320 McRae Avenue

Site Servicing & Stormwater Management Report

SITE PLAN SUBMISSION

GWL Realty Advisors

RVA 194453

April 30, 2021

320 McRae Avenue
Site Servicing & Stormwater Management Report

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

APPENDICES

1.0 INTRODUCTION

R.V. Anderson Associates Limited has been retained by GWL Realty Advisors to provide the site servicing design and stormwater management for the proposed residential and commercial complex at 320 McRae Avenue and 1976 Scott Street. This report will outline the proposed stormwater management measures and site services that will be implemented with the site to be in compliance with the City of Ottawa requirements. The report also addresses the comments received on June 12, 2020.

1.1 Site Description

The site is located at 320 McRae Avenue in the City of Ottawa. It is currently occupied by a one-storey commercial building facing McRae Avenue and two single family homes on Tweedsmuir Avenue as shown in Figure 1.

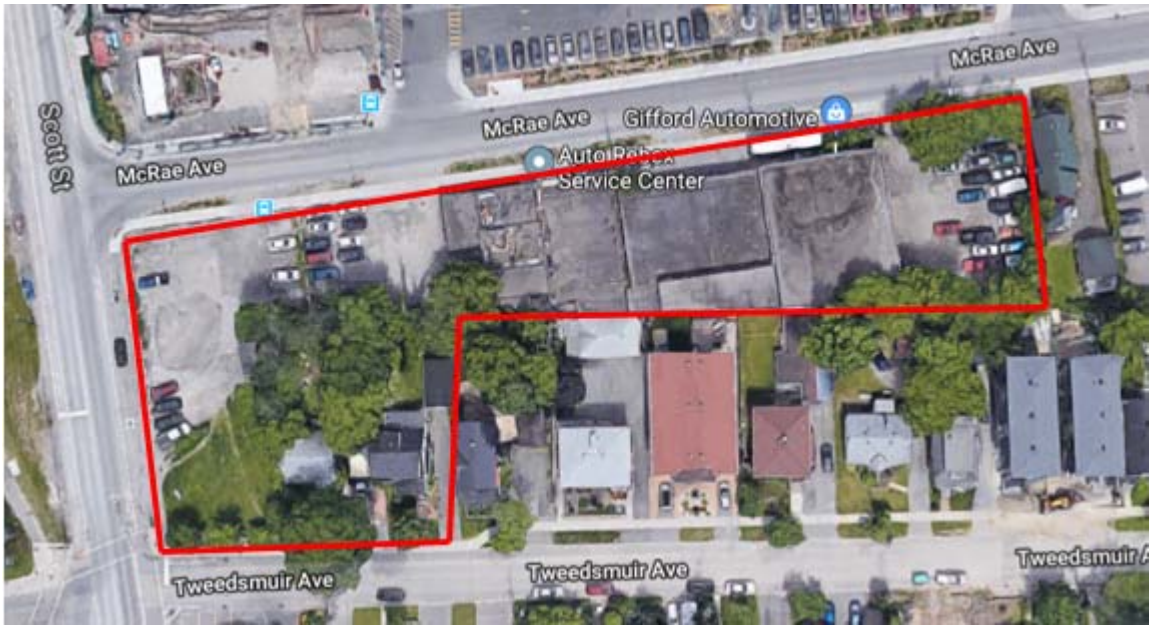

Figure 1: Project Location

The proposed development of the site includes a mixed-use building with underground parking and a park as shown in Figure 2. Refer to the architectural plans for the building layout.

