, Centrale
Planning, Infrastructure and Economic Development Department | Direction générale de la planification de l'infrastructure et du développement économique
City of Ottawa | Ville d'Ottawa
110 Laurier Ave. W. | 110, avenue Laurier Ouest, Ottawa ON K1P 1J1
(613) 580 2424 Ext. | Poste 33017
Int. Mail Code | Code de Courrier Interne 01-14
shawn.wessel@ottawa.ca

*****Please note that, while my work hours may be affected by the current situation, I still have access to email, video conferencing and telephone. Feel free to schedule video conferences and/or telephone calls, as necessary.*****

From: Di Iorio, Tessa <tessa.diiorio@ottawa.ca>
Sent: July 13, 2020 11:04 AM
To: Wessel, Shawn <shawn.wessel@ottawa.ca>
Subject: RE: 335 Roosevelt AVe

Hello Shawn,

Thank you for contacting me for a Source Protection policy screening for the *Planning Act* application at **335 Roosevelt**.

Source Protection Policy Screening:

1. The address lies within the Mississippi-Rideau Source Protection Region and is subject to the policies of the Mississippi-Rideau Source Protection Plan.

2. The western portion of the property (west of Winston Avenue) lies within the Surface Water Intake Protection Zone for the Ottawa River (Britannia) Intake, IPZ-2 (vulnerability score of 8.1) where significant threat policies apply. Policies are only applicable for specific significant drinking water threat activities and policies are only applicable within the area identifies as IPZ-2 (vulnerability score 8.1).

   - The *Clean Water Act* Tables of Circumstances identify circumstances under which certain activities would be considered a significant threat to drinking water, and the Mississippi-Rideau Source Protection Plan contains policies related to significant drinking water threat activities to protect the drinking water supply.
   - Activities that may be considered a significant drinking water threat within the IPZ-2 (score 8.1) include the following:
     - Untreated stormwater from a stormwater retention pond
     - Sewage treatment plant effluent discharges
     - Combined sewer discharge from a stormwater outlet
     - Sewage treatment plant bypass discharge
     - Industrial effluent discharge
     - Waste disposal site
     - Agricultural activities (application or storage of manure or chemical fertilizers or pesticides, or use of land for livestock grazing)
   - If any of the above activities are proposed within the western portion of the property (west of Winston Avenue), then please follow up with me to determine if the activity meets the circumstance to be a significant drinking water threat.
   - If none of the activities listed above are proposed within the IPZ-2 (the western portion of the property), then there are no applicable Source Protection policies related to the IPZ-2.

3. The area is not within a Wellhead Protection Area (WHPA).

4. The area located within a Highly Vulnerable Aquifer (HVA). Note that there are no legally binding policies under the Mississippi-Rideau Source Protection Plan for activities within Highly Vulnerable Aquifers.

5. The area is not within a Significant Groundwater Recharge Area.

Please follow up with confirmation if the above highlighted activities are proposed within the IPZ-2 (western portion of the property, west of Winston Avenue).
And feel free to contact me directly if you have any questions.

Kind Regards,
Tessa

Tessa Di Iorio, M.Sc., P.Geo.
Risk Management Official/Inspector, Hydrogeologist
Infrastructure Services – Asset Management Branch
Planning, Infrastructure and Economic Development
City of Ottawa | Ville d'Ottawa
(613) 580-2424 ext./poste 17658
tessa.diiorio@ottawa.ca

Please note: Due to the current pandemic, I will be working from home until further notice. Contact by email is preferred; I will be checking my voicemail less frequently.

From: Wessel, Shawn <shawn.wessel@ottawa.ca>
Sent: July 10, 2020 1:52 PM
To: Di Iorio, Tessa <tessa.diiorio@ottawa.ca>
Subject: 335 Roosevelt AVe

Good afternoon Tessa

May I request Source Protection Screening for this site.

Have a nice weekend! ☺

If you require additional information or clarification, please do not hesitate to contact me anytime.

Thank you

Regards,

Shawn Wessel, A.Sc.T.,rcji
Project Manager - Infrastructure Approvals

Gestionnaire de projet – Approbation des demandes d'infrastructures

Development Review Central Branch | Direction de l'examen des projets d'aménagement, Centrale
Planning, Infrastructure and Economic Development Department | Direction générale de la planification de l'infrastructure et du développement économique
City of Ottawa | Ville d'Ottawa
110 Laurier Ave. W. | 110, avenue Laurier Ouest, Ottawa ON K1P 1J1
(613) 580 2424 Ext. | Poste 33017
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shawn.wessel@ottawa.ca

*****Please note that, while my work hours may be affected by the current situation, I still have access to email, video conferencing and telephone. Feel free to schedule video conferences and/or telephone calls, as necessary.*****

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Appendix B
Servicing Report Checklist

NOVATECH
Engineers, Planners & Landscape Architects

Development Servicing Study Checklist

Project Name: 335 Roosevelt Avenue
Project Number: 110098
Date: July 17, 2020

4.1 General Content	Addressed (Y/N/NA)	Section	Comments
Executive Summary (for larger reports only).	**NA**		
Date and revision number of the report.	Y	Cover	
Location map and plan showing municipal address, boundary, and layout of proposed development.	Y	Fig 1.1, Fig 1.3	
Plan showing the site and location of all existing services.	Y	Fig 3.1	
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.	NA		
Summary of Pre-consultation Meetings with City and other approval agencies.	Y	AppA	
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.	Y	3, 4, 5	
Statement of objectives and servicing criteria.	Y	1	
Identification of existing and proposed infrastructure available in the immediate area.	Y	3, 4, 5	
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).	Y	8	
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 neighboring properties. This is also required to confirm that the proposed grading will not impede existing major system flow paths.	N		To be provided during detailed design/site plan approval

Development Servicing Study Checklist

Project Name: 335 Roosevelt Avenue
Project Number: 110098
Date: July 17, 2020

4.1 General Content	**Addressed (Y/N/NA)**	**Section**	**Comments**
Identification of potential impacts of proposed piped services on private services (such as wells and septic fields on adjacent lands) and mitigation required to address potential impacts.	NA		
Proposed phasing of the development, if applicable.	NA		
Reference to geotechnical studies and recommendations concerning servicing.	Y	2	
All preliminary and formal site plan submissions should have the following information:			
Metric scale	Y		
North arrow (including construction North)	Y		
Key plan	Y		
Name and contact information of applicant and property owner	Y		
Property limits including bearings and dimensions	Y		
Existing and proposed structures and parking areas	Y		
Easements, road widening and rights-of-way	Y		
Adjacent street names	Y		

NOVATECH
Engineers, Planners & Landscape Architects

Development Servicing Study Checklist

Project Name: 335 Roosevelt Avenue
Project Number: 110098
Date: July 17, 2020

4.2 Water	Addressed (Y/N/NA)	Section	Comments
Confirm consistency with Master Servicing Study, if available.	Y	5	
Availability of public infrastructure to service proposed development.	Y	5	
Identification of system constraints.	Y	5	
Identify boundary conditions.	Y	5	
Confirmation of adequate domestic supply and pressure.	Y	5	
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.	Y	5	
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	5	
Definition of phasing constraints. Hydraulic modeling is required to confirm servicing for all defined phases of the project including the ultimate design.	N		To be provided during detailed design/site plan approval
Address reliability requirements such as appropriate location of shut-off valves.	Y	Fig 3.1	
Check on the necessity of a pressure zone boundary modification.	NA		
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.	Y	5	
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	5, Fig 3.1	
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	5	
Confirmation that water demands are calculated based on the City of Ottawa Design Guidelines.	Y	5	
Provision of a model schematic showing the boundary conditions locations, streets, parcels, and building locations for reference.	NA		

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	
Confirm consistency with Master Servicing Study and/or justifications for deviations.	Y	4	
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.	NA		
Description of existing sanitary sewer available for discharge of wastewater from proposed development.	Y	4	
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	
Calculations related to dry-weather and wet-weather flow rates from the development in standard MOE sanitary sewer design table (Appendix 'C') format.	N		
Description of proposed sewer network including sewers, pumping stations, and forcemains.	Y	4	
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).	NA		
Pumping stations: impacts of proposed development on existing pumping stations or requirements for new pumping station to service development.	NA		
Forcemain capacity in terms of operational redundancy, surge pressure and maximum flow velocity.	NA		
Identification and implementation of the emergency overflow from sanitary pumping stations in relation to the hydraulic grade line to protect against basement flooding.	NA		
Special considerations such as contamination, corrosive environment etc.	NA		

4.4 Stormwater	**Addressed (Y/N/NA)**	**Section**	**Comments**
Description of drainage outlets and downstream constraints including legality of outlet (i.e. municipal drain, right-of-way, watercourse, or private property).	Y	3	
Analysis of the available capacity in existing public infrastructure.	NA		
A drawing showing the subject lands, its surroundings, the receiving watercourse, existing drainage patterns and proposed drainage patterns.	N		To be provided during detailed design/site plan approval
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	3	
Water Quality control objective (basic, normal or enhanced level of protection based on the sensitivities of the receiving watercourse) and storage requirements.	Y	3	
Description of stormwater management concept with facility locations and descriptions with references and supporting information.	Y	3	
Set-back from private sewage disposal systems.	NA		
Watercourse and hazard lands setbacks.	NA		
Record of pre-consultation with the Ontario Ministry of Environment and the Conservation Authority that has jurisdiction on the affected watershed.	NA		
Confirm consistency with sub-watershed and Master Servicing Study, if applicable study exists.	NA		
Storage requirements (complete with calcs) and conveyance capacity for 5 yr and 100 yr events.	Y	3	
Identification of watercourse within the proposed development and how watercourses will be protected, or, if necessary, altered by the proposed development with applicable approvals.	Y	3	
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	3	
Any proposed diversion of drainage catchment areas from one outlet to another.	Y	3	
Proposed minor and major systems including locations and sizes of stormwater trunk sewers, and SWM facilities.	Y	3	
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.	NA		

4.4 Stormwater	**Addressed (Y/N/NA)**	**Section**	**Comments**
Identification of municipal drains and related approval requirements.	NA		
Description of how the conveyance and storage capacity will be achieved for the development.	Y	3	
100 year flood levels and major flow routing to protect proposed development from flooding for establishing minimum building elevations (MBE) and overall grading.	Y	3	
Inclusion of hydraulic analysis including HGL elevations.	NA		
Description of approach to erosion and sediment control during construction for the protection of receiving watercourse or drainage corridors.	Y	7	
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.	NA		
Identification of fill constrains related to floodplain and geotechnical investigation.	NA		

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.	NA		
Application for Certificate of Approval (CofA) under the Ontario Water Resources Act.	Y	9	
Changes to Municipal Drains.	NA		
Other permits (National Capital Commission, Parks Canada, Public Works and Government Services Canada, Ministry of Transportation etc.)	Y	9	

4.6 Conclusion	**Addressed (Y/N/NA)**	**Section**	**Comments**
Clearly stated conclusions and recommendations.	Y	10	
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.	NA		
All draft and final reports shall be signed and stamped by a professional Engineer registered in Ontario.	Y	11	

Appendix C
Storm Sewer Design Sheets and Stormwater Management Calculations

Ministry of the Environment
Ministère de l'Environnement
Ontario

Certificate of Approval (Sewage)
Certificat d'autorisation (eaux usées)

Number / *Numéro* 3-2058-88-006

Whereas / *Attendu que* CITY OF OTTAWA

XXXX

has applied in accordance with Section 24 of the Ontario Water Resources Act for approval of:
a fait, conformément à l'article 24 de la loi sur les ressources en eau de l'Ontario, une demande d'autorisation:
sewers and appurtenances to be constructed in the City of Ottawa, as follows:

Street	From	To
Storm Sewers		
Roosevelt Avenue	Richmond Road	Approx. 290m north of Richmond Road
Winston Avenue	Approx. 35m north of Richmond Road	Wilmont Avenue
Wilmont Avenue	Winston Avenue	Churchill Avenue
Churchill Avenue	Wilmont Avenue	Scott Street
Scott Street	Churchill Avenue	Winona Avenue
Easement (Roosevelt Avenue)	Approx. 290m north of Richmond Road	Approx. 145m west to Dominion Avenue

including stub sewer connections and building sewers from the main sewer to the street line, all in accordance with the plans prepared by Oliver, Mangione, McCalla & Associates Ltd., Consulting Engineers, at a total estimated cost, including engineering and contingencies, of TWO HUNDRED AND FORTY FIVE THOUSAND DOLLARS ($245,000.00). —

237177

CITY OF OTTAWA
DEPARTMENT OF ENGINEERING & WORKS
Rec'd NOV 1 1988
TO: ENG
FILE NO.: EW-1530-27/R 99 (88-52)
FILE NO.:

1153
2174

Now therefore this is to certify that after due enquiry the said proposed works have been approved under Section 24 of the Ontario Water Resources Act.
Le présent document *certifie qu'après vérification en bonne et due forme la construction dudit projet d'ouvrages a été approuvée aux termes de l'article 24 de la loi sur les ressources en eau de l'Ontario.*

DEPT. of CITY CLERK
Record ... / Dépôt des documents
File no. / No du dossier
31 10 88

DATED AT TORONTO this 25th day of October, 1988
DATÉ À TORONTO ce *jour d*

Attn: J.R. Cyr, Clerk, City of Ottawa
cc: Ms. G. Brown, Clerk, R.M. of Ottawa-Carleton
D. Guscott, MOE SE, Reg. Dir.
Oliver, Mangione, McCalla & Assoc. Ltd.

Description of Works
Description des ouvrages
Application is hereby made to the Director for
Le demandeur adresse au directeur par la présente une demande d'autorisation

City of Ottawa
Department of Engineering & Works

Approval to Construct (Describe type of sewers, pumping stations and miscellaneous structures.)
de construire (décrire le type d'égouts, de postes de pompage et d'ouvrages divers).

Storm sewers

And Sewage Treatment Works (Describe type and capacity of major works.)
ainsi que les ouvrages d'épuration des eaux usées suivants (décrire le type et la capacité des principaux ouvrages).

Location of Proposed Sewage Works
Emplacement des ouvrages
Lot, Concession, Municipality & County, District or Region
Lot, concession, municipalité et comté, district ou région

Roosevelt Avenue, Winston Avenue, Wilmont Avenue
Churchill Avenue & Scott Street, City of Ottawa,
Regional Municipality of Ottawa-Carleton

Works will Outlet to (Sewer system, name of receiving stream or lake.)
Les eaux traitées se déverseront dans (réseau d'égouts ou nom du cours d'eau ou du lac récepteur).
1) existing 675 mm dia. storm sewer on Dominion Avenue
2) existing 900 mm dia. storm sewer on Scott Street

This application is made under the provisions of Section 24, Ontario Water Resources Act, R.S.O. 1980, and such other statutes as relate to sewage works.

The applicant agrees that no changes in or deviations from the approved plans and specifications will be made except with the consent and approval of the Director, and agrees, if requested, to submit as-built drawings and cost figures to the Director upon completion of the project.

La présente demande est faite aux termes des dispositions de l'article 24 de la Loi sur les ressources en eau de l'Ontario, L.R.O. de 1980, et des autres lois qui se rapportent aux ouvrages d'adduction et de purification de l'eau.

Le demander s'engage à n'apporter aucune modification aux plans et cahier des charges approuvés, sauf s'il obtient le consentement et l'autorisation du directeur, et s'engage, sur demande, à remettre les plans des ouvrages tels qu'ils ont été construits ainsi que la ventilation détaillée du coût de construction au directeur à la fin des travaux.

Signatures Required
Signatures requises

Applicant / *Demandeur*

Signature / *Signature*: H.W. Pasen

Name (Print or Type) / *Nom (en lettres moulées)*: Corporation of the City Ottawa

Date / *Date*: Sept. 27/88

Mailing Address / *Adresse*: 1355 Bank Street
Ottawa, Ontario
K1H 8K7

Telephone / *N° de téléphone*: 564-1858

Municipality (If not applicant) / *Municipalité (À remplir si le demandeur n'est pas la municipalité.)*

Signature / *Signature*

Name & Title of Municipal Authority / *Nom et titre du responsable municipal*

Date / *Date*

Mailing Address / *Adresse*

Telephone / *N° de téléphone*

Engineer / *Ingénieur*

Eng. Documents Certified by Signature / *Préparation des documents d'ingénierie certifiés par la signature de l'ingénieur autorisé*

J. A. Sawatka

Name of Engineer or Firm / *Nom de l'ingénieur ou de la firme d'ingénierie*: Oliver, Mangione, McCalla & Assoc. Ltd.

Date / *Date*: Sept. 1988

Mailing Address / *Adresse*: 154 Colonnade Road South
Nepean, Ontario
K2E 7J5

Telephone / *N° de téléphone*: 225-9940

REGISTERED PROFESSIONAL ENGINEER PROVINCE OF ONTARIO

Operating Authority (If not applicant) / *Exploitant (À remplir si l'exploitant n'est pas le demandeur.)*

Signature / *Signature*

Name of Operating Authority / *Nom de l'exploitant*

Date / *Date*

Mailing Address / *Adresse*

Telephone / *N° de téléphone*

Cost Summary
Sommaire des coûts

Sewers and Appurtenances *Égouts et accessoires*	$	225,000.
Building Sewer Connections *Raccords de branchements d'égouts*	$	---
Pumping Stations and Forcemains *Postes de pompage et conduites de refoulement*	$	---
Treatment Works and Outfalls *Usines d'épuration et exutoires*	$	---
Engineering and Contingencies *Ingénierie et imprévus*	$	20,000.
Land Charges *Frais fonciers*	$	---
Total ***Total***	$	245,000.

Financing
Financement
Payment by (cash, debentures, loans, etc.)
Paiement (comptant, débentures, emprunts, etc.)

cash

Source of Financing (municipal, private, government)
Source de financement (municipal, privé, gouvernemental)

municipal

Scheduling
Calendrier
Construction Start Date
Date de début des travaux

as soon as possible

Construction Period (years, months)
Durée des travaux (années, mois)

one (1) month

File Number of Ministry of Municipal Affairs
Numéro de dossier du ministère des Affaires municipales

or Registered Plan Number (if applicable)
ou numéro de plan enregistré (s'il y a lieu)

N/A

The certificate of approval will be issued to the applicant. Copies will be sent to the clerks of any affected municipalities which are not applicants. List names and addresses below for any other recipients.

Le certificat d'autorisation sera délivré au demandeur et une copie du certificat envoyée aux secrétaires de toutes les municipalités intéressées qui n'ont pas signé la demande. Indiquer ci-dessous les noms et adresses de tout autre destinataire.

Oliver, Mangione, McCalla & Associates Limited
154 Colonnade Road South
Nepean, Ontario
K2E 7J5

Ministry Use Only
Réservé au ministère
Application Checked by
Demande vérifiée par

Application Recommended for Approval
Autorisation de la demande recommandée
Supervisor, Environmental Approvals Section
Superviseur, Section des approbations environnementales
Date
Date

JNS

Ministry of the Environment | Ministère de l'Environnement

Ontario

Certificate of Approval (Sewage)
Certificat d'autorisation (eaux usées)

Number / Numéro 3-2058-88-006

OLIVER MANGIONE McCALLA & ASSOCIATES LIMITED
OCT 31 1988
RECEIVED

#651C

Whereas / *Attendu que* CITY OF OTTAWA

XXXX

has applied in accordance with Section 24 of the Ontario Water Resources Act for approval of:
a fait, conformément à l'article 24 de la loi sur les ressources en eau de l'Ontario, une demande d'autorisation:
sewers and appurtenances to be constructed in the City of Ottawa, as follows:

Street	From	To
Storm Sewers		
Roosevelt Avenue	Richmond Road	Approx. 290m north of Richmond Road
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Wilmont Avenue	Winston Avenue	Churchill Avenue
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Easement (Roosevelt Avenue)	Approx. 290m north of Richmond Road	Approx. 145m west to Dominion Avenue

including stub sewer connections and building sewers from the main sewer to the street line, all in accordance with the plans prepared by Oliver, Mangione, McCalla & Associates Ltd., Consulting Engineers, at a total estimated cost, including engineering and contingencies, of TWO HUNDRED AND FORTY FIVE THOUSAND DOLLARS ($245,000.00).

THIS IS A TRUE COPY OF THE ORIGINAL CERTIFICATE MAILED ON ....OCT 26 1988............

(Signed)

Now therefore this is to certify that after due enquiry the said proposed works have been approved under Section 24 of the Ontario Water Resources Act.
Le présent document certifie qu'après vérification en bonne et due forme la construction dudit projet d'ouvrages a été approuvée aux termes de l'article 24 de la loi sur les ressources en eau de l'Ontario.

DATED AT TORONTO this / ***DATÉ À TORONTO le*** 25th **day of** / ***jour d*** October, 1988

Attn: D.R. Cyr, Clerk, City of Ottawa
cc: ... G. Brown, Clerk, R.M. of Ottawa-Carleton
P. Guscott, MOE SE, Reg. Dir.
Oliver, Mangione, McCalla & Assoc. Ltd.

OLIVER, MANGIONE, McCALLA & ASSOCIATES LIMITED
STORM SEWER COMPUTATION SHEET

JOB: City of Ottawa - Winston Avenue to Winona Avenue Storm Sewer
PROJECT:No: 2179 Contract No: 88-52

Ottawa Rainfall Curves
5 Year Frequency
$Q = 2.78\ RIA$

MANHOLE		AREA HECTARES		RUNOFF COEFFICIENT			FLOW TIME (MIN.)				PROFILE				MISC.
From	To	Increment	A ha Total	Increment	Adjustment of Coefficient	I Adjusted	To Upper End	In Section	R Rainfall mm/hr	Q Flow ℓ/sec	Pipe Dia. mm	Grade %	Velocity m/sec	Length m	Capacity l/sec.
WINSTON AVENUE															
[illegible]	MH2		0.40			0.45	20	1.09	66.29	33.17	300	0.4	.87	57.0	63.8
[illegible]	MH3	0.50	0.90			0.45	21.09	1.24	64.02	72.08	375	0.4	1.01	75.0	115.7
[illegible]	MH4	0.50	1.40			0.45	22.33	1.37	61.71	108.08	375	0.4	1.01	83.0	115.7
WILMONT STREET															
[illegible]	MH5	0.65	2.05			0.45	23.70	1.15	59.92	153.67	450	0.3	0.99	68.5	162.9
[illegible]	MH6	0.65	2.70			0.45	24.85	1.09	58.34	197.06	450	0.45	1.22	80	199.5

REGISTERED PROFESSIONAL ENGINEER
J. N. Sawarna
PROVINCE OF ONTARIO

OLIVER, MANGIONE, McCALLA & ASSOCIATES LIMITED

STORM SEWER COMPUTATION SHEET

[illegible]: City of Ottawa - Winston Avenue to Winona Avenue Storm Sewer

[illegible] No: 2179 Contract No: 88-52

Ottawa Rainfall Curves
5 Year Frequency
Q = 2.78 RIA

MANHOLE		AREA HECTARES		RUNOFF COEFFICIENT			FLOW TIME (MIN.)				PROFILE				MISC.
From	To	Increment	A ha Total	Increment	Adjustment of Coefficient	I Adjusted	To Upper End	In Section	R Rainfall mm/hr	Q Flow ℓ/sec	Pipe Dia. mm	Grade %	Velocity m/sec	Length m	Capacity l/sec.
[illegible] STREET															
[illegible]	MH7	0.55	3.25			0.45	25.94	1.15	56.77	230.81	525	0.30	1.10	76.0	245.8
[illegible] HILL AVENUE															
[illegible]	MH8	4.70	7.95			0.45	27.09	0.49	56.13	558.24	600	1.30	2.50	73.5	730.4
[illegible] STREET															
[illegible]	MH9	0.50	8.45			0.45	27.58	0.62	55.34	585.00	600	0.90	2.08	77.5	607.7
[illegible]	Exist. MH @ Winona	-	8.45			0.45	28.20	0.05	55.28	584.36	600	2.00	3.10	10.0	905.90

REGISTERED PROFESSIONAL ENGINEER
J. N. Sawarna
PROVINCE OF ONTARIO

STORM SEWER COMPUTATION SHEET

JOB: City of Ottawa - Roosevelt Avenue to Dominion Avenue Storm Sewer

PROJECT:No: 2179 Contract No: 88-52

Ottawa Rainfall Curves
5 Year Frequency
Q = 2.78 RIA

MANHOLE		AREA HECTARES		RUNOFF COEFFICIENT			FLOW TIME (MIN.)				PROFILE				MISC.
From	To	Increment	A ha Total	Increment	Adjustment of Coefficient	Adjusted I	To Upper End	In Section	R Rainfall mm/hr	Q Flow ℓ/sec	Pipe Dia. mm	Grade %	Velocity m/sec	Length m	Capacity l/sec.
ROOSEVELT AVENUE															
MH1	MH2		0.70			0.45	20	1.82	64.93	56.86	300	0.40	0.87	95	63.80
MH2	MH3	0.80	1.50			0.45	21.82	1.57	63.91	119.93	375	0.44	1.06	100	121.30
MH3	MH4	0.80	2.30			0.45	23.39	1.42	59.98	172.58	450	0.40	1.15	98	188.1
PARK															
MH4	MH5	0.10	2.40			0.45	25.81	1.20	58.24	174.86	450	0.40	1.15	82.5	188.1
MH5	MH6 outlet @ Dominion Avenue	3.10	5.50			0.45	26.01	0.79	57.15	393.22	600	0.40	1.39	66.0	405.1

J. N. Sawarna
REGISTERED PROFESSIONAL ENGINEER
PROVINCE OF ONTARIO

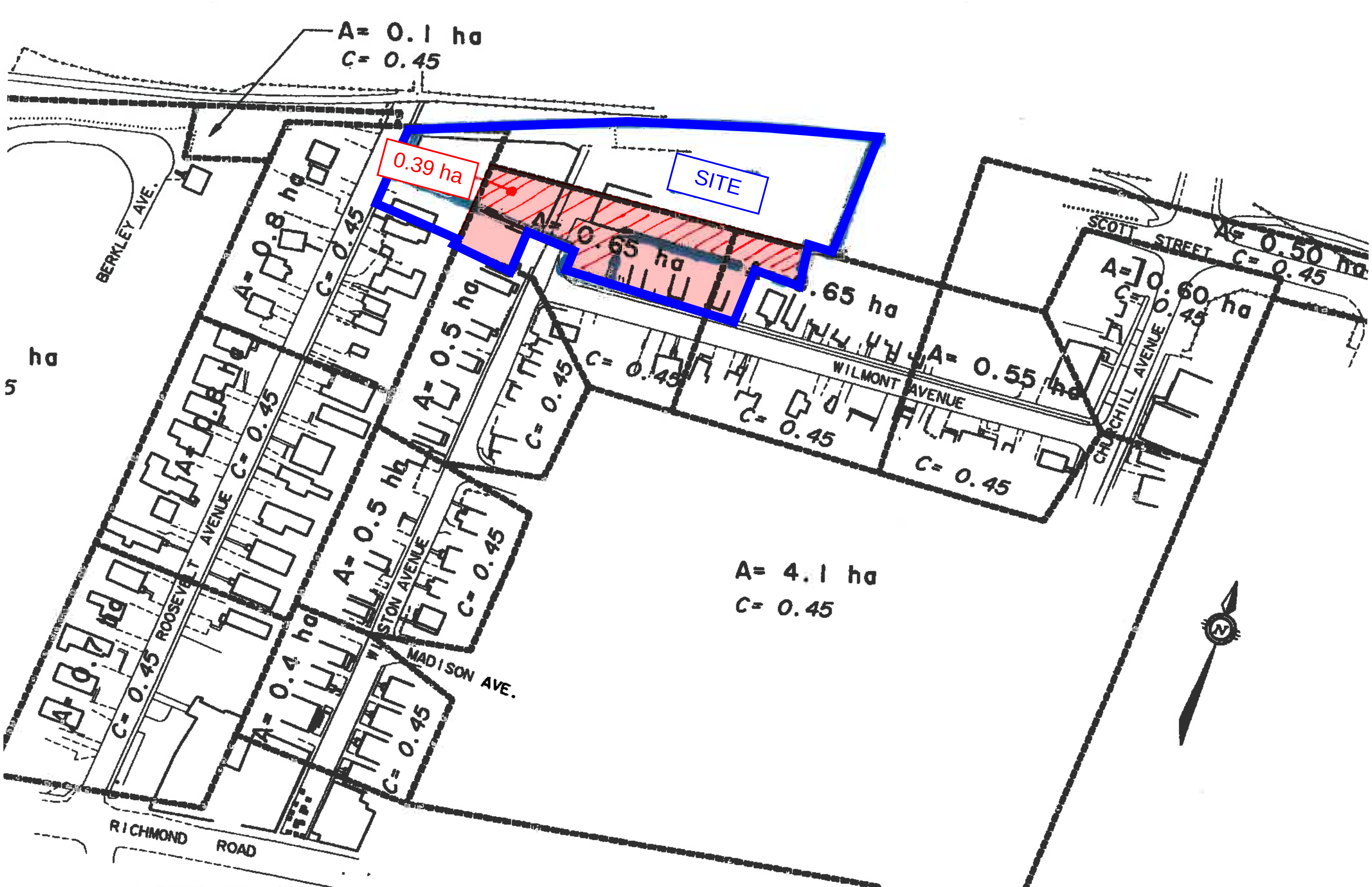


FIGURE 4 STORM DRAINAGE AREA PLAN

335 ROOSEVELT STREET
WILLMONT AVENUE STORM SEWER: HYDRAULIC GRADE LINE ANALYSIS (5-YEAR EVENT)
ORIGINAL DESING + SITE@ C=0.20

This spreadsheet uses the Darcy-Weisbach equation to calculate hydraulic losses through a pipe network with a specified flow rate. Minor losses are accounted for including both pipe bend losses and structure losses.
The spreadsheet returns the upstream hydraulic grade line if surcharged, or the pipe obvert if free flow conditions exist. The slope of the HGL is calculated and the minimum USF elevations can be established +0.30m above the HGL.

LOCATION	MANHOLE		INVERT ELEVATION		GROUND ELEVATION	COVER	PIPE PARAMETERS			TOTAL FLOW	Q_{cap}	Q_{in}/Q_{cap}	COMPUTATIONAL COLUMNS					HEAD LOSS	SURCHARGE	HGL			PIPE SLOPE	MIN. USF ELEVATION
	Upstream	Downstream	U/S (m)	D/S (m)	Upstream (m)	Upstream (m)	Dia (mm)	Length (m)	'n'	(m^3/s)	(m^3/s)		Pipe Area (m^2)	L/D	Friction Factor (f)	Velocity V (m/s)	$V^2/2g$	HL (m)	Upstream (m)	U/S (m)	D/S (m)	SLOPE (%)	(%)	Upstream (m)
																				63.10	<- OUTLET TO RICHMOND SEWER			
WILMONT	28522	28523	62.84	62.58	66.23	2.865	525	84.50	0.013	0.235	0.249	0.94	0.223	161	0.02610	1.05	0.06	0.26	0.00	63.37	63.11	0.31	0.31	63.67
	28521	28522	63.27	62.91	66.56	2.840	450	78.60	0.013	0.201	0.201	1.00	0.164	175	0.02747	1.22	0.08	0.39	0.03	63.75	63.37	0.49	0.46	64.05
	28520	28521	63.54	63.28	66.59	2.600	450	68.70	0.013	0.159	0.183	0.87	0.164	153	0.02747	0.97	0.05	0.29	0.05	64.04	63.75	0.42	0.38	64.34
WINSTON	28519	28520	63.86	63.62	66.89	2.455	375	81.60	0.013	0.108	0.099	1.09	0.114	218	0.02920	0.95	0.05	0.31	0.11	64.34	64.04	0.37	0.29	64.64
	28518	28519	64.20	63.91	66.81	2.235	375	76.60	0.013	0.072	0.113	0.64	0.114	204	0.02920	0.63	0.02	0.13	0.00	64.58	64.34	0.30	0.38	64.88
	28517	28518	65.86	64.23	68.45	2.290	300	57.90	0.013	0.033	0.169	0.19	0.073	193	0.03145	0.45	0.01	0.07	0.00	66.16	64.58	2.74	2.82	66.46

DESIGN PARAMETERS

DOWNSTREAM WATER LEVEL at Outlet = 67.57m (EPA SWMM MAX HGL)
RETURN FREQUENCY = 5 YEAR
MINIMUM VELOCITY= 0.80 m/s
MANNING'S n= 0.013
MIN. HGL CLEARANCE 0.30m

HGL=Major + Minor Losses
Major Loss= Pipe Friction (Darcy-Weisbach)
Minor Loss= Head loss correction for flow through MH, changes in pipe size, and pipe bends
Friction Factor= 8g/c^2, where c=(1/n)*(D/4)^1/6

Designed: MS
Checked: RSC
Dwg. Reference:

PROJECT:
335 ROOSEVELT AVENUE

CLIENT:
UNIFORM URBAN DEVELOPMENTS

Date: June 15, 2011

M:\2003\103106\Data\Calculations\110098_Wilmont HGL.xls\Orig. Design+ Site @0.20.xls

335 ROOSEVELT STREET
WILLMONT AVENUE STORM SEWER: HYDRAULIC GRADE LINE ANALYSIS (5-YEAR EVENT)
ORIGINAL DESING + SITE @ C=0.45

This spreadsheet uses the Darcy-Weisbach equation to calculate hydraulic losses through a pipe network with a specified flow rate. Minor losses are accounted for including both pipe bend losses and structure losses.
The spreadsheet returns the upstream hydraulic grade line if surcharged, or the pipe obvert if free flow conditions exist. The slope of the HGL is calculated and the minimum USF elevations can be established +0.30m above the HGL.