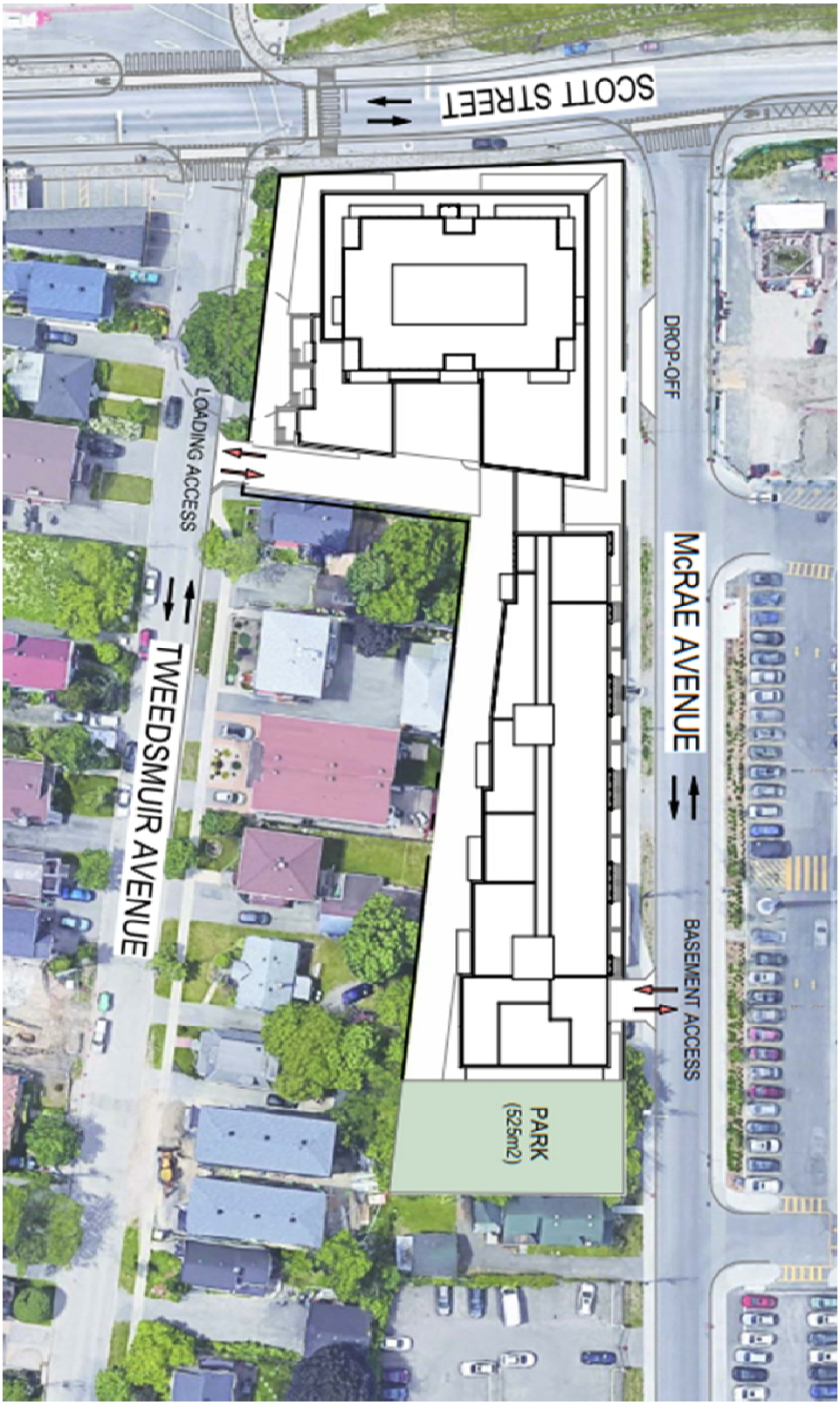

Figure 2: Proposed Development

2.0 STORMWATER MANAGEMENT

2.1 Design Criteria

The following design criteria are proposed as a result of correspondence with the City of Ottawa. The correspondence can be found in Appendix B:

Peak Flow - Control post-development flows from a 100-yr storm to a 5-year storm with a runoff coefficient (C) of 0.5.

Calculated Method - Modified Rational Method using spreadsheet.

Storage Method - Underground storage.

Proposed Drainage - The proposed site storm lateral will discharge to the existing storm sewer on Tweedsmuir Avenue.

Coefficients of Runoff -

	5 Year	**100 Year**
Roof:	C=0.95	C=1.00
Hard Landscape:	C=0.90	C=1.00
Grass:	C=0.20	C=0.25

Rainfall Intensities - City of Ottawa IDF rainfall curve for 100-year storms to generate the intensity formula as follows (See Appendix B for IDF curves):

$$i_{5yr} = \frac{998.071}{(T + 6.053)^{0.814}} \quad \text{– equation (2)}$$

$$i_{100yr} = \frac{1735.688}{(T + 6.014)^{0.820}} \quad \text{– equation (3)}$$

where:
i – Rainfall intensity (mm/hr)
T – Time (min)

2.2 Proposed Approach

To accommodate the volumes calculated below, storage will be provided in a storage tank adjacent to the building.

For the purposes of this report, we have used a modified rational method approach. This method was selected considering the relatively small size of individual drainage areas for the site.

This approach involves using the City of Ottawa IDF charts and equations described above to determine the storage required. For each five-minute interval, an associated flow is calculated using the rational method:

$$Q = \frac{CIA}{3600}$$

where:

Q = Flow (L/s)
C = Runoff Coefficient
I = Rainfall Intensity (mm/hr)
A = Area (m^2)

The flow contributing to storage on-site is the post-development flow minus the allowable discharge rate. The quantity of storage required is calculated by multiplying the flow contributing to storage by the five-minute time interval. The accumulated storage is summed for each five-minute time interval to determine the peak storage required.

2.2.1 Water Quality Requirements

The proposed site development does not include surface parking and the majority of stormwater falling on the site is rooftop and landscaped areas. Roofs and landscaped areas are generally deemed as clean for the purpose of protecting surface water quality and aquatic habitat. The Rideau Valley Conservation Authority (RVCA) has confirmed that stormwater runoff from the site does not require additional quality control measures save and except best management practices. Refer to the attached correspondence with RVCA included in Appendix B.

2.3 Design Calculations

2.3.1 Proposed Site

Drawing C-01 (Appendix A) shows the proposed building and site layout. The total area of the site is 5263 m^2. The proposed site development consists of a mixed-use residential/commercial building with a site area of 4743 m^2 and a park with an area of 520 m^2. The building site and the park will be serviced separately as the park is intended to be developed at a later date. Stormwater management for the park is not considered in this report as it will be designed by others at a later date, however temporary grading is provided to ensure a positive drainage to the street until the park is designed /

developed. Services for the park have been designed to the property line to facilitate this future development.

2.3.2 Adjacent Site Drainage

In addition to the proposed site, a portion of the adjacent properties on Tweedsmuir Avenue currently drain across the subject site. To maintain the drainage for these properties, a roof drain has been provided at the low point within property #315 Tweedsmuir. The flow from the adjacent properties will be conveyed to the storage tank. The allowable discharge will be increased by the 5 year discharge for the contributing area under the existing runoff coefficient for the area. Additional storage in the tank will be allotted for the flow from this area from a storm with a return period between 5 and 100 years.

2.3.3 Site Characteristics

The proposed building site consists of roof and hard landscape areas, with the corresponding City of Ottawa standard runoff coefficients shown in Table 1. The adjacent site runoff characteristics are shown on Table 2.