LOCATION	MANHOLE		INVERT ELEVATION		GROUND ELEVATION	COVER	PIPE PARAMETERS			TOTAL FLOW	Q_{cap}	Q_{in}/Q_{cap}	COMPUTATIONAL COLUMNS					HEAD LOSS	SURCHARGE	HGL			PIPE SLOPE	MIN. USF ELEVATION
	Upstream	Downstream	U/S (m)	D/S (m)	Upstream (m)	Upstream (m)	Dia (mm)	Length (m)	'n'	(m^3/s)	(m^3/s)		Pipe Area (m^2)	L/D	Friction Factor (f)	Velocity V (m/s)	$V^2/2g$	HL (m)	Upstream (m)	U/S (m)	D/S (m)	SLOPE (%)	(%)	Upstream (m)
																				63.10	<- OUTLET TO RICHMOND SEWER			
WILMONT	28522	28523	62.84	62.58	66.23	2.865	525	84.50	0.013	0.262	0.249	1.05	0.223	161	0.02610	1.17	0.07	0.32	0.06	63.42	63.11	0.38	0.31	63.72
	28521	28522	63.27	62.91	66.56	2.840	450	78.80	0.013	0.234	0.201	1.16	0.164	175	0.02747	1.43	0.10	0.52	0.23	63.95	63.42	0.67	0.46	64.25
	28520	28521	63.54	63.28	66.59	2.600	450	68.70	0.013	0.186	0.183	1.02	0.164	153	0.02747	1.13	0.07	0.39	0.35	64.34	63.95	0.57	0.38	64.64
WINSTON	28519	28520	63.86	63.62	66.69	2.455	375	81.60	0.013	0.108	0.099	1.09	0.114	218	0.02920	0.95	0.05	0.31	0.41	64.65	64.34	0.37	0.29	64.95
	28518	28519	64.20	63.91	66.81	2.235	375	76.60	0.013	0.072	0.113	0.64	0.114	204	0.02920	0.63	0.02	0.13	0.20	64.77	64.65	0.17	0.38	65.07
	28517	28518	65.86	64.23	68.45	2.290	300	57.90	0.013	0.033	0.169	0.19	0.073	193	0.03145	0.45	0.01	0.07	0.00	66.16	64.77	2.40	2.82	66.46

DESIGN PARAMETERS

DOWNSTREAM WATER LEVEL at Outlet = 67.57m (EPA SWMM MAX HGL)
RETURN FREQUENCY = 5 YEAR
MINIMUM VELOCITY= 0.80 m/s
MANNING'S n= 0.013
MIN. HGL CLEARANCE 0.30m

HGL=Major + Minor Losses
Major Loss= Pipe Friction (Darcy-Weisbach)
Minor Loss= Head loss correction for flow through MH, changes in pipe size, and pipe bends
Friction Factor= 8g/c^2, where c=(1/n)*(D/4)^1/6

Designed: MS
Checked: RSC
Dwg. Reference:

PROJECT:
335 ROOSEVELT AVENUE

CLIENT:
UNIFORM URBAN DEVELOPMENTS

Date: June 15, 2011

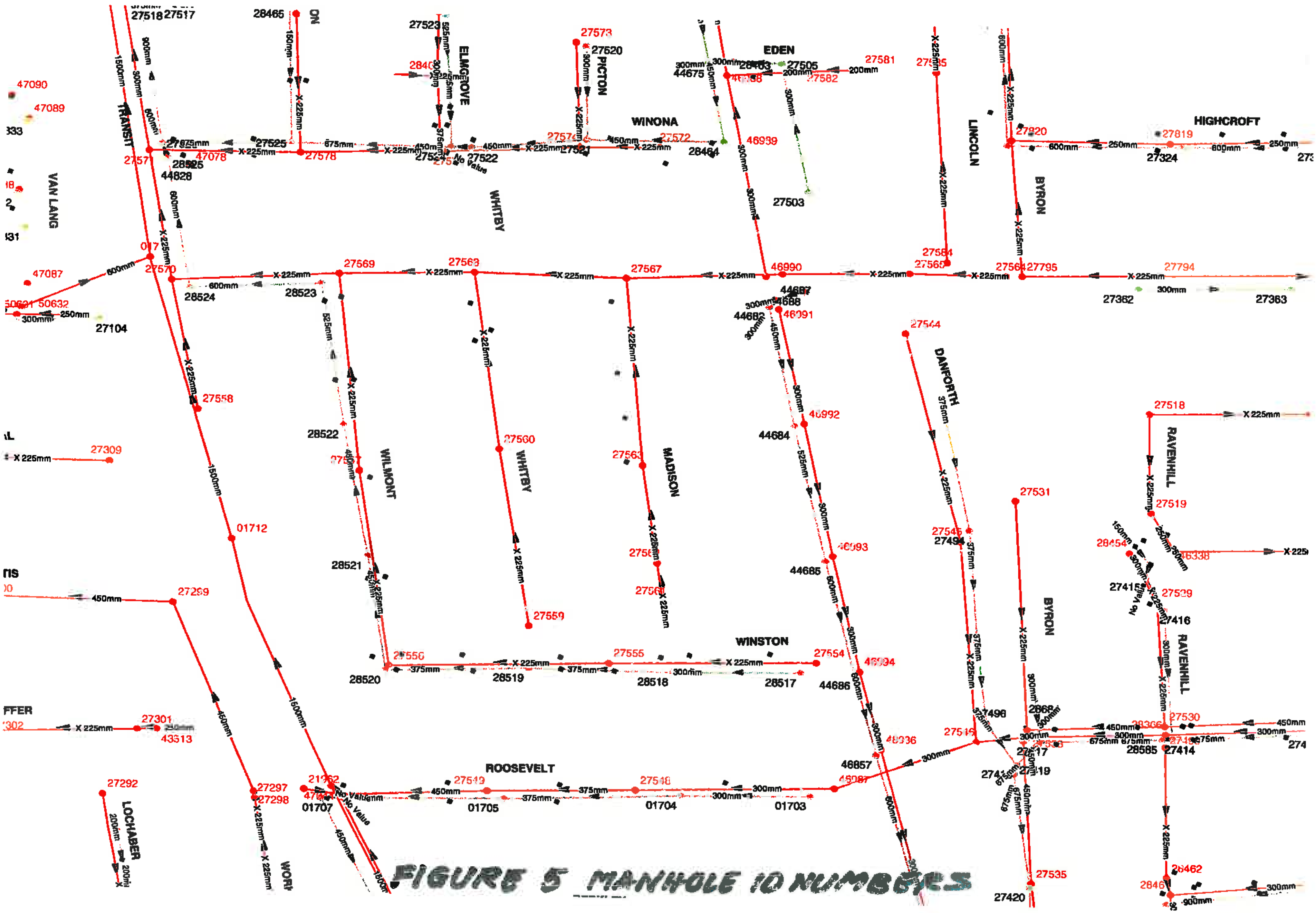


FIGURE 5 MANHOLE ID NUMBERS

TABLE 1A: Post-Development Runoff Coefficient "C" - A-01

Area	Surface	Ha	"C"	C_{avg}	$*C_{100}$
Total	Hard	0.030	0.90	0.55	0.63
0.060	Soft	0.030	0.20		

Runoff Coefficient Equation
$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{Tot}$
* Runoff Coefficient increases by 25% up to a maximum value of 1.00 for the 100-Year event

TABLE 1B: Post-Development A-01 Flows

Outlet Options	**Area (ha)**	C_{avg}	**Tc (min)**	$Q_{5\ Year}$ **(L/s)**	$Q_{100\ Year}$ **(L/s)**
Roosevelt Ave/ Wilmont Ave/ Transitway	0.060	0.55	20	**6.4**	**12.5**

Time of Concentration	Tc=	20	min
Intensity (5 Year Event)	I_5=	70.25	mm/hr
Intensity (100 Year Event)	I_{100}=	119.95	mm/hr

100 year Intensity = 1735.688 / (Time in min + 6.014) $^{0.820}$
5 year Intensity = 998.071 / (Time in min + 6.053) $^{0.814}$

Equations:
$Q = 2.78 \times C \times I \times A$
Where:
C is the runoff coefficient
I is the rainfall intensity, City of Ottawa IDF
A is the total drainage area

TABLE 2A: Post-Development Runoff Coefficient "C" - B-01 & C-01

			5 Year Event		100 Year Event	
Area		Ha	"C"	C_{avg}	"C" + 25%	*C_{avg}
Total	Hard	0.510	0.90	0.74	1.00	0.83
0.660	Soft	0.150	0.20		0.25	

TABLE 2B: 5 YEAR EVENT QUANTITY STORAGE REQUIREMENT - B-01 & C-01

0.660 =Area (ha)
0.74 = C

Return Period	Time (min)	Intensity (mm/hr)	Flow Q (L/s)	Allowable Runoff (L/s)	Net Flow to be Stored (L/s)	Storage Req'd (m^3)
5 YEAR	30	53.93	73.31	16.7	56.61	101.90
	35	48.52	65.96	16.7	49.26	103.44
	40	**44.18**	**60.07**	**16.7**	**43.37**	**104.08**
	45	40.63	55.23	16.7	38.53	104.03
	50	37.65	51.19	16.7	34.49	103.46

TABLE 2C: 100 YEAR EVENT QUANTITY STORAGE REQUIREMENT - B-01 & C-01

0.66 =Area (ha)
0.83 = C

Return Period	Time (min)	Intensity (mm/hr)	Flow Q (L/s)	Allowable Runoff (L/s)	Net Flow to be Stored (L/s)	Storage Req'd (m^3)
100 YEAR	70	49.79	75.78	16.7	59.08	248.15
	75	47.26	71.93	16.7	55.23	248.51
	80	**44.99**	**68.48**	**16.7**	**51.78**	**248.54**
	85	42.95	65.38	16.7	48.68	248.26
	90	41.11	62.57	16.7	45.87	247.71

Equations:
Flow Equation
$Q = 2.78 \times C \times I \times A$
Where:
C is the runoff coefficient
I is the rainfall intensity, City of Ottawa IDF
A is the total drainage area

Runoff Coefficient Equation
$C_5 = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{Tot}$
$C_{100} = (A_{hard} \times 1.0 + A_{soft} \times 0.25)/A_{Tot}$

Table 4: Post-Development Stormwater Mangement Summary

Area ID	Area (ha)	1:5 Year Weighted Cw	5 Year Storm Event		100 Year Storm Event	
			Release (L/s)	Req'd Vol (cu.m)	Release (L/s)	Req'd Vol (cu.m)
A-01	0.060	0.55	6.4	N/A	12.5	N/A
B-01/C-01	0.660	0.74	16.7	104.08	16.7	248.54
Total Flow to Wilmont Ave			**23.1**		**29.2**	
Allowable Flow to Wilmont Ave			**29.2**		**29.2**	

TABLE 1A: Post-Development Runoff Coefficient "C" - A-01

Area	Surface	Ha	"C"	C_{avg}	*C_{100}
Total	Hard	0.030	0.90	0.55	0.63
0.060	Soft	0.030	0.20		

Runoff Coefficient Equation
$C = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{Tot}$
* Runoff Coefficient increases by 25% up to a maximum value of 1.00 for the 100-Year event

TABLE 1B: Post-Development A-01 Flows

Outlet Options	Area (ha)	C_{avg}	Tc (min)	$Q_{5\ Year}$ (L/s)	$Q_{100\ Year}$ (L/s)
Roosevelt Ave/ Wilmont Ave/ Transitway	0.060	0.55	20	**6.4**	**12.5**

Time of Concentration	Tc=	20	min
Intensity (5 Year Event)	I_5=	70.25	mm/hr
Intensity (100 Year Event)	I_{100}=	119.95	mm/hr

100 year Intensity = 1735.688 / (Time in min + 6.014) $^{0.820}$
5 year Intensity = 998.071 / (Time in min + 6.053) $^{0.814}$

Equations:
$Q = 2.78 \times C \times I \times A$
Where:
C is the runoff coefficient
I is the rainfall intensity, City of Ottawa IDF
A is the total drainage area

TABLE 2A: Post-Development Runoff Coefficient "C" - B-01

			5 Year Event		100 Year Event	
Area		Ha	"C"	C_{avg}	"C" + 25%	*C_{avg}
Total	Hard	0.310	0.90	0.90	1.00	1.00
0.310	Soft	0.000	0.20		0.25	

TABLE 2B: 5 YEAR EVENT QUANTITY STORAGE REQUIREMENT - B-01

0.310 =Area (ha)
0.90 = C

Return Period	Time (min)	Intensity (mm/hr)	Flow Q (L/s)	Allowable Runoff (L/s)	Net Flow to be Stored (L/s)	Storage Req'd (m^3)
5 YEAR	20	70.25	54.49	12.6	41.85	50.22
	25	60.90	47.23	12.6	34.59	51.89
	30	**53.93**	**41.83**	**12.6**	**29.19**	**52.54**
	35	48.52	37.63	12.6	24.99	52.48
	40	44.18	34.27	12.6	21.63	51.91

TABLE 2C: 100 YEAR EVENT QUANTITY STORAGE REQUIREMENT - B-01

0.31 =Area (ha)
1.00 = C

Return Period	Time (min)	Intensity (mm/hr)	Flow Q (L/s)	Allowable Runoff (L/s)	Net Flow to be Stored (L/s)	Storage Req'd (m^3)
100 YEAR	30	91.87	79.17	17.6	61.57	110.83
	35	82.58	71.17	17.6	53.57	112.49
	40	**75.15**	**64.76**	**17.6**	**47.16**	**113.18**
	45	69.05	59.51	17.6	41.91	113.15
	50	63.95	55.12	17.6	37.52	112.55

Equations:
Flow Equation
$Q = 2.78 \times C \times I \times A$
Where:
C is the runoff coefficient
I is the rainfall intensity, City of Ottawa IDF
A is the total drainage area

Runoff Coefficient Equation
$C_5 = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{Tot}$
$C_{100} = (A_{hard} \times 1.0 + A_{soft} \times 0.25)/A_{Tot}$

TABLE 3A: Post-Development Runoff Coefficient "C" - C-01

			5 Year Event		100 Year Event	
Area		Ha	"C"	C_{avg}	"C" + 25%	*C_{avg}
Total	Hard	0.200	0.90	0.60	1.00	0.68
0.350	Soft	0.150	0.20		0.25	

TABLE 3B: 5 YEAR EVENT QUANTITY STORAGE REQUIREMENT - C-01

0.350 =Area (ha)
0.60 = C

Return Period	Time (min)	Intensity (mm/hr)	Flow Q (L/s)	Allowable Runoff (L/s)	Net Flow to be Stored (L/s)	Storage Req'd (m^3)
5 YEAR	0	230.48	134.56	40.20	94.36	0.00
	5	141.18	82.42	40.20	42.22	12.67
	10	**104.19**	**60.83**	**40.20**	**20.63**	**12.38**
	15	83.56	48.78	40.20	8.58	7.72
	20	70.25	41.01	40.20	0.81	0.98

TABLE 3C: 100 YEAR EVENT QUANTITY STORAGE REQUIREMENT - C-01

0.35 =Area (ha)
0.68 = C

Return Period	Time (min)	Intensity (mm/hr)	Flow Q (L/s)	Allowable Runoff (L/s)	Net Flow to be Stored (L/s)	Storage Req'd (m^3)
100 YEAR	5	242.70	160.25	40.20	120.05	36.01
	10	178.56	117.89	40.20	77.69	46.62
	15	**142.89**	**94.35**	**40.20**	**54.15**	**48.73**
	20	119.95	79.20	40.20	39.00	46.80
	25	103.85	68.57	40.20	28.37	42.55

Equations:
Flow Equation
Q = 2.78 x C x I x A
Where:
C is the runoff coefficient
I is the rainfall intensity, City of Ottawa IDF
A is the total drainage area

Runoff Coefficient Equation
$C_5 = (A_{hard} \times 0.9 + A_{soft} \times 0.2)/A_{Tot}$
$C_{100} = (A_{hard} \times 1.0 + A_{soft} \times 0.25)/A_{Tot}$

Table 4: Post-Development Stormwater Mangement Summary

Area ID	Area (ha)	1:5 Year Weighted Cw	5 Year Storm Event		100 Year Storm Event	
			Release (L/s)	Req'd Vol (cu.m)	Release (L/s)	Req'd Vol (cu.m)
A-01	0.060	0.55	6.4	N/A	12.5	N/A
B-01	0.310	0.90	12.6	52.54	17.6	113.18
C-01	0.350	0.60	40.2	12.38	40.2	48.73
Total Flow to Wilmont Ave			**59.2**		**70.3**	
Allowable Flow to Wilmont Ave			**70.3**		**70.3**	

Appendix D
Sanitary Sewer Design Sheets and Sanitary Calculations

Ottawa

335 Roosevelt Avenue

New Sanitary Flows

Site		
Number of Units	361	
Persons per Unit	1.8	
Population	650	
Residential Peak Factor	3.9	
Average Residential Flow	280	L/c/day
Peak Residential Flow	**6.59**	**L/s**
Site Area	0.72	ha
Infiltration Allowance	0.33	L/s/ha
Peak Extraneous Flows	**0.24**	**L/s**
Peak Sanitary Flow	**6.83**	**L/s**

Existing Sanitary Flows

Roosevelt		
Peak Sanitary Flow	**2.33**	**L/s**
Dominion Ave		
Peak Sanitary Flow	**6.17**	**L/s**
Tay St		
Peak Sanitary Flow	**0.25**	**L/s**
Berkley Ave		
Peak Sanitary Flow	**1.52**	**L/s**
Richmond Rd.		
Peak Sanitary Flow	**78.10**	**L/s**
Total Ex. Sanitary Flows	**88.36**	**L/s**

TOTAL FLOW to 1500 SAN TRUNK

95.19 L/s

335 Roosevelt Avenue
Existing Sanitary Flows

Single Family	29	
Persons per Unit	3.4	
Semi Detached	10	
Persons per Unit	2.7	
Population	126	
Residential Peak Factor	4.0	
Average Residential Flow	280	L/c/day
Peak Residential Flow	**1.30**	**L/s**
Commercial Area	0.26	ha
Average Commercial Flow	28000	L/ha/day
Commercial Peak Factor	1	
Peak Commercial Flow	**0.08**	**L/s**
Site Area	2.85	ha
Infiltration Allowance	0.33	L/s/ha
Peak Extraneous Flows	**0.94**	**L/s**
Peak Sanitary Flow	**2.33**	**L/s**

Richmond Rd, to Berkley

Peak Sanitary Flow	**78.10**	**L/s**

(Design Flow from 2003 Project Richmond Rd Rehabilitation)

Berkeley Ave

Single Family	18	
Persons per Unit	3.4	
Semi Detached	4	
Persons per Unit	2.7	
Duplex	1	
Persons per Unit	2.3	
Townhouse	8	
Persons per Unit	2.7	
Population	96	
Residential Peak Factor	4.0	
Average Residential Flow	280	L/c/day
Peak Residential Flow	**0.99**	**L/s**
Site Area	1.58	ha
Infiltration Allowance	0.33	L/s/ha
Peak Extraneous Flows	**0.52**	**L/s**
Peak Sanitary Flow	**1.52**	**L/s**

Tay St.

Townhouse	6	
Persons per Unit	2.7	
Population	16	
Residential Peak Factor	4.0	
Average Residential Flow	280	L/c/day
Peak Residential Flow	**0.17**	**L/s**
Site Area	0.25	ha
Infiltration Allowance	0.33	L/s/ha
Peak Extraneous Flows	**0.08**	**L/s**
Peak Sanitary Flow	**0.25**	**L/s**

Dominion Ave

Single Family	9	
Persons per Unit	3.4	
Barclay Apt Units	94	
Persons per Unit	1.8	
Plaza Towers Apt Units	197	
Persons per Unit	1.8	
Population	554	
Residential Peak Factor	4.0	
Average Residential Flow	280	L/c/day
Peak Residential Flow	**5.68**	**L/s**
Site Area	1.49	ha
Infiltration Allowance	0.33	L/s/ha
Peak Extraneous Flows	**0.49**	**L/s**
Peak Sanitary Flow	**6.17**	**L/s**

NOVATECH ENGINEERING CONSULTANTS LTD.

SANITARY SEWER DESIGN SHEET

RICHMOND ROAD REHABILITATION
BROADVIEW AVE. to ROOSEVELT AVE.
CITY OF OTTAWA

Flow for Zoning = City of Ottawa Guidelines
Peak Factor M = City of Ottawa Guidelines
Weeping Tile Flow Allowance 1.0 L/ha/s
Infiltration Flow Allowance = 0.2 L/ha/s

Manning Coefficient n = 0.013

Location			Individual												Sewer Data						
Street	From	To	Area No.	Zone Type	Ave. Flow for Zoning (L/s/ha)	Area (ha)	Incr. Flow (L/s)	Area in Run (ha)	Flow in Run (L/s)	Peak Factor M	Peak Flow (L/s)	Extr. Flow (L/s)		Design Flow (L/s)	Type of Pipe	Pipe Size (mm)	Pipe Length (m)	Grade %	Capacity Full (L/s)	Velocity Full (m/s)	Velocity Actual (m/s)
WESTERLY, Churchill to Berkley:																					
Richmond Rd. [FUTURE]	307A	293A	A1	C1	3.00	0.55	1.7	0.55	1.7	6.00	10.2	0.7	*	10.9	PVC	250	67.92	0.44	41.2	0.8	0.6
Richmond Rd. [FUTURE]	293A	293B	A2	C1	3.00	0.65	2.0	1.20	3.6	6.00	21.6	1.4	*	23.0	PVC	250	79.07	0.44	41.2	0.8	0.7
Richmond Rd. [FUTURE]	293B	294A	A3	C1	3.00	0.51	1.5	1.71	5.1	6.00	30.6	2.1	*	32.7	PVC	300	69.42	0.40	63.8	0.9	0.8
Richmond Rd. [FUTURE]	294A	294B	A4	C1	3.00	0.37	1.1	2.08	6.2	6.00	37.2	2.5	*	39.7	PVC	300	46.90	0.40	63.8	0.9	0.8
Richmond Rd.	294B	295A	A5	C1	3.00	0.75	2.3	2.83	8.5	6.00	51.0	3.4	*	54.4	PVC	300	114.50	1.00	100.9	1.4	1.2
EASTERLY, Golden to Berkley:																					
			A7	R6	2.90	0.83	2.4														
Richmond Rd.	295B	295A	A6	C1	3.00	0.51	1.5	1.34	3.9	6.00	23.4	0.3		23.7	PVC	300	60.99	0.50	71.3	1.0	0.7
Berkley Ave.	295A	EXIST	—	—	—	—	—	—	—	—	—	—		78.1	EXIST	375	107.00	0.20	81.8	0.7	0.7

4.17

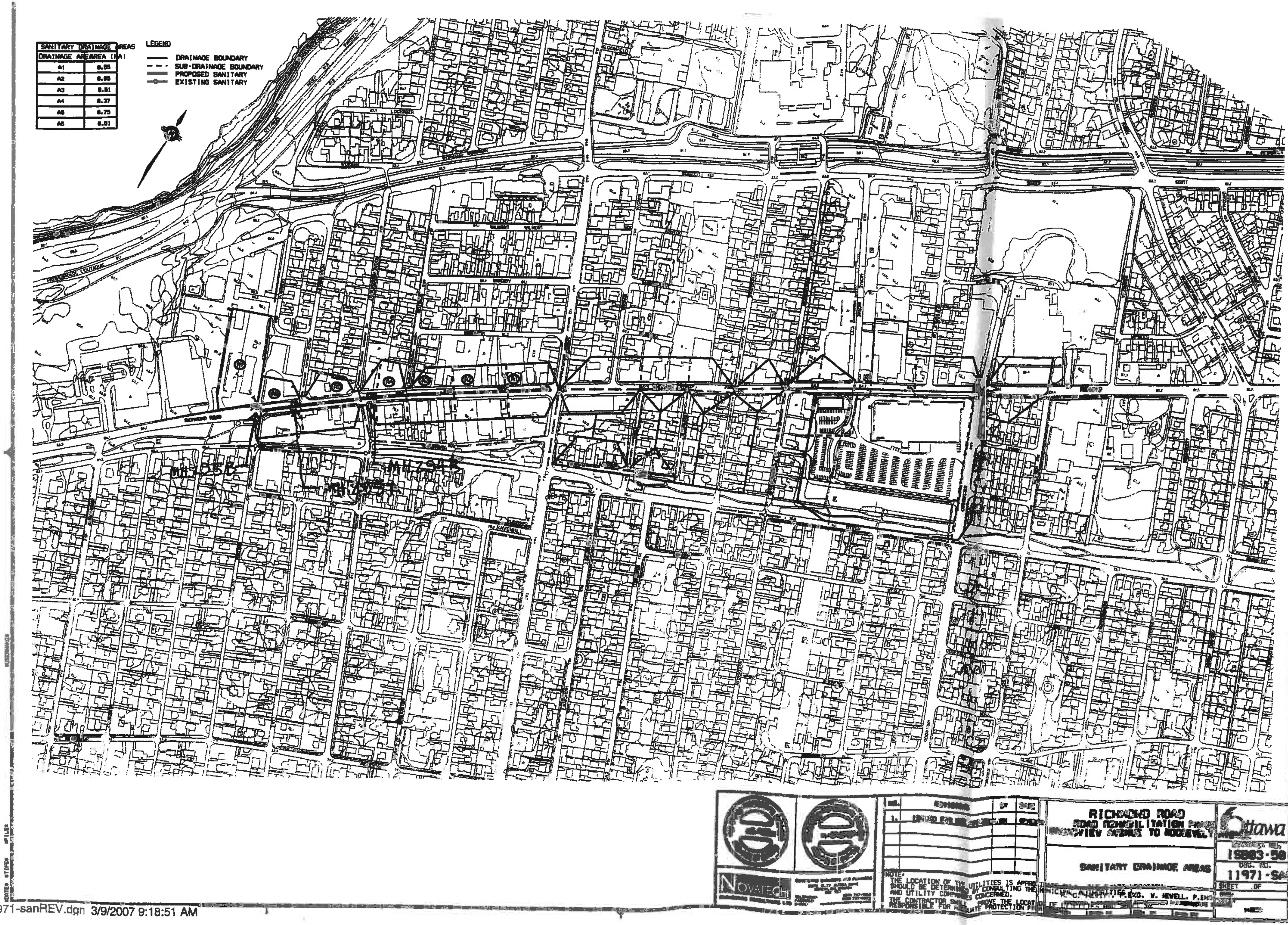
* Includes Weeping Tile Flow Allowance

Design: W. Hernberger	Project: Richmond Road Rehabilitation Broadview Ave. to Roosevelt Ave.
Checked:	
Date: June, 2003	Client: City of Ottawa

REGISTERED PROFESSIONAL ENGINEER W. HERNBERGER June 3/03 PROVINCE OF ONTARIO

K:\1998\98002\SEWERS\RichmondRdSanitary_BroadviewToRoosevelt.xls

Tab Name: SANITARY June03



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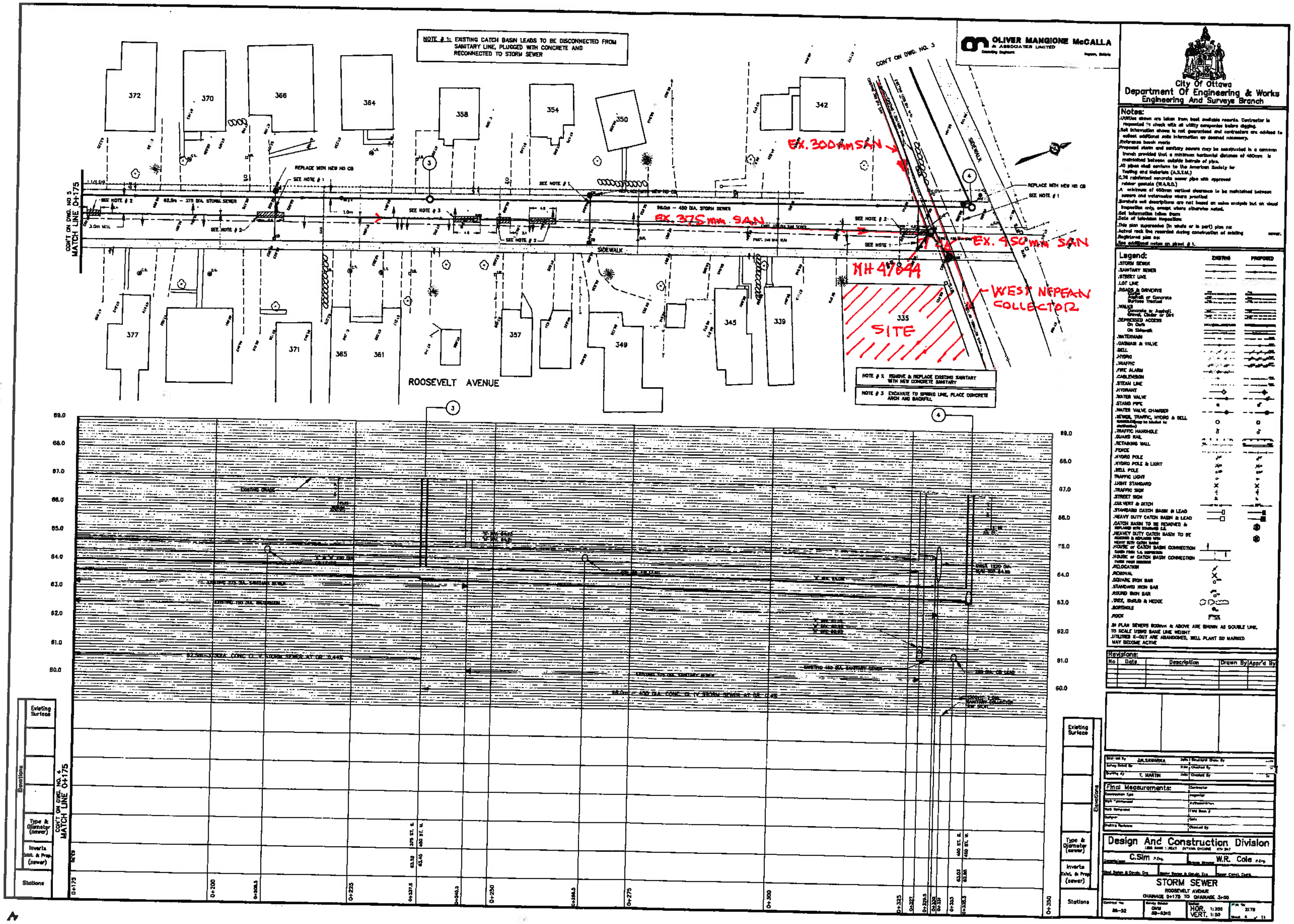
2179 4 of 11



NOTE # 1: EXISTING CATCH BASIN LEADS TO BE DISCONNECTED FROM SANITARY LINE, PLUGGED WITH CONCRETE AND RECONNECTED TO STORM SEWER
OLIVER MANGIONE McCALLA & ASSOCIATES LIMITED
City Of Ottawa
Department Of Engineering & Works
Engineering And Survey Branch
Notes:
Legend:
EX. 300mm SAN
EX. 375mm SAN
EX. 450mm SAN
MH 47644
WEST NEPEAN COLLECTOR
SITE
335
ROOSEVELT AVENUE
SIDEWALK
CON'T ON DWG. NO. 3
CON'T ON DWG. NO. 5 MATCH LINE 0+175
NOTE # 2 REMOVE & REPLACE EXISTING SANITARY WITH NEW CONCRETE SANITARY
NOTE # 3 EXCAVATE TO SPRING LINE, PLACE CONCRETE ARCH AND BACKFILL
372
370
366
364
358
354
350
342
377
371
365
361
357
349
345
339
Existing Surface
Elevations
Type & Diameter (sewer)
Inverts Exist. & Prop. (sewer)
Stations
Revisions:
Final Measurements:
T. MARTIN
Design And Construction Division
C.Sim P.Eng.
W.R. Cole P.Eng.
STORM SEWER
ROOSEVELT AVENUE
CHAINAGE 0+175 TO CHAINAGE 3+50
HOR. 1:200
VERT. 1:50
2179

Appendix E
Boundary Conditions, Water Demands and FUS Calculations

Ben Sweet

From: Wu, John <John.Wu@ottawa.ca>
Sent: Tuesday, May 5, 2020 3:28 PM
To: Ben Sweet
Subject: RE: 335 Roosevelt Ave - Boundary Conditions
Attachments: 335 Roosevelt May 2020.pdf

Here is the result:

The following are boundary conditions, HGL, for hydraulic analysis at 335 Roosevelt (zone 1W) assumed to be connected to the 152mm on Roosevelt and 152mm on Winston (see attached PDF for location).

	152mm on Roosevelt	152mm on Winston
Minimum HGL	108.4m	106.0m
Maximum HGL	115.0m	114.7m
Max Day + Fire Flow (167 L/s)	110.0m	Available FF = 65 L/s @20psi

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.

It looks, the looping is required.

John

From: Ben Sweet <b.sweet@novatech-eng.com>
Sent: April 30, 2020 2:32 PM
To: Wu, John <John.Wu@ottawa.ca>
Cc: Sam Bahia <s.bahia@novatech-eng.com>
Subject: 335 Roosevelt Ave - Boundary Conditions

Hi John,

I hope all is well.

Could you please provide water boundary conditions for the above noted site based on the info provided below. If you have any question, or require additional information, let me know.

Roosevelt Ave – Connection 1 (Building West demand)

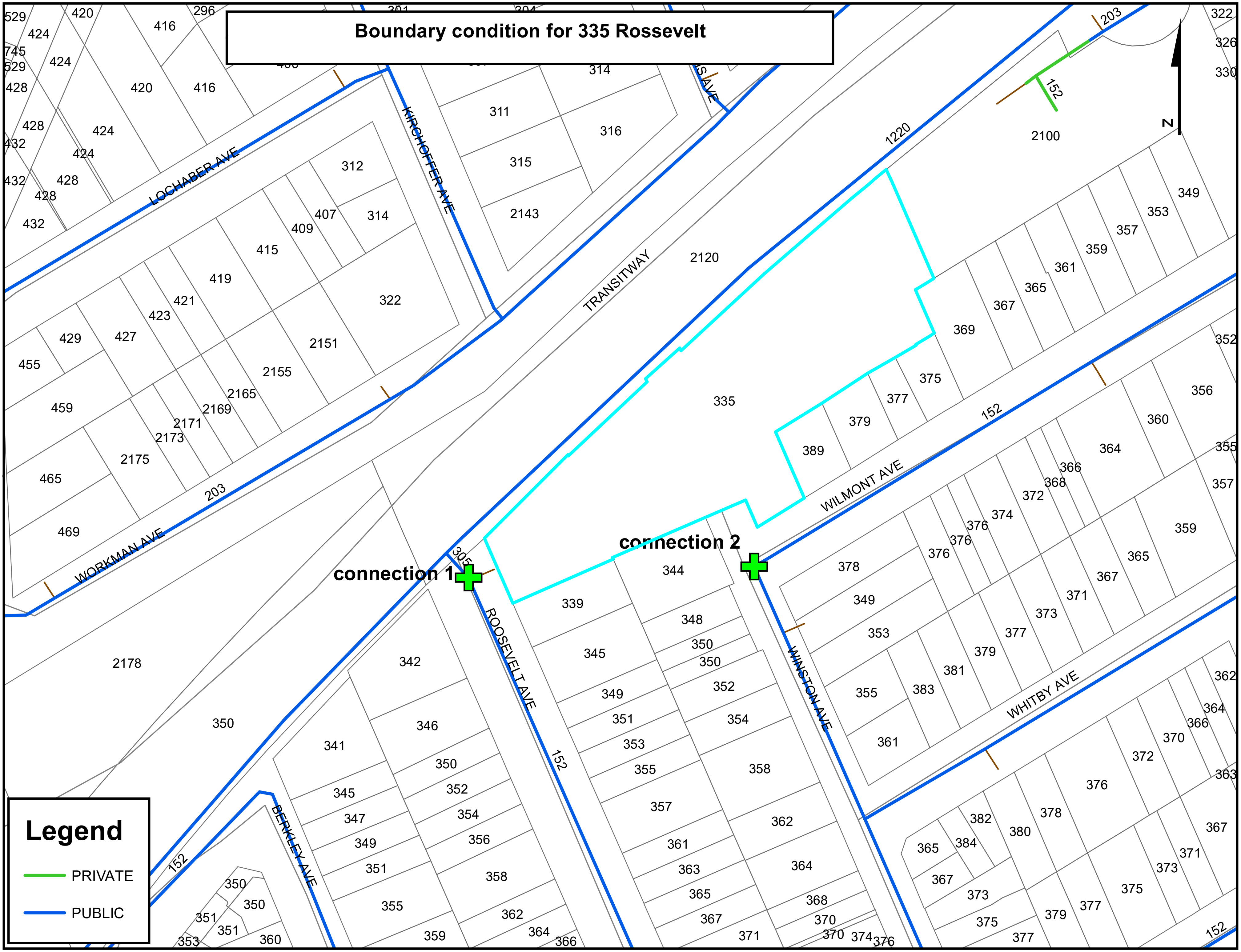
- i. The water connection will be made at Roosevelt Ave (connection 1), see figure attached.
- ii. Residential development with required fire flows: 67 L/s, 83 L/s and 167 L/s see FUS calcs attached.
- iii. Average daily demand: 1.26 L/s.
- iv. Maximum daily demand: 3.15 L/s.
- v. Maximum hourly daily demand: 6.93 L/s.