Table 1: Surface Drainage Areas

Surface Type	Surface Area (m^2)	Runoff Coefficient (5-year)	Runoff Coefficient (100-year)
Hard Landscape	1456	0.90	1.00
Roof	2951	0.95	1.00
Soft Landscape	336	0.20	0.25
Total Surface Area (m^2)	**4743**	**0.88**	**0.95**

Table 2: Adjacent Site Surface Drainage Areas

Surface Type	Surface Area (m^2)	Runoff Coefficient (5-year)	Runoff Coefficient (100-year)
Hard Landscape	106	0.90	1.00
Roof	106	0.95	1.00
Soft Landscape	267	0.20	0.25
Total Surface Area (m^2)	**479**	**0.52**	**0.58**

2.3.4 Allowable Discharge

The allowable peak discharge rate for the building site is equal to the 5-year peak development flow controlled at a time of concentration of 20 minutes and a maximum runoff coefficient of 0.5. Based on this time of concentration, the 5-year rainfall intensity can be calculated as follows:

$$i_{5yr} = \frac{998.071}{(T + 6.053)^{0.814}}$$

$$i_{5yr} = 70.25 \text{ mm/hr}$$

The allowable runoff for the site can then be calculated as follows:

$$Q_{ALLSITE} = \frac{0.5 \text{ x } 70.25 \text{mm/hr x } 4743 \text{m}^2}{3600}$$

$$Q_{ALLSITE} = 46.3 \text{L/s}$$

In addition, the 5 year flow from the adjacent site is added to this amount and is calculated as follows.

$$Q_{ADJ} = \frac{0.52 \text{ x } 70.25 \text{mm/hr x } 479 \text{m}^2}{3600}$$

$$Q_{ADJ} = 4.9 \text{L/s}$$

Thus the total allowable flow is calculated as $Q_{ALLSITE}$ + Q_{ADJ} = 46.3L/s + 4.9L/s = 51.2L/s

This is the total allowable flow from the site, given the requirements of the site.

2.3.5 Storage Requirements

As outlined above, in order to control the total flow from the site to the allowable flow rate of 51.2 L/s, underground storage will be used.

The total surface area of the building site and adjacent properties described above is 4743m^2 + 479m^2, consisting of grassed area/soft landscaping area (336m^2 + 106m^2), hard landscaping/paved surface area (1456m^2 + 267m^2), and roof area (2951m^2 + 106m^2).

Of this surface area, the section between the building and the back of the sidewalk around the building will generally sheet drain freely onto the surrounding streets. This

area is 848m^2, consisting of soft surface (107 m^2) and the remainder hard surface (741 m^2). Flow from this area is as follows:

$$C_{free-100year} = \frac{[1.0(741) + 0.25(107) + 1.0(0)]}{848} = 0.91$$

$$Q_{free} = \frac{0.91 \text{ x } 120.0\text{mm/hr x } 848\text{m}^2}{3600}$$

$$Q_{free} = 25.6 \text{ L/s}$$

The allowable stormwater flow for the remaining controlled surface and roof areas can be calculated by subtracting the proposed free flowing surface from the overall allowable flow.

$$Q_{storage} = Q_{ALL} - Q_{free}$$

$$Q_{storage} = 51.2\text{L/s} - 25.6L/s$$

$$Q_{storage} = 25.6\text{L/s}$$

The remaining surface area, including the adjacent properties is 4374m^2 consisting of grassed area 335m^2, hard landscaping 982m^2 and roof 3057m^2, from which runoff will be contained in a storage tank adjacent to the building.

The overall weighted runoff coefficients for this remaining controlled surface area are calculated using standard City of Ottawa runoff coefficients as:

$$C_{surface-100year} = \frac{[1.0(3057) + 0.25(335) + 1.0(982)]}{4374} = 0.94$$

$$C_{surface-5year} = \frac{[0.95(3057) + 0.20(335) + 0.9(982)]}{4374} = 0.88$$

See Table 3 below for the summary of required storage of surface runoff. Refer to Appendix B for the design calculations. Since the tank will be drained by gravity, the discharge will vary as the tank fills. To account for this, the tank will be sized with the discharge rate of 50% of the allowable discharge. Thus the average tank discharge rate discharge from the tank will be 10.2 L/s.

Table 3: Surface & Roof Discharge and Storage Summary

Area (m^2)	Weighted Runoff Coefficient 5-year	Weighted Runoff Coefficient 100-year	Allowable Discharge (L/s)	Average Discharge (L/s)	5-Year Storage Required (m^3)	100-Year Storage Required (m^3)
4374	0.88	0.94	25.6	12.8	82.73	185.27

2.3.6 Storage Tank Details

The tank must be sized to contain the full 100-year storage volume detailed in Table 3 above. Note that RVA has calculated the required storage volume only and detailed a schematic of the tank sizing and access / overflow, and the ICD model. The structural details of the tank is to be designed by others. The storage tank will be located within the access area accessible from Tweedsmuir Avenue, in the approximate location shown on Drawing C-01. Refer to structural drawings for design and details.

Discharge from the storage tank to the storm lateral must be restricted to the allowable discharge rate by an inlet control device. In the event that the 100-year storm is exceeded, an overflow will be provided through the grated access cover and will flow overland through the uncontrolled surface area towards Tweedsmuir Avenue.

The inlet control device chosen for this project is the Hydrovex 100 VHV-1.

Refer to Appendix B for the storm design sheets and ICD information.

2.3.7 Proposed Storm Sewer Lateral

The site will be serviced with one (1) connection to the City storm sewer network. The proposed sewer lateral connection is a 300mm storm lateral connecting to the 1200 mm diameter storm sewer on Tweedsmuir Avenue.

The location of the service connection is shown on Drawing C-01 in Appendix A.

2.3.8 Foundation Drainage

The foundation drainage system was expected to be designed as per the recommendations in the Geotechnical Investigation report prepared by Pinchin dated April 14, 2020. The recommendations include:

- A perimeter drainage system consisting of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. The clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent).
- An underfloor drainage system beneath the slab, constructed in similar fashion to the foundation drainage.

Refer to the Geotechnical Investigation report for further details, and foundation drainage drawings found in Appendix 5.

The long term flow rate entering the foundation drainage system is expected to be 65,000L/day which is detailed in the Water Taking & Discharge Plans report prepared by Pinchin April 7, 2020.