Winston Ave – Connection 2 (Building East demand)

- i. The water connection will be made at Winston Ave (connection 2), see figure attached.
- ii. Residential development with required fire flows: 67 L/s, 83 L/s and 167 L/s see FUS calcs attached.
- iii. Average daily demand: 1.07 L/s.
- iv. Maximum daily demand: 2.68 L/s.
- v. Maximum hourly daily demand: 5.90 L/s.

Ben Sweet, P.Eng., Project Coordinator | Land Development
NOVATECH Engineers, Planners & Landscape Architects
240 Michael Cowpland Drive, Suite 200, Ottawa, ON, K2M 1P6 | Tel: 613.254.9643 x 250 | Fax: 613.254.5867

Boundary condition for 335 Rossevelt



Engineers, Planners & Landscape Architects

Project: 335 Roosevelt Ave
Proj. No.: 110098
Design: BS

Residential Water Demand

No. of Units	=	361		
	=	1.8	persons/unit	
Flow/capita	=	280	L/day/person	
Average Day	=	181944	L/day	(No. Units x No. People x Residential Flow)
	=	2.11	L/s	Greater than $50m^3$ YES
Maximum Day	=	454860	L/day	(2.5 x Average Day)
	=	5.26	L/s	
Peak Hour	=	1000692	L/day	(2.2 x Maximum Day)
	=	11.58	L/s	
Governing FUS	=	4000	L/min	
	=	83	L/s	Refer to FUS calculation sheet

FUS - Fire Flow Calculations

As per 1999 Fire Underwriter's Survey Guidelines

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #:	110098
Project Name:	335 Roosevelt Ave
Date:	7/17/2020
Input By:	Ben Sweet
Reviewed By:	Sam Bahia

Legend: Input by User; No Information or Input Required

Building Description: Building West (21 Storeys)

Fire Resistive Construction

Step			Choose		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Wood frame		1.5	0.6	
		Ordinary construction		1		
		Non-combustible construction		0.8		
		Modified Fire resistive construction (2 hrs)	Yes	0.6		
		Fire resistive construction (> 3 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	1010			
		Number of Floors/Storeys	21			
		Protected Openings (1 hr)	Yes			
		Area of structure considered (m^2)			**1,515**	
	F	**Base fire flow without reductions**				**5,000**
		$F = 220\ C\ (A)^{0.5}$				
Reductions or Surcharges						
3	**Occupancy hazard reduction or surcharge**			**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**4,250**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**			**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	Yes	-30%	-30%	**-1,700**
		Standard Water Supply	Yes	-10%	-10%	
		Fully Supervised System	No	-10%		
				Cumulative Total	**-40%**	
5	**Exposure Surcharge (cumulative %)**				**Surcharge**	
	(3)	North Side	30.1- 45 m		5%	**1,913**
		East Side	20.1 - 30 m		10%	
		South Side	3.1 - 10 m		20%	
		West Side	20.1 - 30 m		10%	
				Cumulative Total	**45%**	
Results						
6	**(1) + (2) + (3)**	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**4,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**67**
				or	USGPM	1,057
7	**Storage Volume**	Required Duration of Fire Flow (hours)			Hours	1.5
		Required Volume of Fire Flow (m^3)			m^3	360

FUS - Fire Flow Calculations

As per 1999 Fire Underwriter's Survey Guidelines

NOVATECH
Engineers, Planners & Landscape Architects

Novatech Project #:	110098
Project Name:	335 Roosevelt Ave
Date:	7/17/2020
Input By:	Ben Sweet
Reviewed By:	Sam Bahia

Legend: Input by User; No Information or Input Required

Building Description:	Building East (18 Storeys)
	Fire Resistive Construction

Step			Choose		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Wood frame		1.5	0.6	
		Ordinary construction		1		
		Non-combustible construction		0.8		
		Modified Fire resistive construction (2 hrs)	Yes	0.6		
		Fire resistive construction (> 3 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	1010			
		Number of Floors/Storeys	18			
		Protected Openings (1 hr)	Yes			
		Area of structure considered (m^2)			**1,515**	
	F	**Base fire flow without reductions**				**5,000**
		$F = 220\ C\ (A)^{0.5}$				
Reductions or Surcharges						
3	**Occupancy hazard reduction or surcharge**			**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**4,250**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**			**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	Yes	-30%	-30%	**-1,700**
		Standard Water Supply	Yes	-10%	-10%	
		Fully Supervised System	No	-10%		
				Cumulative Total	**-40%**	
5	**Exposure Surcharge (cumulative %)**				**Surcharge**	
	(3)	North Side	30.1- 45 m		5%	**2,125**
		East Side	3.1 - 10 m		20%	
		South Side	10.1 - 20 m		15%	
		West Side	20.1 - 30 m		10%	
				Cumulative Total	**50%**	
Results						
6	(1) + (2) + (3)	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**5,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**83**
				or	USGPM	1,321
7	**Storage Volume**	Required Duration of Fire Flow (hours)			Hours	1.75
		Required Volume of Fire Flow (m^3)			m^3	525

FUS - Fire Flow Calculations

As per 1999 Fire Underwriter's Survey Guidelines

NOVATECH
Engineers, Planners & Landscape Architects

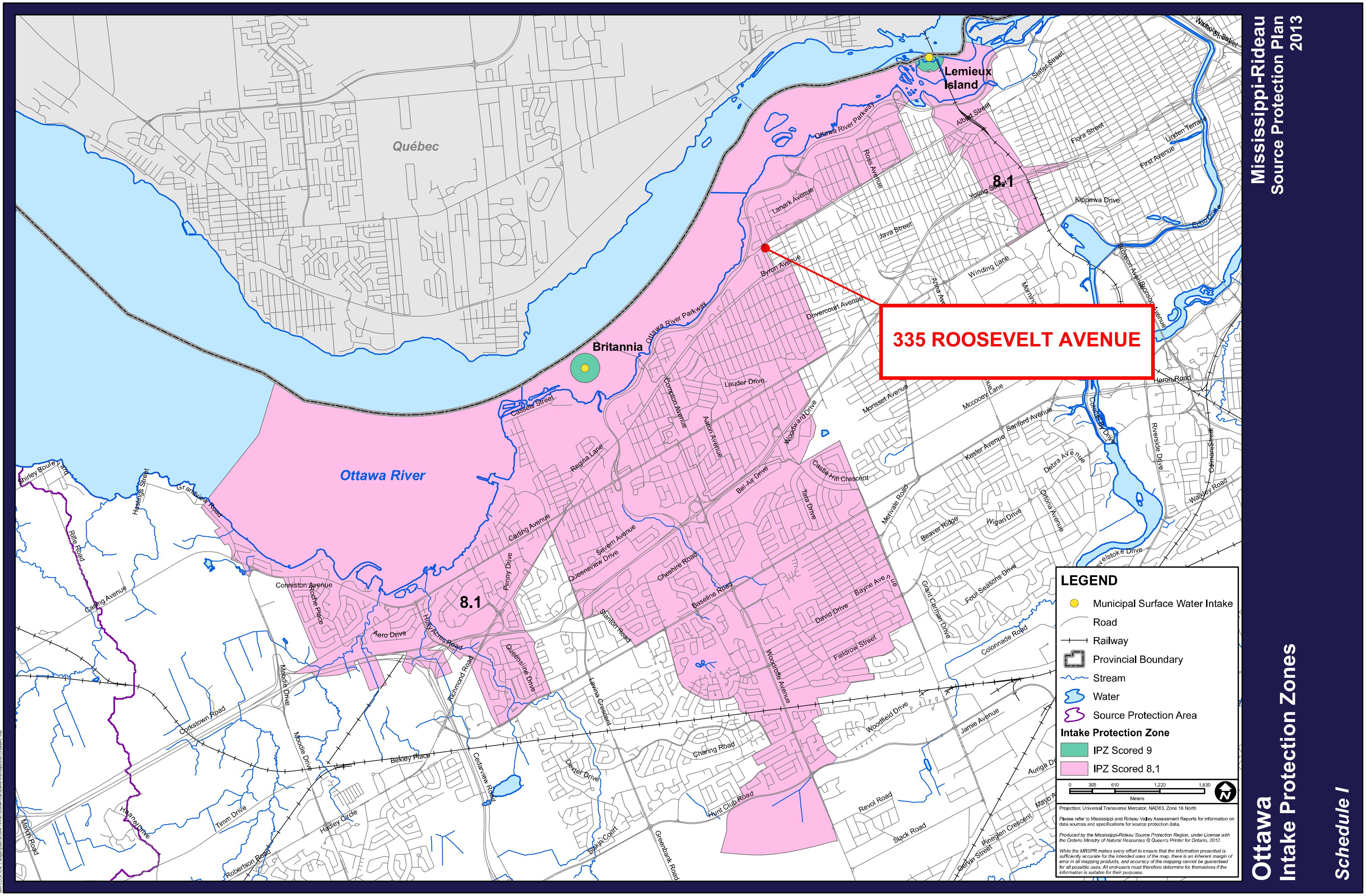
Novatech Project #: 110098
Project Name: 335 Roosevelt Ave
Date: 7/17/2020
Input By: Ben Sweet
Reviewed By: Sam Bahia

Legend: Input by User; No Information or Input Required

Building Description: Low Rise Buildings (Block C and D - worst case scenario)
Fire Resistive Construction

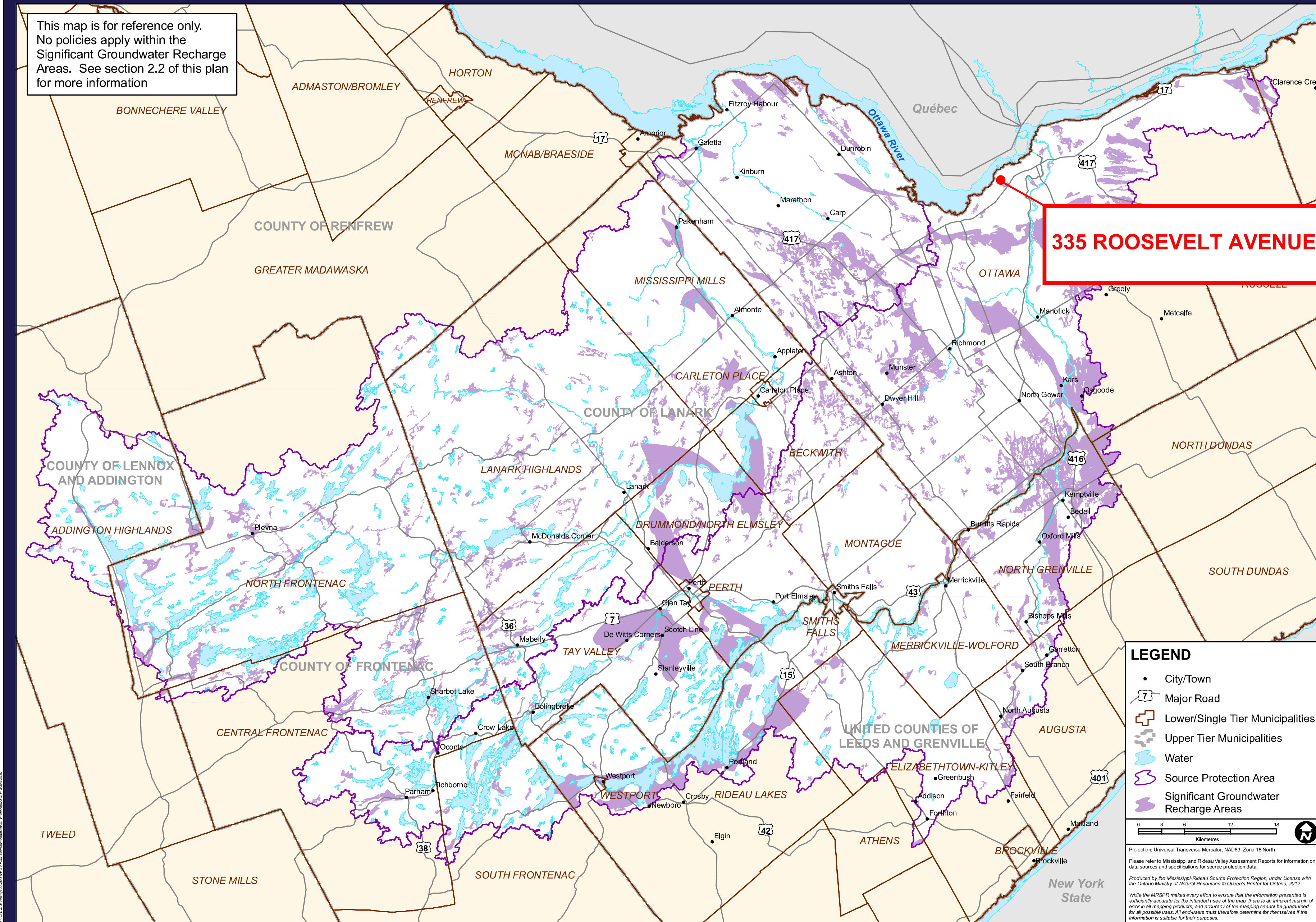
Step			Choose		Value Used	Total Fire Flow (L/min)
Base Fire Flow						
1	**Construction Material**			**Multiplier**		
	Coefficient related to type of construction C	Wood frame		1.5	0.6	
		Ordinary construction		1		
		Non-combustible construction		0.8		
		Modified Fire resistive construction (2 hrs)	Yes	0.6		
		Fire resistive construction (> 3 hrs)		0.6		
2	**Floor Area**					
	A	Building Footprint (m^2)	645			
		Number of Floors/Storeys	3			
		Protected Openings (1 hr)	Yes			
		Area of structure considered (m^2)			**968**	
	F	**Base fire flow without reductions**				**4,000**
		$F = 220\ C\ (A)^{0.5}$				
Reductions or Surcharges						
3	**Occupancy hazard reduction or surcharge**			**Reduction/Surcharge**		
	(1)	Non-combustible		-25%	-15%	**3,400**
		Limited combustible	Yes	-15%		
		Combustible		0%		
		Free burning		15%		
		Rapid burning		25%		
4	**Sprinkler Reduction**			**Reduction**		
	(2)	Adequately Designed System (NFPA 13)	Yes	-30%	-30%	**-1,360**
		Standard Water Supply	Yes	-10%	-10%	
		Fully Supervised System	No	-10%		
				Cumulative Total	**-40%**	
5	**Exposure Surcharge (cumulative %)**				**Surcharge**	
	(3)	North Side	10.1 - 20 m		15%	**2,040**
		East Side	3.1 - 10 m		20%	
		South Side	20.1 - 30 m		10%	
		West Side	10.1 - 20 m		15%	
				Cumulative Total	**60%**	
Results						
6	(1) + (2) + (3)	**Total Required Fire Flow, rounded to nearest 1000L/min**			**L/min**	**4,000**
		(2,000 L/min < Fire Flow < 45,000 L/min)		or	L/s	**67**
				or	USGPM	1,057
7	**Storage Volume**	Required Duration of Fire Flow (hours)			Hours	1.5
		Required Volume of Fire Flow (m^3)			m^3	360