The quality of water collected by the foundation drainage system is examined in the Phase II ESA report, draft issued November 30, 2020, the Remedial Technology Evaluation – Overview letter, dated November 26, 2020, the Water Taking & Discharge Plans report dated April 7, 2020, and the Remedial Plan for Addressing Groundwater Impacts Memo dated December 4, 2020 (attached in Appendix A) all prepared by Pinchin. Based on the findings of the above reports, it is understood that the groundwater onsite was found to have contaminants exceeding the allowable levels entering either the sanitary or storm sewer systems as specified in the Sewer Use Bylaw. The above named reports propose remedial measures including removing the impacted soil from the site and providing groundwater monitoring to assess residual contaminant levels. Following remedial measures, three scenarios are presented which all require ultimate discharge to the sanitary system.

- Scenario 1: Remedial actions bring groundwater quality below both MECP Table 7 standards and sanitary sewer discharge standards. In this case foundation drainage will be pumped to the sanitary system.
- Scenario 2: Remedial actions fail to bring groundwater quality below MECP Table 7 standards but successfully meet sanitary sewer discharge standards. In this case foundation drainage will be pumped to the sanitary system.
- Scenario 3: Remedial actions fail to bring groundwater quality below both MECP Table 7 standards and also fail to successfully meet sanitary sewer discharge standards. In this case Pinchin has recommended treatment of the groundwater be completed by passing it through activated carbon cylinders being pumped to the sanitary system.

Refer to the Environmental and Water Taking Reports for further information.

Any water treatment systems, if required, will be designed by others as part of the building systems.

As all remedial scenarios presented require discharge to sanitary sewer, the foundation drainage flow rate of 65,000 L/day (0.75L/s) is included in the site flow rates for the sanitary sewer capacity analysis in the site servicing report. It is understood that the water collected by the foundation drainage will be collected in a sump pit(s) and pumped via internal plumbing with the connection to the sanitary system within the building.

3.0 SITE SERVICING

This section of the report provides a summary of the water supply and sanitary servicing to the site. The layout of site servicing including water, sanitary and storm services is shown in Appendix A.

3.1 Design Criteria

Based on the size and use of the proposed building, the water demand was calculated using the City of Ottawa Design Guidelines for Water Distribution (July 2010). The wastewater demand was calculated using on the City of Ottawa Sewer Design Guidelines (October 2012) and accompanying technical bulletins.

3.2 Water Service

The water demand for the proposed development area is calculated using the City of Ottawa Design Guidelines for Water Distribution (July 2010). The existing and proposed development consists of residential and commercial areas allocated as per the following table:

Table 4: Site Statistics

Type	Units/Area	Persons Per Unit	Population
EXISTING SITE			
Single Family	2 Units	3.4	6.8
Commercial	0.12 ha	-	-
PROPOSED SITE			
Townhouse	11 Units	2.7	29.7
Bachelor	58 Units	1.4	81.2
1 Bedroom	186 Units	1.4	260.4
2 Bedroom	82 Units	2.1	172.2
3 Bedroom	10 Units	3.1	31
Commercial	0.09 ha	-	-
Total Proposed	**0.09ha (commercial)**	**-**	**574.5**

For residential development, an average water consumption rate of 350 L/c/day is used. The maximum daily flow is calculated as:

$$\text{Residential Max Daily Flow (W)} = 2.5 \text{ x Average Daily Flow}$$

The maximum hourly flow is calculated as:

$$\text{Residential Max Hourly Flow (W)} = 2.2 \text{ x Max Daily Flow}$$

For commercial development, an average water consumption rate of 25000 L/gross ha/d, as per Section 4.2.8 of the design guidelines. The maximum daily flow for commercial areas is calculated as:

$$\text{Commercial Max Daily Flow (W)} = 1.5 \text{ x Average Daily Flow}$$

The maximum hourly flow for commercial areas is calculated as:

$$\text{Commercial Max Hourly Flow (W)} = 1.8 \text{ x Max Daily Flow}$$

Water flows for the proposed building calculated using the method above are summarized in Table 5.

Table 5: Water Flows

Type	Population or Area	Average Daily Flow (L/s)	Maximum Daily Flow (L/s)	Maximum Hourly Flow (L/s)
EXISTING SITE				
Residential	7 persons	0.03	0.07	0.15
Commercial	0.12 ha	0.03	0.05	0.09
	Total	0.06	0.12	0.24
PROPOSED SITE				
Residential	575 persons	2.33	5.82	12.80
Commercial	0.09 ha	0.03	0.04	0.07
	Total	2.35	5.86	12.87

Since the basic water demand is greater than 50 m^3/day (0.6 L/s), the proposed site will be serviced with two (2) connections from city watermains to avoid the creation of a vulnerable service area.

3.2.1 Proposed Water Service Connections

The site is located in Zone 1W of the City of Ottawa's water distribution system.

The proposed water service connections are:

- 150 mm water service entering at the northeast corner of the building (on the north side) and connected to the 203mm diameter watermain on McRae Avenue to the north of the existing valve.
- 150mm water service entering at the northeast corner of the building (on the east side) and connected to the 203mm diameter watermain on McRae Avenue to the south of the existing valve.

The locations of the service connections are shown on Drawing C-01 in Appendix A.

3.2.2 Fire Flow

The fire flow required for each building was calculated using the Fire Underwriters Survey Method (1999), as follows:

$$F = 220C\sqrt{A}$$

where:

F = the required fire flow in litres per minute.
C = coefficient related to the type of construction
A = floor area in square metres

The building was considered to be of ISO Construction class 5 (modified fire resistive), which correspondences to a construction type coefficient (C) of 0.6.

Per the FUS method for fire-resistive construction type, the floor area was calculated as the two largest adjoining floors (levels 2 & 3) plus 50 percent of the floors immediately above them up to eight floors (levels 4 to 11).

The maximum fire flow required for the building as calculated per the method above is 9,000 L/min (150 L/s). Refer to the calculations included in Appendix C for more detail.

The following boundary conditions were provided by the City of Ottawa:

- Minimum HGL = 108.5m
- Maximum HGL = 115.5m
- MaxDay + FireFlow (150 L/s) = 103.0m (McRae Ave connection)

There are four hydrants adjacent to the site: two on McRae Avenue, one on Scott Street, and one on Tweedsmuir Avenue, as indicated on Drawing C-01. Hydrant testing in the

area is recommended to confirm the available flow and pressure to confirm the fire protection supply.

3.3 Sanitary Service

Based on the City of Ottawa Sewer Design Guidelines (October 2012) and accompanying technical bulletins, an average wastewater rate of 280 L/c/day is used for residential buildings. The maximum daily flow rate is calculated as follows:

$$\text{Residential Max Daily Flow (Sanitary)} = \text{Residential Average Daily Flow (Sanitary)} * \text{Peak Factor}$$

where: $$\text{Peak Factor} = 1 + \left(\frac{14}{4+\left(\frac{\text{Population}}{1000}\right)^{0.5}}\right) * \text{K}$$

In addition, according to the design guidelines an average wastewater rate of 28,000 L/gross ha/d is used for the commercial areas. According to the guidelines, since the commercial area on site less than 20% of the total area, the maximum daily flow is:

$$\text{Commercial Max Daily Flow (Sanitary)} = 1.0 \text{ x Average Daily Flow(Sanitary)}$$

Additionally, extraneous flows can be calculated as follows:

$$\text{Q}_{\text{extraneous}} = 0.33\text{L/s} * Area$$

Table 6 below presents the wastewater flows for the proposed building calculated using the method above. Sanitary flows are provided for information only. Capacity of the sewer system has not been verified.

Table 6: Wastewater Flows

Type	Units / Area	Average Daily Flow (L/s)	Maximum Daily Flow (L/s)
Residential	347 units	1.86	6.24
Commercial	0.09 ha	0.03	0.03
Extraneous	2.086 ha	0.69	0.69
Total		**2.58**	**6.96**

3.3.1 Foundation Drainage

The foundation drainage system has been designed as per the recommendations in the Geotechnical Investigation report prepared by Pinchin. The recommendations include:

- A perimeter drainage system consisting of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. The clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent).
- An underfloor drainage system beneath the slab, constructed in similar fashion to the foundation drainage.

Refer to the Geotechnical Report which can be found in Appendix 4 for further details. Refer to foundation drainage design drawings which can be found in Appendix 5.

The long term flow rate entering the foundation drainage system is expected to be 65,000L/day which is detailed in the Water Taking & Discharge Plans report prepared by Pinchin and confirmed in an email which can both be found in Appendix 4.

The quality of water collected by the foundation drainage system is examined in the Phase II ESA report, the Remedial Technology Evaluation – Overview letter, the Water Taking & Discharge Plans report, and the Remedial Plan for Addressing Groundwater Impacts at 320 McRae Avenue, 1976 Scott Street, 311 and 315 Tweedsmuir Avenue letter all prepared by Pinchin which can be found in Appendix 4. Based on the findings of the above reports, it is understood that the groundwater onsite was found to have contaminants exceeding the allowable levels entering either the sanitary or storm sewer systems as specified in the Sewer Use Bylaw. The above named reports, propose remedial measures including removing the impacted soil from the site and providing groundwater monitoring to assess residual contaminant levels. Following remedial measures, three scenarios are presented which all require ultimate discharge to the sanitary system.

- Scenario 1: Remedial actions bring groundwater quality below both MECP Table 7 standards and sanitary sewer discharge standards. In this case foundation drainage will be pumped to the sanitary system.
- Scenario 2: Remedial actions fail to bring groundwater quality below MECP Table 7 standards but successfully meet sanitary sewer discharge standards. In this case foundation drainage will be pumped to the sanitary system.
- Scenario 3: Remedial actions fail to bring groundwater quality below both MECP Table 7 standards and also fail to successfully meet sanitary sewer discharge standards. In this case Pinchin has recommended treatment of the groundwater be completed by passing it through activated carbon cylinders being pumped to the sanitary system.

Refer to the Environmental and Water Taking Reports for further information.

Any water treatment systems will be designed by others as part of the building systems.

As all remedial scenarios presented require discharge to sanitary sewer, the foundation drainage flow rate of 65,000 L/day (0.75L/s) will be included in the site flow rates in the sanitary sewer capacity analysis. It is understood that the water collected by the foundation drainage will be collected in a sump it and pumped via internal plumbing with the connection to the sanitary system within the building.

3.3.2 Proposed Sanitary Sewer Lateral

The proposed development area will be serviced with one (1) connection to the City sewer network. The proposed sewer lateral connection is a 250mm sanitary lateral connecting to the 250 mm diameter sanitary sewer on McRae Avenue. A new manhole will be installed to connect to the main sewer. The location of the service connection is shown on Drawing C-01 in Appendix A.

3.3.3 Existing Conditions

There are 2 single family homes and 1170 m^2 of commercial space (automotive service centers) on the current site. The wastewater flows for the existing site are presented below in Table 7.

Table 7: Existing Site Wastewater Flows

Type	Units / Area	Average Daily Flow (L/s)	Maximum Daily Flow (L/s)
Residential	2 units	0.02	0.08
Commercial	0.117 ha	0.04	0.04
Extraneous	2.086 ha	0.69	0.69
Total		**0.75**	**0.81**

The existing buildings on the site will be demolished as part of the proposed development. The existing service connections will be removed from the site during construction and capped at the property line.

3.3.4 Sewer Capacity Analysis

The existing sanitary sewers downstream of the site were analyzed to determine whether sufficient capacity exists to convey the additional flows from the proposed development. The flows from site considered are the Maximum Daily flow in the

developed site (6.96L/s) plus the foundation drainage flows (0.75L/s), which then have the pre-development flows subtracted (0.81L/s) for a total flow from site of 6.90L/s. The capacities and pre/post development flows of the affected sewers are shown in the table below and calculations can be found in Appendix C:

Table 8: Existing Site Wastewater Flows

Sewer Location / Description	Slope	Capacity (L/S)	Pre-Development Flow (L/s)	Post-Development Flow (L/s)
250mm Diameter Sewer at north end of McRae Avenue	0.79%	54.8	5.45	12.35
300mm Diameter on Scott Street Between McRae and Tweedsmuir	0.43%	66.3	17.47	24.37
375mm Diameter Crossing Scott Street Near Tweedsmuir	1.00%	182.9	58.65	65.55
375mm Diameter adjacent Scott Street Near Tweedsmuir Discharging to Trunk	1.00%	182.9	58.65	65.55

Per the above table, there is sufficient capacity in all the downstream sewers to the trunk to convey the additional flows from the proposed site.