Appendix F
Mississippi-Rideau Source Protection Plan



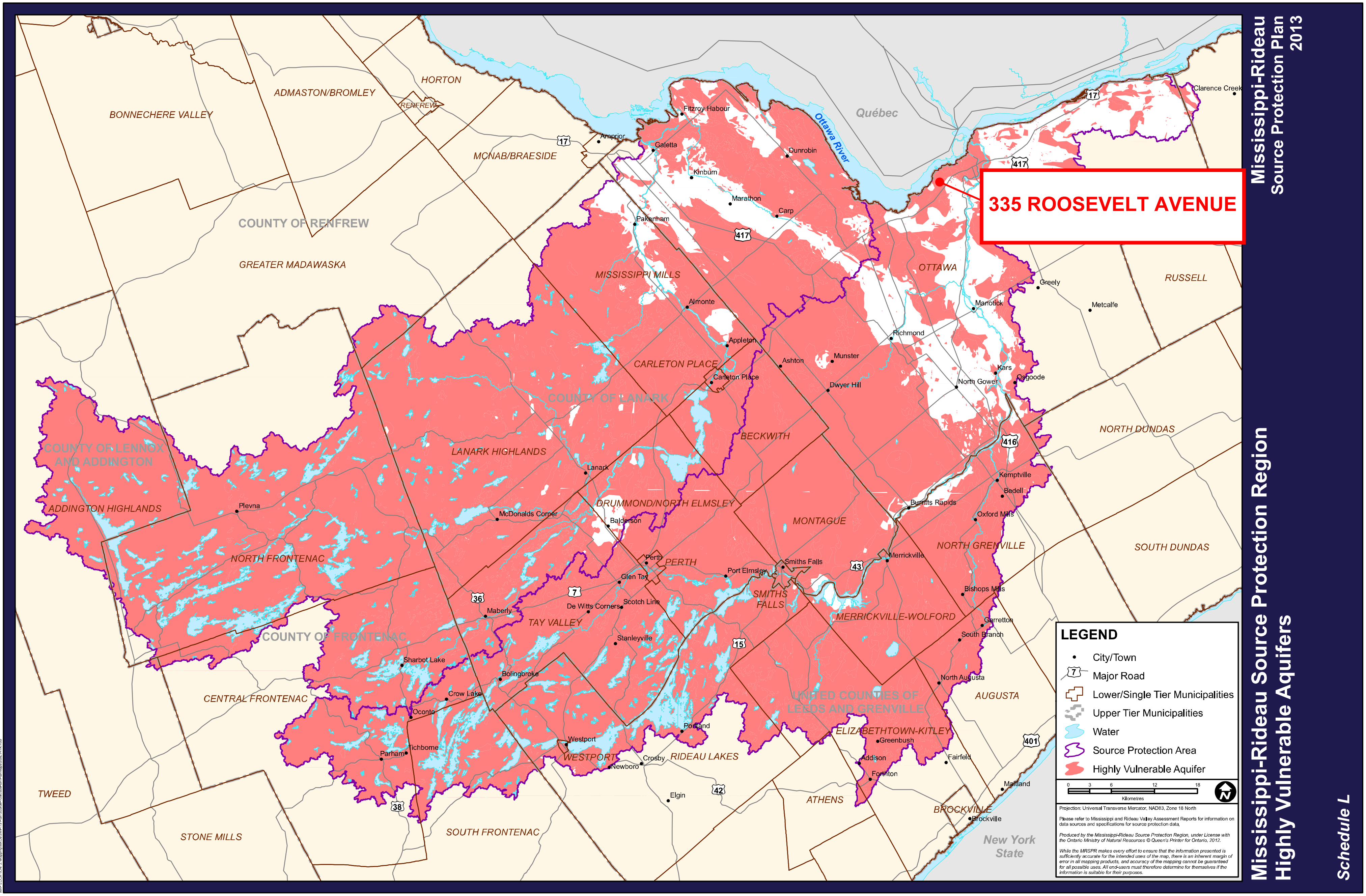
Mississippi-Rideau
Source Protection Plan
2013
Ottawa
Intake Protection Zones
Schedule I
335 ROOSEVELT AVENUE
Québec
Ottawa River
Britannia
Lemieux Island
8.1
8.1
LEGEND
Municipal Surface Water Intake
Road
Railway
Provincial Boundary
Stream
Water
Source Protection Area
Intake Protection Zone
IPZ Scored 9
IPZ Scored 8.1
0
305
610
1,220
1,830
Meters

Mississippi-Rideau Source Protection Plan 2013



Mississippi-Rideau Source Protection Region
Significant Groundwater Recharge Areas

Schedule M



Mississippi-Rideau
Source Protection Plan
2013
Mississippi-Rideau Source Protection Region
Highly Vulnerable Aquifers
Schedule L
335 ROOSEVELT AVENUE
Québec
Ottawa River
New York State
COUNTY OF RENFREW
COUNTY OF LANARK
COUNTY OF LENNOX AND ADDINGTON
COUNTY OF FRONTENAC
UNITED COUNTIES OF LEEDS AND GRENVILLE
BONNECHERE VALLEY
ADMASTON/BROMLEY
HORTON
RENFREW
MCNAB/BRAESIDE
GREATER MADAWASKA
MISSISSIPPI MILLS
OTTAWA
RUSSELL
CARLETON PLACE
BECKWITH
LANARK HIGHLANDS
NORTH DUNDAS
ADDINGTON HIGHLANDS
DRUMMOND/NORTH ELMSLEY
MONTAGUE
NORTH GRENVILLE
SOUTH DUNDAS
NORTH FRONTENAC
PERTH
TAY VALLEY
SMITHS FALLS
MERRICKVILLE-WOLFORD
CENTRAL FRONTENAC
AUGUSTA
ELIZABETHTOWN-KITLEY
WESTPORT
RIDEAU LAKES
TWEED
ATHENS
BROCKVILLE
STONE MILLS
SOUTH FRONTENAC
Fitzroy Habour
Arnprior
Galetta
Dunrobin
Kinburn
Marathon
Carp
Pakenham
Clarence Creek
Greely
Metcalfe
Manotick
Almonte
Richmond
Appleton
Ashton
Munster
Carleton Place
Dwyer Hill
Kars
North Gower
Osgoode
Lanark
Kemptville
Bedell
Plevna
McDonalds Corner
Burritts Rapids
Oxford Mills
Balderson
Merrickville
Perth
Smiths Falls
Port Elmsley
Glen Tay
De Witts Corners
Scotch Line
Bishops Mills
Maberly
Stanleyville
Garretton
South Branch
Sharbot Lake
Bolingbroke
North Augusta
Crow Lake
Oconto
Portland
Greenbush
Westport
Parham
Tichborne
Crosby
Newboro
Addison
Fairfield
Forthton
Elgin
Maitland
Brockville
17
417
416
7
36
15
43
42
38
401
LEGEND
City/Town
Major Road
Lower/Single Tier Municipalities
Upper Tier Municipalities
Water
Source Protection Area
Highly Vulnerable Aquifer
Kilometres
Projection: Universal Transverse Mercator, NAD83, Zone 18 North

Appendix G
Geotechnical Investigation

patersongroup

Geotechnical Engineering

Environmental Engineering

Hydrogeology

Geological Engineering

Materials Testing

Building Science

Geotechnical Investigation

Proposed Residential Development
335 Roosevelt Avenue
Ottawa, Ontario

Prepared For

Uniform Development

July 26, 2011

Report: PG2178-1

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TABLE OF CONTENTS

APPENDICES

1.0 INTRODUCTION

Paterson Group (Paterson) was commissioned by Uniform Developments (Uniform) to prepare a geotechnical report for a proposed residential development to be located at 335 Roosevelt Avenue in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were 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 the design.

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of this present investigation, therefore, the present report does not address environmental issues. A Phase I-II was completed for this subject site by Paterson but is presented under a separate cover.

2.0 PROPOSED DEVELOPMENT

It is our understanding that the proposed residential development will consist of two (2) high-rise residential buildings to be located on the eastern and western portion of the property. The buildings are expected to be 10 and 13 storeys high. There will be two (2) levels of underground parking that is understood to encompass the entire site.

3.0 METHOD OF INVESTIGATION

3.1 Field Investigation

Field Program

The field program for the investigation was carried out on November 9 and 10, 2010. At that time, five (5) boreholes were advanced to a maximum depth of 9.5 m. The borehole locations were distributed in a manner to provide general coverage of the subject site. The locations of the boreholes are shown on Drawing PG2178-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were put down using a truck-mounted auger drill rig operated by a two-person crew. All fieldwork was conducted under the full-time supervision of Paterson personnel under the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations, sampling and testing the overburden. In addition, bedrock was cored at each borehole location using diamond drilling procedures.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site, placed in sealed plastic bags, and transported to our laboratory for further review. The depths at which the split-spoon and auger samples were recovered from the boreholes are shown as SS and AU, respectively, on the Soil Profile and Test Data sheets 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.

Diamond drilling was carried out in each borehole to determine the nature of the bedrock. Total core recovery (TCR) and rock quality designation (RQD) values were calculated for each drilled section (core run) of bedrock and are shown on the borehole logs. The TCR value is the ratio, in percentage, of the length of the bedrock sample recovered over the length of the core run. The RQD value is the ratio, in percentage, of the total length of rock pieces longer than 100 mm in one core run over the length of the core run. Each of these values are indicative of the quality of the bedrock.

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

Groundwater

A flexible polyethylene standpipe was installed in BH 1, BH 2 and BH 4. PVC monitoring wells (50 mm diameter) were installed in BH 3 and BH 5. These were installed to permit the monitoring of the groundwater level subsequent to the completion of the sampling program.

3.2 Field Survey

The borehole locations were selected, determined in the field and surveyed by Paterson. The ground surface elevation at each borehole location was referenced to a temporary benchmark (TBM), consisting of a magnetic nail in a utility pole. A geodetic elevation of 67.30 m has been provided to the TBM by Annis O'Sullivan Vollebekk Ltd. The location of the TBM and boreholes, as well as, the ground surface elevation at each borehole are presented on Drawing PG2178-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

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

4.0 OBSERVATIONS

4.1 Surface Conditions

At the time of the field program, three (3) existing buildings were present on the subject site. The remainder of the site was asphalt covered with the exception of a gravel area on the south portion of the property.

The site is bordered to the north by the transitway, to the west by Roosevelt Avenue, to the south by Winston Avenue and Wilmont Avenue, and to the east by a 7 storey residential building. The westernmost building was noted to be approximately 0.6 m below Roosevelt Avenue. Additionally, the transit-way located north of the subject site was noted to be approximately 6 m below the elevation of 335 Roosevelt Avenue. The subject site is relatively flat.

4.2 Subsurface Profile

The subsurface profile at the borehole locations consist of either asphaltic concrete or silty sand fill overlying fill consisting of silty sand with some gravel and clay. Native silty clay or silt was encountered below the fill material at most of the boreholes. Bedrock was encountered at depths between 0.7 and 1 m depths. Specific details of the soil profile at each borehole location are presented on the Soil Profile and Test Data sheets in Appendix 1.

The bedrock was cored at all borehole locations to determine its nature and quality. Based on the results of coring, the bedrock consists of limestone with layers of black shale. Values for TCR and RQD were calculated for each rock core and the quality of the bedrock was assessed based on these results.

Based on the observations, the upper 0.5 to 2 m of the bedrock is of poor to fair quality while the lower portion of the core is of good to excellent quality. The bedrock consists of limestone with interbedded shale, with a black shale limestone extending through the rock at depths between 1.5 and 3 m.

Based on available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and dolomite of the Gull River formation, which is encountered at depths varying between 1 and 2 m.

4.3 Groundwater

Groundwater levels (GWL) were measured in all boreholes on November 16, 2010. The measured GWL readings are presented in Table 2. It should be noted that groundwater levels are subject to seasonal fluctuations. Therefore, the groundwater level could vary at the time of construction.

Table 1 - Groundwater Level Readings

Borehole Number	Ground Elevation (m)	Groundwater Levels		Recording Date
		Depth (m)	Elevation (m)	
BH 1	66.39	4.88	61.51	November 16, 2010
BH 2	66.37	6.53	59.84	November 16, 2010
BH 3	66.43	Dry	--	November 16, 2010
BH 4	66.64	3.84	62.80	November 16, 2010
BH 5	66.50	4.97	61.53	November 16, 2010

5.0 DISCUSSION

5.1 Geotechnical Assessment

From a geotechnical point of view, the subject site is considered suitable for the proposed development.

Considering that the site is underlain by shallow bedrock (within 1 m of the surface), shoring may not be necessary if the excavation of the overburden soils can be stepped back from the bedrock excavation face. Temporary rock bolts may be required to stabilize the walls of the excavation through bedrock.

Bedrock excavation is expected for the construction of the underground parking levels of the proposed residential development. Line drilling of the perimeter and rock blasting and/or hoe ramming are expected for the removal of the bedrock.

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

5.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed building, it is anticipated that all existing overburden material will be excavated. Bedrock excavation will be required for the construction of the underground parking garage.

Bedrock Removal

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

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

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

An existing watermain is located directly north of the subject site, between the property line and the transitway. It is recommended that bedrock removal be completed by hoe ramming in close proximity to the watermain. Vibration monitors should be installed on the watermain to measure the vibrations and to ensure that it stays below the recommended guideline of 15 mm/s. Refer to Subsection 6.7 of this report for further information.

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

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

Vibration Considerations

Construction operations are also the cause of 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, as much as possible, a cooperative environment with the residents.

The following construction equipment could be the source of vibrations: hoe ram, compactor, crane, truck traffic, etc. Vibrations, whether it is caused by blasting operations or by construction operations could be the cause of the source of detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations be limited.

Fill Placement

It is expected that a concrete slab will be poured directly over bedrock; therefore, fill used for grading beneath building will not be required, other than around the footings, as required.

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill where settlement of the ground surface is of minor concern. 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 standard Proctor maximum dry density (SPMDD).

Excavated shale deteriorates upon exposure to air and is not generally suitable for re-use as an engineered fill.

5.3 Foundation Design

It is understood that footings will be founded on bedrock. Footings placed on a clean, surface sounded bedrock surface at this elevation can be designed using a bearing resistance value at serviceability limit states (SLS) of **1,000 kPa** and a factored bearing resistance value at ultimate limit states (ULS) of **1,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.

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.

A bearing resistance value at SLS of **2,000 kPa** and a factored bearing resistance value at ULS of **3,500 kPa** could be used if the bedrock is free of seams, fractures and voids within 1.5 m below the bedrock surface. This could be verified by completing and probing 50 mm diameter drill holes to a depth of 1.5 m below the founding level along the footing alignments. The drill holes should be spaced on about a 10 m grid interval or one (1) hole per significant pad footing. The drill hole inspection should be carried out by the geotechnical consultant.

Footings bearing on surface sounded bedrock and designed using the above mentioned bearing pressures will be subjected to negligible post-construction total and differential settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as Class C for the foundations considered at this site. Soils underlying the subject site are not susceptible to liquefaction. A higher site class, such as Class B or A, could be applicable for this subject site. However, this should be confirmed with site specific shear wave velocity testing. For preliminary design purposes, a Site Class A can be used. Reference should be made to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

5.5 Basement Wall

It is understood that the basement walls are to be poured against a waterproofing system, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.25 is recommended in conjunction with a bulk unit weight of 24.5 kN/m^3 (effective 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.

Where soil is to be retained, there are several combinations of backfill materials and retained soils that could be applicable for the proposed retaining walls and basement walls. However, provided free-draining granular backfill is used, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a drained unit weight of 20 kN/m^3. It is anticipated that the soils against the foundation wall will be drained. An interface friction angle of 17 degrees between the wall and the backfill material is applicable for the abovenoted parameters. For undrained conditions, the effective unit weight of soil (13 kN/m^3) should be used to calculate the earth pressure component below the groundwater table, and hydrostatic pressure should be added within this portion to calculate the total static earth pressure.

The earth pressures acting on earth retaining structures are dependent on the characteristics of the structure, particularly with respect to whether it is a “yielding” or an “unyielding” structure. A basement wall, which is restrained laterally by the floors of the structure, is generally considered to be an unyielding structure. It is recommended that the at-rest earth pressure case be used for basement walls under static conditions.

During an earthquake event, a basement wall is considered to be a “yielding” earth retaining structure, due to the magnitude of wall rotation. Therefore, an active earth pressure should be calculated for seismic design considerations.

Two (2) distinct conditions, static and seismic, must be reviewed for design calculations. The parameters for design calculations for the two (2) conditions are presented below.

Static Earth Pressures

Under static conditions, the retaining walls and basement walls may be designed using a triangular earth pressure distribution with a maximum stress value at the base of the wall equal to $K_o \gamma H$ where:

K_o - At-rest earth pressure coefficient = 0.5
γ - unit weight of the fill = 20 kN/m^3
H - height of the retained fill against the wall, m

An additional pressure having a magnitude equal to $K_o q$ and acting on the entire height of the wall must be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall.

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

The earth pressures acting on earth retaining structures are dependent on the characteristics of the structure, particularly with respect to whether it is a “yielding” or an “unyielding” structure. A basement wall, which is restrained laterally by the floors of the structure, is generally considered to be an unyielding structure. It is recommended that the at-rest earth pressure case be used for basement walls under static conditions.

During an earthquake event, a basement wall is considered to be a “yielding” earth retaining structure, due to the magnitude of wall rotation. Therefore, an active earth pressure should be calculated for seismic design considerations.

Seismic Earth Pressures

Seismic loading conditions influence the earth pressures that will act on earth retaining structures during seismic events. In Ottawa, the peak ground acceleration (PGA) is 0.42 for the OBC 2006.

The magnitude of seismic earth pressures acting on a structure is dependent upon the relative flexibility of the structure. Isolated free-standing retaining walls are generally flexible enough to be considered as "yielding" earth retaining structures. During an earthquake event, a basement wall is considered to be a "yielding" earth retaining structure, due to the magnitude of wall rotation.

The total active earth force acting on a wall under seismic conditions can be estimated using a pseudo-static approach based on the Mononobe-Okabe (M-O) Method. The seismic intensity is represented by the horizontal seismic coefficient, k_h. For yielding structures, the value of k_h can be taken to be one half of PGA. Note that the vertical seismic coefficient is taken to be zero.

The M-O Method is used to calculate the total active earth pressure (P_{AE}). The resulting force is then split into the static (active) (P_A) and seismic component (ΔP_{AE}). The total active earth pressure (P_{AE}) can be calculated using $0.5K_{AE}\ \gamma H^2$ where:

K_{AE} - Dynamic active earth pressure coefficient. For the conditions previously stated, K_{AE} is 0.21.
γ - unit weight of the fill of the applicable retained soil (kN/m^3)
H - height of the wall (m)

The static component (P_A) can be calculated using $K_A\ \gamma\ H$ where:

K_A = dynamic active earth pressure coefficient, 0.33
γ = unit weight of the fill of the applicable retained soil (kN/m^3)
H = height of the wall (m)

The dynamic seismic component (ΔP_{AE}) can be calculated by $\Delta P_{AE} = P_{AE} - P_A$.

The static component (P_A) is a conventional triangular shaped pressure distribution with the resultant located H/3 up from the wall base. The seismic component (ΔP_{AE}) is acting approximately 0.6H up from the wall base.

On this basis, the total active pressure (P_{AE}) will act from a height:

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

The earth pressures calculated are unfactored. For the ULS case, the earth pressure loads must be factored as live loads, as per OBC 2006.