3.4 City Park Land

The area to the south of the site has been set aside as a future City owned park. It is our understanding that this area will be developed by others in the future. As such, the grading shown is temporary and servicing will be done in the future if required. Services to the park will be brought to the property line and capped as follows:

The storm service is a 300mm diameter service installed at 1% slope connecting to the 900mm diameter storm sewer on McRae Avenue.

The sanitary service is a 200mm diameter service installed at 2% slope connecting to the 250mm diameter storm sewer on McRae Avenue.

The water service is a 50mm diameter PEX service installed at a depth of 2.4m connecting to the existing 150mm diameter watermain on McRae Avenue. The water service will terminate at the edge of the park in a new park water meter chamber per W31.1.

4.0 EROSION AND SEDIMENT CONTROL

Erosion and sediment control measures (in accordance with the requirements of OPSS 805 – November 2018 for temporary measures) consisting of both permanent and temporary measures shall be implemented prior to the commencement of construction activities to ensure that sediment is contained within the site. Permanent erosion control measures shall ensure that potential long-term and localized erosion problems are dealt with prior to their occurrence.

4.1 Temporary Sediment Control Measures

Filter fabric shall be installed under the frame of all proposed and existing catchbasins and storm manholes immediately adjacent to any disturbed areas prior to construction to prevent sediment from entering into the sewer system. The filter fabric shall remain in-place for the duration of construction activities and shall not be removed until such time as the landscaping has been established and upon authorization by the Engineer. Light duty sediment fencing shall also be placed around the perimeter of the site for the duration of the construction.

Refer to Drawing C-02 for specific erosion and sediment control measures to be installed and monitored during construction.

5.0 CONCLUSION

The design of the stormwater management system serves to control the 100-year peak post-development flows to that of the 5-year peak flow at a runoff coefficient 0.5 as recommended by the City of Ottawa. On-site storage is proposed below the surface within storage tanks on the west side of the building during the 5-year and 100-year storm events, designed by others. Discharge from the storage tanks into City's sewer system will be via gravity with an inlet control device. It will be the owners' responsibility to maintain the stormwater storage tank, and inlet control device in good working condition.

Given that the runoff coefficient for the site is being lowered to 0.5 which is lower than the runoff coefficient under existing conditions, the existing storm sewers on McRae Avenue and Scott Street are assumed to have adequate capacity to accommodate stormwater flow from the proposed buildings.

Fire flow requirements were calculated; however, capacity in the system must be confirmed with the City, based on boundary flow conditions.

We trust this Site Servicing and Stormwater Management report complies with the City of Ottawa requirements and we look forward to receiving your approval.

R.V. ANDERSON ASSOCIATES LIMITED

Trevor Kealey
2021.05.03
09:23:48 -
04'00'

Prepared by:
Nathaniel Rodgers, P.Eng.

Reviewed by:
Trevor Kealey, P.Eng.

APPENDIX 1

Drawings

[redacted]

NOTES:

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ARCHITECTURAL DRAWINGS BY NEUF ARCHITECTES SENCRL. REFER TO ARCHITECTURAL PLANS FOR DIMENSIONS.
2. ALL WATERMAINS TO BE INSULATED IF LESS THAN 2.4 METERS COVER AS PER CITY OF OTTAWA STANDARD DETAIL W22. AT ANY PROXIMITY OF SEWER MANHOLES, INSULATE WATERMAIN AS PER CITY DETAIL W23.
3. SEWERS ARE TO MAINTAIN 500mm BARRELL TO BARRELL CLEARANCE ABOVE AND 250mm BARRELL TO BARRELL CLEARANCE BELOW WATERMAINS AT ALL CROSSINGS (AS PER CITY DWGS W25 AND W25.2). IF 22° BENDS ARE USED ON THE WATER MAIN, THEY MUST BE ONE METER AWAY FROM THE SEWER.
4. THRUST BLOCKS TO BE AS PER CITY OF OTTAWA STANDARD DRAWINGS W25.3 AND W25.4. RESTRAINING AND RETAINING RINGS TO BE INSTALLED IN ACCORDANCE WITH CITY STANDARD DETAILS W25.5 AND W25.6.
5. TEMPORARY SUPPORT OF EXISTING UNDERGROUND UTILITIES IN ACCORDANCE WITH CITY STANDARD DETAIL W28.
6. WATERMAIN TRENCH AND BEDDING TO BE INSTALLED AS PER CITY DETAIL W17.
7. TAPPING VALVE SYSTEM CONNECTION TO CITY WATERMAIN BY CITY FORCES; EXCAVATION, BACKFILLING AND REINSTATEMENT BY CONTRACTOR.
8. CONTRACTOR SHALL BE RESPONSIBLE FOR LOCATION OF ALL EXISTING U/G AND OVERHEAD UTILITIES. VARIOUS UTILITIES CONCERNED TO BE GIVEN REQUIRED ADVANCE NOTICE PRIOR TO ANY DIGGING FOR STAKE OUT. THE OWNER AND CONSULTANT ASSUMES NO RESPONSIBILITY FOR THE ACCURACY OF THE LOCATION OF EXISTING UTILITIES AS INDICATED ON THIS DRAWING.
9. UTILITY INFORMATION WAS VERIFIED IN THE FIELD WHERE POSSIBLE. INDIVIDUAL COMPANIES SHOULD BE CONTACTED BY THE CONTRACTOR PRIOR TO COMMENCEMENT FOR CONFIRMATION OF EXISTENCE AND LOCATION OF UTILITIES.
10. WATER SERVICE, STORM SEWERS AND APPURTENANCE TO COMPLY WITH THE REQUIREMENTS OF THE LATEST REVISION OF THE ONTARIO PLUMBING CODE AND APPLICABLE CITY OF OTTAWA ENGINEERING STANDARDS.
11. ALL SANITARY SEWERS TO BE INSULATED IF LESS THAN 2.0 METERS COVER. ALL STORM SEWERS TO BE INSULATED IF LESS THAN 2.0 METERS COVER. INSULATE AS PER CITY OF OTTAWA STANDARD DETAIL W22. ALL BUILDING CONNECTIONS TO HAVE SUFFICIENT COVER OR INSULATION IS REQUIRED.
12. CONTRACTOR SHALL CONTACT THE CONSULTANT, R.V. ANDERSON PRIOR TO BACKFILLING OF THE WATER SERVICE CONNECTIONS FOR THE PROPOSED LINES AND TIE-INS TO EXISTING LINES FOR AS-BUILT LOCATION RECORDS AND INSPECTION.
13. ANY ASPHALT CUT SHALL BE SAW CUT ON BOTH SIDES OF THE TRENCH FOR THE ENTIRE LENGTH OF THE EXCAVATION FOR PIPE INSTALLATIONS. REINSTATEMENT OF THE ROADS SHALL MATCH EXISTING OR MEET CITY STANDARD R10.
14. ANY CONCRETE CUT SHALL BE REMOVED AT EXPANSION JOINTS. IF NO JOINTS EXIST, THE CONCRETE SHALL BE SAW CUT ON BOTH SIDES OF THE TRENCH FOR THE ENTIRE LENGTH OF THE EXCAVATION FOR PIPE INSTALLATIONS. REINSTATEMENT SHALL MATCH EXISTING OR MEET CITY REQUIREMENTS.
15. PIPE BEDDING SHALL BE GRANULAR "A" AS PER CITY DETAIL S6, AND SHALL BE COMPACTED TO 95% SPD AND APPROVED SELECT NATIVE BACK FILL COMPACTED TO 95% SPD.
16. DRAWINGS TO BE READ IN CONJUNCTION WITH CONTRACT SPECIFICATIONS.
17. GRANULAR LAYERS BENEATH NEW ASPHALT SURFACES ON PROPERTY SHALL BE PLACED AT A THICKNESS NOT EXCEEDING 300mm. THE GRANULAR 'A' AND GRANULAR 'B' TYPE II IS TO BE COMPACTED TO A MINIMUM OF 100% SPMDD USING SUITABLE VIBRATORY EQUIPMENT.
18. THE APPROVAL OF THIS PLAN DOES NOT EXEMPT THE OWNER'S BONDED CONTRACTOR FROM THE REQUIREMENTS TO OBTAIN THE VARIOUS PERMITS/APPROVALS NORMALLY REQUIRED TO COMPLETE A CONSTRUCTION PROJECT, SUCH AS, BUT NOT LIMITED TO THE FOLLOWING: ROAD CUT PERMITS, SEWER PERMITS, APPROACH APPROVAL PERMITS, RELOCATION OF SERVICES, COMMITTEE OF ADJUSTMENT, ENCROACHMENT AGREEMENTS, WATER PERMIT, ETC
19. THE OWNER SHALL REQUIRE THAT THE SITE SERVICING CONTRACTOR PERFORM FIELD TESTS FOR QUALITY CONTROL OF ALL SANITARY SEWERS. SPECIFICALLY, THE LEAKAGE TESTING SHALL BE COMPLETED IN ACCORDANCE WITH OPSS 410.07.01.16 AND 407.07.26. THE FIELD TESTS SHALL BE PERFORMED IN THE PRESENCE OF A CERTIFIED PROFESSIONAL ENGINEER WHO SHALL SUBMIT A CERTIFIED COPY OF THE TEST RESULTS.
20. REFER TO LANDSCAPE DRAWINGS FOR DETAILS ON LANDSCAPING TREATMENTS AND PLANTINGS.
21. SEWERS TO BE CONSTRUCTED AS PER CITY OF OTTAWA SPECIFICATIONS - SPECIAL PROVISION F-4100, ALL SEWER STRUCTURES AS PER F-4070, ALL WATER MAINS AS PER F-4411 AND ALL ASSOCIATED SPECIFICATIONS. IRON ADJUSTMENTS PER F-4080.
22. EROSION AND SEDIMENT CONTROL MEASURES (IN ACCORDANCE WITH THE REQUIREMENTS OF OPSS 805 - NOVEMBER 2018 FOR TEMPORARY MEASURES) CONSISTING OF BOTH PERMANENT AND TEMPORARY MEASURES SHALL BE IMPLEMENTED PRIOR TO THE COMMENCEMENT OF CONSTRUCTION ACTIVITIES TO ENSURE THAT SEDIMENT IS CONTAINED WITHIN THE SITE. PERMANENT EROSION CONTROL MEASURES SHALL ENSURE THAT POTENTIAL LONG-TERM AND LOCALIZED EROSION PROBLEMS ARE DEALT WITH PRIOR TO THEIR OCCURRENCE. FILTER FABRIC SHALL BE INSTALLED UNDER THE FRAME OF ALL PROPOSED AND EXISTING CATCHBASINS AND STORM MANHOLES IMMEDIATELY ADJACENT TO ANY DISTURBED AREAS PRIOR TO CONSTRUCTION TO PREVENT SEDIMENT FROM ENTERING INTO THE STORM SEWER SYSTEM. THE FILTER FABRIC SHALL REMAIN IN-PLACE FOR THE DURATION OF CONSTRUCTION ACTIVITIES AND SHALL NOT BE REMOVED UNTIL SUCH TIME AS THE LANDSCAPING HAS BEEN ESTABLISHED AND UPON AUTHORIZATION BY THE ENGINEER. LIGHT DUTY SEDIMENT FENCING SHALL ALSO BE PLACED AROUND THE PERIMETER OF THE SITE FOR THE DURATION OF THE CONSTRUCTION.
23. THE CONTRACTOR ACKNOWLEDGES THAT FAILURE TO IMPLEMENT APPROPRIATE EROSION AND SEDIMENT CONTROL MEASURES MAY BE SUBJECT TO PENALTIES IMPOSED BY ANY APPLICABLE REGULATORY AGENCY.
24. CONNECTION OF THE WATER SERVICES TO THE CITY WATERMAIN BY CITY FORCES; EXCAVATION, BACKFILLING AND REINSTATEMENT BY CONTRACTOR.
25. REFER TO THE STORM WATER MANAGEMENT & SITE SERVICING REPORTS FROM RV ANDERSON DATED FEBRUARY 19, 2020 FOR FURTHER DETAILS.
26. REFER TO GEOTECHNICAL REPORT BY PINCHIN LTD. DATED APRIL 29, 2020 FOR SOILS INFO.
27. SIDEWALK DEPRESSIONS PER CITY DETAIL SC6.
28. REFER TO MECHANICAL DRAWINGS FOR INTERNAL PLUMBING INCLUDING WATER METER, BACKFLOW PREVENTION, INTERNAL PIPING, FOUNDATION DRAINAGE CONNECTION ETC.