5.6 Rock Anchor Design

It is expected that rock anchors will be required to resist hydrostatic uplift forces. The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

It is expected that the centre to centre spacing between the grid of rock anchors will be 3.0 m. Assuming an apex angle of 60° for the failure cone, it is likely that interaction will develop between failure cones of anchors. As a result, the following recommendations are provided on the assumption that group interaction will occur between the anchors. The effect of assuming group interaction is a reduction in the overall strength of each anchor; therefore, this assumption is considered conservative.

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

It is also recommended, where applicable, that anchors in close proximity to each other be grouted at the same time. This will ensure that any fractures or voids are completely in-filled and that fluid grout does not flow from a grouted hole to an adjacent empty hole.

Anchors can be of the “passive” or the “post-tensioned” type, depending on whether the anchor tendon is provided with a post-tensioned load prior to being put into service. To resist hydrostatic uplift pressures, a passive rock anchor system can be used. It should be noted that a post-tensioned anchor will take the uplift load with much less potential deflection than a passive anchor.

Regardless of whether an anchor is of the passive or the post tensioned type, it is recommended that the anchor be provided with a bonded length, or fixed anchor length, at the base of the anchor, which will provide the anchor capacity. In addition, each anchor should have an unbonded length, or free anchor length, between the rock surface and the start of the bonded length. Since the depth at which the apex of the shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less geotechnical resistance, than one where the bonded length is limited to the bottom part of the overall anchor.

Permanent anchors should be provided with corrosion protection. As a minimum, this requires that the entire drill hole be filled with cement grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break within the fully grouted drill hole.

Grout to Rock Bond

Generally, the unconfined compressive strength of limestone ranges between about 60 and 90 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.2 MPa**, incorporating a resistance factor of 0.3, can be used. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

As discussed previously, the geotechnical capacity of the rock anchors depends on the dimensions of the rock anchors and the configuration of the anchorage system. A **Rock Mass Rating (RMR) of 50** was assigned to the bedrock, and Hoek and Brown parameters (**m and s**) were taken as **0.128 and 0.00009**, respectively. For design purposes, we assumed that all rock anchors will be placed at least 1.2 m apart to reduce group anchor effects.

Recommended Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.2 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Fair quality Shale Hoek and Brown parameters	50 m=0.128 and s=0.00009
Unconfined compressive strength - Limestone bedrock	60 MPa
Unit weight - Submerged Bedrock	15 kN/m^3
Apex angle of failure cone	60°
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the diameter of the drill holes. Recommended anchor lengths for a 50, 75 and 100 mm diameter hole are provided in Table 3. The anchor lengths are designed to resist a **900 kN force** with a **3.0 m centre-to-centre spacing.**

Table 3 - Recommended Rock Anchor Lengths			
Diameter of Drill Hole (mm)	**Anchor Lengths (m)**		
	Bonded Length	**Unbonded Length**	**Total Length**
50	5.75	0.75	6.5
75	3.8	1.7	5.5
100	2.9	2.2	5.1

It is recommended that the anchor drill hole diameter be a minimum of 2 times the rock anchor tendon diameter. The anchor drill holes should be inspected by geotechnical personnel and should be thoroughly flushed clean prior to grouting. The use of a grout tube to place grout from the bottom up in the anchor holes is further recommended.

It should be noted that due to the intended use of the rock anchors and nature of the passive rock anchor design, proof testing is not required provided that the grout installation is adequately completed to the satisfaction of the geotechnical consultant. It is recommended that compressive strength testing be completed for the rock anchor grout. A set of grout cubes, consisting of 3 “gangs” of 3 cubes each, should be tested for each day that grout is prepared.

5.7 Pavement Design

Asphalt pavement is not anticipated to be required at the subject site. However, should pavement be reconsidered for the project, the recommended pavement structures shown in Tables 4 and 5 would be applicable.

Table 4 - 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 5 - Recommended Pavement Structure - Access Lanes

Thickness mm	Material Description
40	**Wear Course -** HL-3 or Superpave 12.5 Asphaltic Concrete
50	**Binder Course -** HL-8 or Superpave 19.0 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
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 and replaced with Ontario Provincial Standard Specification (OPSS) Granular B Type I or Type II material.

The pavement granulars (base and subbase) should be placed in maximum 300 mm thick layers and compacted to a minimum of 100% of the materials' SPMDDs using suitable compaction equipment.

6.0 DESIGN AND CONSTRUCTION PRECAUTIONS

6.1 Foundation Drainage and Backfill

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is understood that insufficient room is available for exterior backfill below the bedrock surface. The following system is suggested:

- ❏ Bedrock vertical surface
- ❏ Metal “V” pan
- ❏ Composite drainage layer

It is recommended that the composite drainage system (such as Miradrain G100N 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 may be required to control water infiltration due to groundwater lowering within the bedrock. For design purposes, we recommend that 100 or 150 mm in perforated pipes be placed at 3 to 4.5 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.

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 of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided in this regard.

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

6.3 Excavation Side Slopes

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 a Type 2 and 3 soil according to the Occupational Health and Safety Act and Regulations for Construction Projects.

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

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

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

6.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for pipe bedding for sewer and water pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the material’s SPMDD.

It is expected that the silt may be used above cover material if the excavation operations are carried out in dry weather conditions. Well fractured bedrock should be acceptable as backfill provided the rock fill is placed only from at least 300 mm above the top of the service pipe and that all stones 200 mm or larger in their longest dimension are removed.

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 differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material's SPMDD. No stones 200 mm or greater in their longest dimension should be reused. Within the frost zone (1.8 m below finished grade), non frost susceptible materials should be used when backfilling trenches below the original bedrock level.

6.5 Groundwater Control

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.

The flow of groundwater into the excavation through the overburden materials is expected to be controllable using properly sized pumps and sumps.

A temporary Ontario Ministry of Environment (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. At least 3 to 4 months should be allowed for completion of the application and issuance of the permit by the MOE. The permits are valid for a period of one (1) year from the time of issuance.

6.6 Winter Construction

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

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

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the use of straw, propane heaters and tarpaulins or other suitable means. In this regard, the base of the excavations should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and/or the footings are protected with sufficient soil cover to prevent freezing at the founding level. Placing concrete directly over cold bedrock surfaces is not recommended.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice in the trenches. As well, pavement construction is difficult during winter. The subgrade consists of frost susceptible soils which will experience total and differential frost heaving as the work takes place. Also, the introduction of frost, snow or ice into the pavement materials, which is difficult to avoid, could adversely affect the performance of the pavement structure.

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

6.7 Protection of Existing Watermain

During the bedrock removal program for the proposed development, the existing watermain along the northern boundary of the subject site will require protection.

Bedrock Condition

Based on the recent findings, the bedrock was encountered at an average depth of 770 mm which is approximately at elevation 65.63 m.

The bedrock quality improves with depth. The upper portion of the bedrock is relatively fractured and weathered to approximately elevation 65 m which is approximately 0.9 m below the existing grade. Below this elevation, the bedrock quality is generally fair to good based on the rock quality designation (RQD) findings.

During an exploratory investigation to determine the bedrock condition adjacent to the watermain, two test pits (TP-1 and TP-2) were excavated using a vacuum truck. The bedrock appeared to be intact and in close proximity to the watermain. Our photographs and test hole location plan are enclosed for your records.

Paterson also undertook a test pit excavation program on the subject property along the northern boundary on September 13, 2010. Three test pits were excavated using a rubber tired backhoe and our findings can be summarized as follows:

Subsurface Conditions	Test Pit 1	Test Pit 2	Test Pit 3
Pavement structure overlying sandy silt deposit thickness	810 mm	810 mm	710 mm
Weathered bedrock thickness	100 mm	none	none
Sound bedrock depth	910 mm	810 mm	710 mm

A sketch of the test pit locations is enclosed.

Bedrock Removal along the Northern Boundary

The bedrock removal for the subject site will be carried out using a combination of blasting and hoe-ramming techniques, especially along the northern boundary where the existing watermain is located. The bedrock removal along the northern boundary will be carried out as follows:

❏ For the bedrock removal program along the northern boundary adjacent to the watermain will be set at a **minimum of 2 m from the outer edge of the existing watermain**. The bedrock within this 2 m section will be reinforced as noted below. This reinforcement of the bedrock will be applicable for approximately the eastern third of the northern portion. No bedrock reinforcement will be required when the bedrock excavation face is greater than 2 m from the existing watermain outer edge.

❏ Prior to undertaking any blasting in close proximity to the watermain, it is recommended that the bedrock ledge be reinforced along the watermain in the north east boundary of the subject site. To accomplish this reinforcement, we suggest that 4 m deep core holes be drilled at every 450 mm centres and that a vertical 25 mm in diameter reinforcing steel bars be grouted (40 MPa grout) to full depth in each core hole (minimum 50 mm diameter hole). The location of the bedrock reinforcement will be 300 mm from the final excavation bedrock face (1.7 m from the outer edge of the existing watermain).

❏ After the removal of the bedrock, the final face of the excavation will be reviewed by a geotechnical engineer to determine if further reinforcement is required (rock bolts or anchors).

- The purpose of the reinforcement within the bedrock will provide stability to the rock face and will prevent any lateral movement of the rock mass during the excavation program. This type of reinforcement is often used to maintain a vertical rock cut where a hi-rise building is being supported at the edge of the excavation where no undermining or movement can be tolerated. Therefore, this will be an appropriate methodology for the lateral support of the watermain.

- The blasting and excavation contractor will be responsible for the submission of a blasting and excavation work procedure document for approval prior to undertaking the blasting program. The sensitivity of the watermain and the above requirements will be incorporated in the project specifications prior to tendering and to inform the contractors.

- Blasting can be used for most of the bedrock removal up to a minimum distance of 2.4 m from the outer edge of the existing watermain. Subject to monitoring, a minimum line drilling spacing of 300 mm c/c will be required at the 2.4 m blasting boundary limit.

- The blasting contractor will control the blasting operation to keep peak particle velocities below 15 mm/s at the property boundary. It is expected that the blasting contractor will commence the blasting operation at the opposite end of the site so that blasting patterns and vibrations can be monitored and verified prior to attempting any blasting along the northern boundary adjacent to the existing watermain. This approach will allow the blasting contractor to adjust and control the blasting operation. Furthermore, the first blast will be used as a test blast and vibration monitoring equipment will be installed in close proximity to the blast to resemble the potential conditions that can be experienced along the northern boundary. Paterson personnel along with City personnel and associated consultants will be invited to attend and witness the test blast program.

- Blasting operations will be reviewed and the 2.4 m minimum distance from the watermain may be increased if vibrations from the blasting operation are questionable.

- Within the minimum 2.4 m distance from the watermain, the bedrock will be removed using hoe-ramming or grinding techniques. Blasting will not be permitted. Line drilling spacing will be decreased to 200 mm c/c along the proposed excavation boundary. Similar to the blasting operations, hoe-ramming or grinding operations will be governed by the vibrations they produce along the property boundary adjacent to the watermain.

Monitoring and Reporting

- Two seismographs will be installed directly on the bedrock along the northern property line to monitor vibrations. Each blasting event will be reviewed and reported to the blasting contractor and the site superintendent. One of the seismographs can be installed directly on the watermain (manhole location). The seismograph installed on watermain can be moved to other locations in line with the blasting provided a manhole is available for access.

- A weekly summary report will be issued presenting our findings and observations. Any concerns identified during the monitoring will be immediately reported and the rock removal operations in the immediate area will be temporarily halted to address the concern.

7.0 RECOMMENDATIONS

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

- ❑ Review of the bedrock excavation faces and the installation of the rock anchors, if applicable.
- ❑ Observation of all bearing surfaces prior to the placement of concrete.
- ❑ Sampling and testing of the concrete and fill materials used.
- ❑ Periodic observation of the condition of unsupported excavation side slopes in excess of 3.0 m in height, if applicable.
- ❑ Observation of all subgrades prior to backfilling.
- ❑ Density tests to determine the level of compaction achieved.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon 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 that we be permitted to review our recommendations when the drawings and specifications are complete.

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

A geotechnical 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 Uniform Developments and 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.

Stephanie Boisvenue, B.Eng.

Carlos P. Da Silva, P.Eng.

LICENSED PROFESSIONAL ENGINEER
C. P. DA SILVA
PROVINCE OF ONTARIO

Report Distribution:

- ❑ Uniform Development (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

PROTECTION OF EXISTING WATERMAIN INFORMATION

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** 9 November 2010

FILE NO. PG2178

HOLE NO. BH 1

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Dark brown silty sand, some gravel	0.46	SS	2		50+
Grey-brown **SILTY CLAY,** some sand	0.74	RC	1	94	22
BEDROCK: Grey limestone - black shaley limestone from 1.5 to 2.5m depth	9.47	RC	2	100	51
		RC	3	100	78
		RC	4	100	84
		RC	5	100	35
		RC	6	100	96
		RC	7	100	100
End of Borehole (GWL @ 4.88m-Nov. 16/10)					

DEPTH (m)	ELEV. (m)
0	66.39
1	65.39
2	64.39
3	63.39
4	62.39
5	61.39
6	60.39
7	59.39
8	58.39
9	57.39

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

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 2

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.37
Asphaltic concrete 0.05							
FILL: Dark brown silty sand, some gravel 0.46		AU	1				
Grey-brown SILTY CLAY 0.60		SS	2	0	50+		
		RC	1	84	58	1	65.37
		RC	2	100	57	2	64.37
BEDROCK: Grey limestone - black shaley limestone from 1.6m to 2.5m depth						3	63.37
		RC	3	98	83		
						4	62.37
		RC	4	100	92	5	61.37
						6	60.37
		RC	5	98	98		
						7	59.37
		RC	6	100	93	8	58.37
						9	57.37
9.42		SS	7	100	100		
End of Borehole (GWL @ 6.53m-Nov. 16/10)							

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

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 9 November 2010

FILE NO. PG2178

HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL; Brown silty sand with gravel	0.60				
FILL: Light brown silty sand, some gravel, trace clay	0.97	SS	2	0	50+
BEDROCK: Grey limestone - black shaley limestone from 1.5m to 1.7m depth	9.40	RC	1	88	63
		RC	2	97	35
		RC	3	100	75
		RC	4	98	87
		RC	5	98	85
		RC	6	100	89
		RC	7	96	96
End of Borehole (BH dry - Nov. 16/10)					

DEPTH (m)	ELEV. (m)
0	66.43
1	65.43
2	64.43
3	63.43
4	62.43
5	61.43
6	60.43
7	59.43
8	58.43
9	57.43

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

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 November 2010

FILE NO. PG2178

HOLE NO. BH 4

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.64
Asphaltic concrete	0.04						
Crushed stone	0.15	AU	1				
FILL: Brown silty sand, some gravel	0.91	SS	2	90	10	1	65.64
Grey-brown **SILT**	1.17						
BEDROCK: Shaley limestone	3.00	RC	1	92	36	2	64.64
BEDROCK: Grey limestone		RC	2	100	55	3	63.64
		RC	3	100	60	4	62.64
						5	61.64
		RC	4	100	93	6	60.64
						7	59.64
		RC	5	100	92	8	58.64
	9.37	RC	6	100	100	9	57.64
End of Borehole							
(GWL @ 3.84m-Nov. 16/10)							

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

Piezometer Construction

patersongroup Consulting Engineers

28 Concourse Gate, Unit 1, Ottawa, ON K2E 7T7

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Residential Development-335 Roosevelt Ave.
Ottawa, Ontario

DATUM TBM - Mag nail in utility pole, along southeast property line. Geodetic elevation = 67.30m.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 10 November 2010

FILE NO. PG2178

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	66.50
FILL: Dark brown silty sand with gravel, trace metals, brick and asphalt — 0.76		SS	1	50	8		
FILL: Black silty sand — 0.94		SS	2	78	50+		
Light brown **SILT** — 0.99		RC	1	86	0	1	65.50
BEDROCK: Grey limestone - shaley limestone from 1.9m to 2.0m depth		RC	2	97	45	2	64.50
						3	63.50
		RC	3	98	85		
						4	62.50
		RC	4	100	77	5	61.50
						6	60.50
		RC	5	100	95		
						7	59.50
		RC	6	100	82	8	58.50
		RC	7	100	83	9	57.50
9.40							
End of Borehole (GWL @ 4.97m-Nov. 16/10)							

Pen. Resist. Blows/0.3m
● 50 mm Dia. Cone
○ Water Content %
20 40 60 80

Piezometer Construction

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

SYMBOLS AND TERMS

SOIL DESCRIPTION

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

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

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

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

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

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

SYMBOLS AND TERMS (continued)

SOIL DESCRIPTION (continued)

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

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

ROCK DESCRIPTION

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

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

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

SAMPLE TYPES

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

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

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

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

CONSOLIDATION TEST

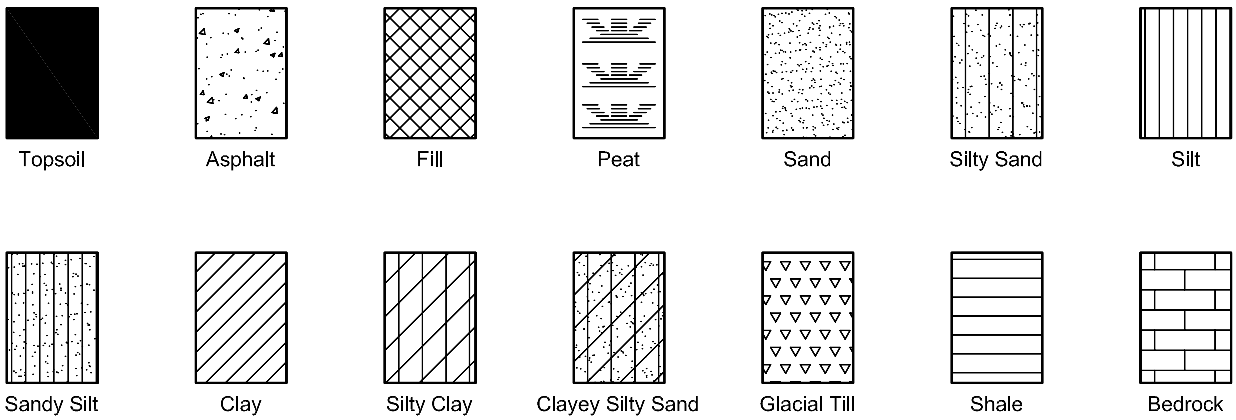
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

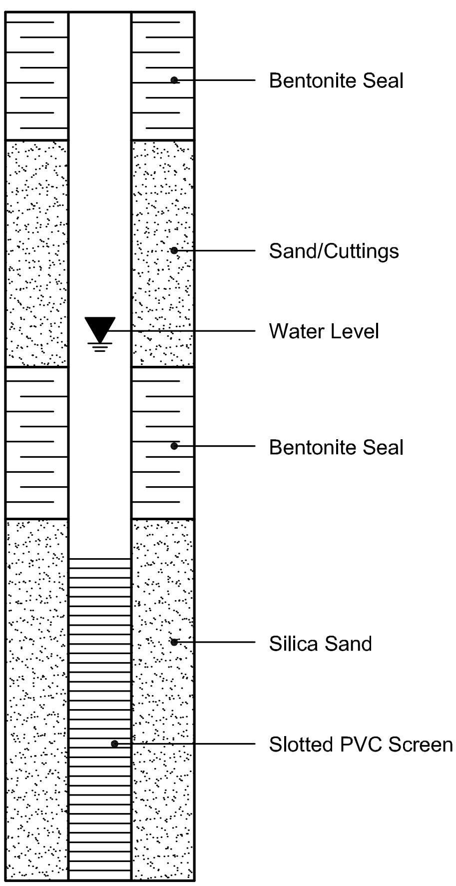
SYMBOLS AND TERMS (continued)

STRATA PLOT

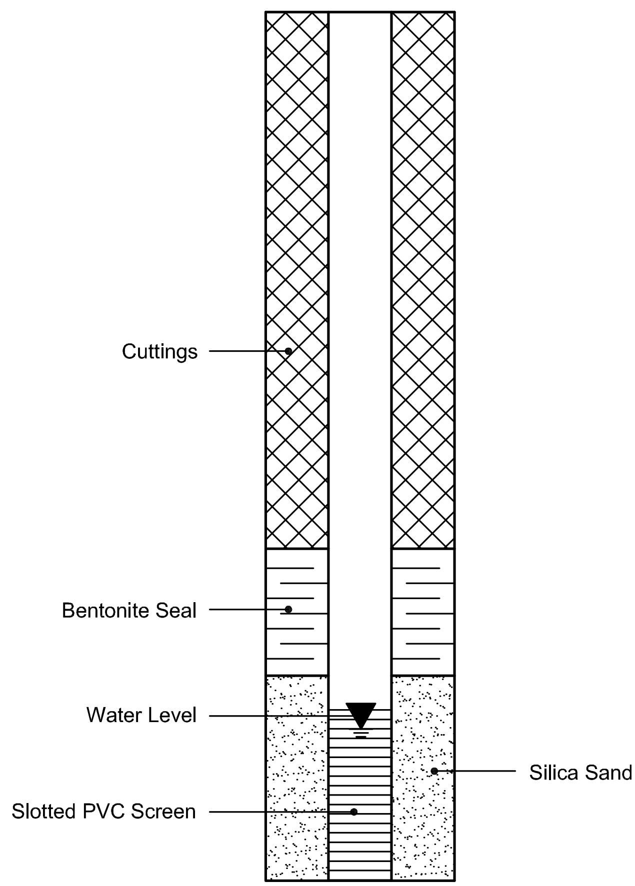


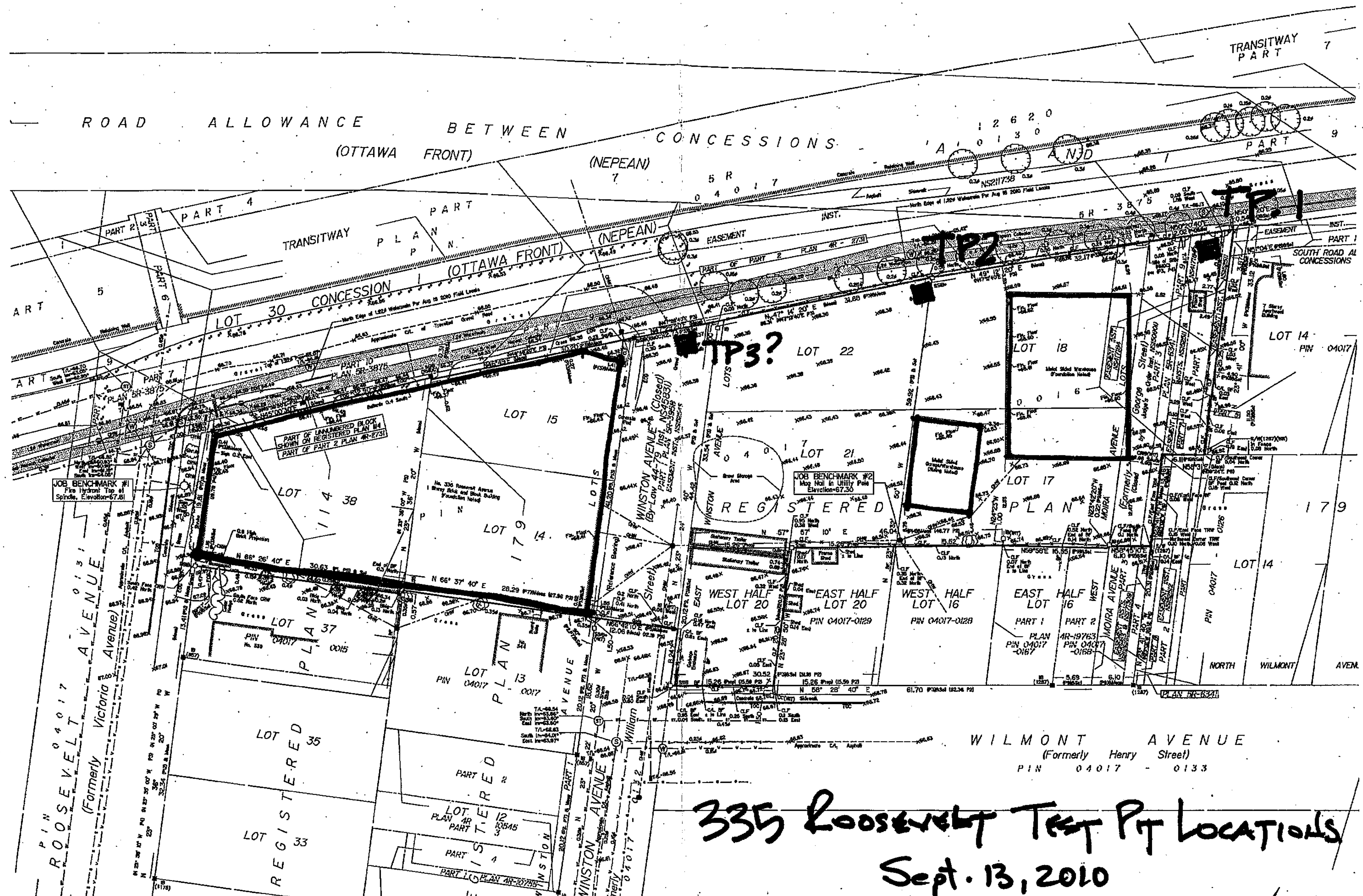
MONITORING WELL AND PIEZOMETER CONSTRUCTION

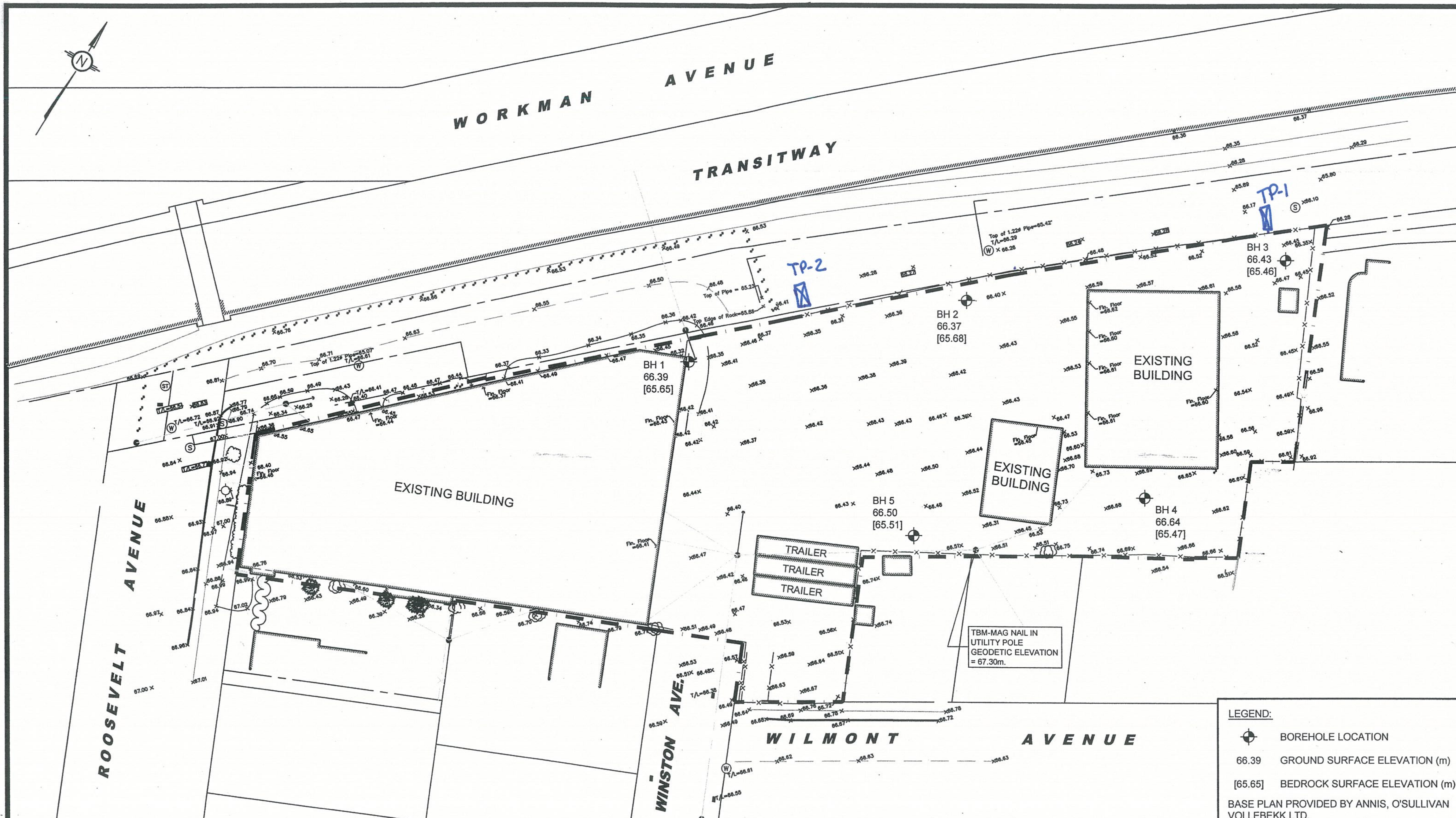
MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION




ROAD ALLOWANCE BETWEEN CONCESSIONS 'A' AND 1
(OTTAWA FRONT)
(NEPEAN)
TRANSITWAY
LOT 30 CONCESSION 1 (OTTAWA FRONT) (NEPEAN)
TP 1
TP2
TP3?
LOT 22
LOT 18
LOT 15
LOT 14
LOT 21
LOT 17
LOT 38
JOB BENCHMARK #1
JOB BENCHMARK #2
REGISTERED PLAN 179
WINSTON AVENUE (Closed)
WEST HALF LOT 20
EAST HALF LOT 20
WEST HALF LOT 16
EAST HALF LOT 16
LOT 37
LOT 13
LOT 35
LOT 33
LOT 12
ROOSEVELT AVENUE (Formerly Victoria Avenue)
WILMONT AVENUE (Formerly Henry Street)
WINSTON AVENUE
MOIRA AVENUE
335 Roosevelt Test Pit Locations
Sept. 13, 2010



paterson group consulting engineers 28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7	Scale: 1:500 Des.: SB Dwn: MPG Chkd: DG	UNIFORM DEVELOPMENTS GEOTECHNICAL INVESTIGATION PROP. RESIDENTIAL DEVELOPMENT-335 ROOSEVELT AVE. OTTAWA, ONTARIO	TEST HOLE LOCATION PLAN

Dwg. No.	PG2178-1
Report No.:	PG2178-1
Date:	11/2010

11x17

Access Manhole North West of Test Pit 2

Test Pit 1

Access Manhole North West of Test Pit 2

Test Pit 1

Test Pit 2

Test Pit 3

Test Pit 2

Test Pit 3

Test Pit 3

Test Pit 3

Test Pit 3

Test Pit 3

Test Pit 3

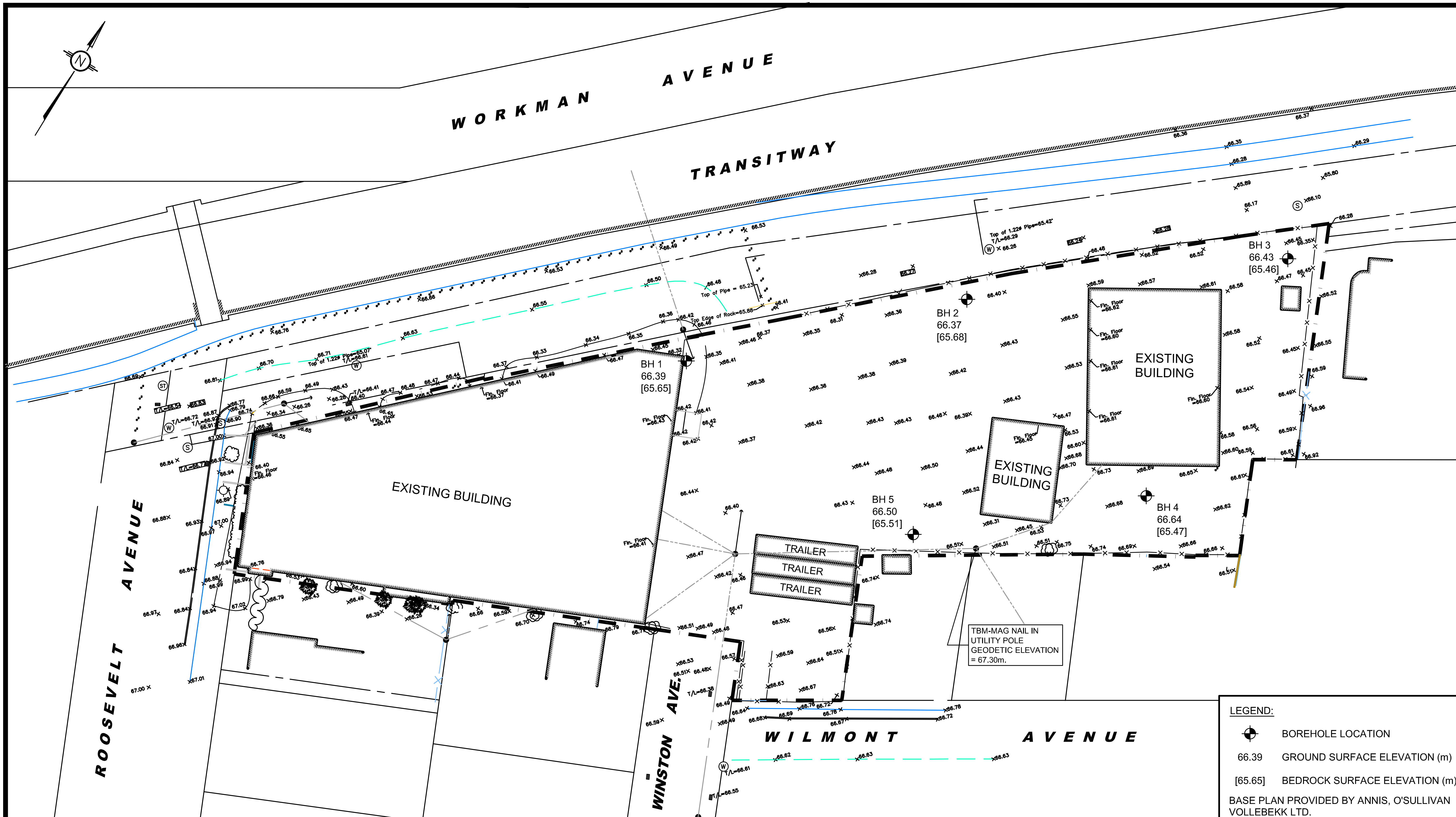
APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG2178-1 - TEST HOLE LOCATION PLAN

FIGURE 1

KEY PLAN



paterson group
consulting engineers
28 Concourse Gate, Unit 1, Ottawa, Ontario K2E 7T7

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UNIFORM DEVELOPMENTS

GEOTECHNICAL INVESTIGATION
PROP. RESIDENTIAL DEVELOPMENT-335 ROOSEVELT AVE.
OTTAWA, ONTARIO

TEST HOLE LOCATION PLAN

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