CONTAMINATED WATER / FOUNDATION DRAIN INFORMATION:

THE QUALITY OF WATER COLLECTED BY THE FOUNDATION DRAINAGE SYSTEM IS EXAMINED IN THE PHASE II ESA REPORT, DRAFT ISSUED NOVEMBER 30, 2020, THE REMEDIAL TECHNOLOGY EVALUATION - OVERVIEW LETTER, DATED NOVEMBER 26, 2020, THE WATER TAKING & DISCHARGE PLANS REPORT DATED APRIL 7, 2020, AND THE REMEDIAL PLAN FOR ADDRESSING GROUNDWATER IMPACTS MEMO DATED DECEMBER 4, 2020 (ATTACHED IN APPENDIX A) ALL PREPARED BY PINCHIN. BASED ON THE FINDINGS OF THE ABOVE REPORTS, IT IS UNDERSTOOD THAT THE GROUNDWATER ONSITE WAS FOUND TO HAVE CONTAMINANTS EXCEEDING THE ALLOWABLE LEVELS ENTERING EITHER THE SANITARY OR STORM SEWER SYSTEMS AS SPECIFIED IN THE SEWER USE BYLAW. THE ABOVE NAMED REPORTS PROPOSE REMEDIAL MEASURES INCLUDING REMOVING THE IMPACTED SOIL FROM THE SITE AND PROVIDING GROUNDWATER MONITORING TO ASSESS RESIDUAL CONTAMINANT LEVELS. FOLLOWING REMEDIAL MEASURES, THREE SCENARIOS ARE PRESENTED WHICH ALL REQUIRE ULTIMATE DISCHARGE TO THE SANITARY SYSTEM.

- SCENARIO 1: REMEDIAL ACTIONS BRING GROUNDWATER QUALITY BELOW BOTH MECP TABLE 7 STANDARDS AND SANITARY SEWER DISCHARGE STANDARDS. IN THIS CASE FOUNDATION DRAINAGE WILL BE PUMPED TO THE SANITARY SYSTEM.
- SCENARIO 2: REMEDIAL ACTIONS FAIL TO BRING GROUNDWATER QUALITY BELOW MECP TABLE 7 STANDARDS BUT SUCCESSFULLY MEET SANITARY SEWER DISCHARGE STANDARDS. IN THIS CASE FOUNDATION DRAINAGE WILL BE PUMPED TO THE SANITARY SYSTEM.
- SCENARIO 3: REMEDIAL ACTIONS FAIL TO BRING GROUNDWATER QUALITY BELOW BOTH MECP TABLE 7 STANDARDS AND ALSO FAIL TO SUCCESSFULLY MEET SANITARY SEWER DISCHARGE STANDARDS. IN THIS CASE PINCHIN HAS RECOMMENDED TREATMENT OF THE GROUNDWATER BE COMPLETED BY PASSING IT THROUGH ACTIVATED CARBON CYLINDERS BEING PUMPED TO THE SANITARY SYSTEM.

REFER TO THE ENVIRONMENTAL AND WATER TAKING REPORTS FOR FURTHER INFORMATION.

ANY WATER TREATMENT SYSTEMS, IF REQUIRED, WILL BE DESIGNED BY OTHERS AS PART OF THE BUILDING SYSTEMS.

AS ALL REMEDIAL SCENARIOS PRESENTED REQUIRE DISCHARGE TO SANITARY SEWER, THE FOUNDATION DRAINAGE FLOW RATE OF 65,000 L/DAY (0.75L/S) IS INCLUDED IN THE SITE FLOW RATES FOR THE SANITARY SEWER CAPACITY ANALYSIS IN THE SITE SERVICING REPORT. IT IS UNDERSTOOD THAT THE WATER COLLECTED BY THE FOUNDATION DRAINAGE WILL BE COLLECTED IN A SUMP PIT(S) AND PUMPED VIA INTERNAL PLUMBING WITH THE CONNECTION TO THE SANITARY SYSTEM WITHIN THE BUILDING.

STORM INVERT SCHEDULE

	STRUCTURE	GROUND	INVERT	COMMENTS
	RD#1	63.76	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#2	64.17	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#3	64.24	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#4	64.29	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#5	64.30	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#6	63.35	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#7	64.50	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#8	64.69	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#9	63.85	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#10	63.85	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#11	63.85	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	RD#12	63.85	SEE MECHANICAL	ROOF DRAIN: SEE MECHANICAL
	CB#1	62.92	EXISTING	REPLACE FRAME AND COVER WITH SURFACE STYLE PER CITY DETAIL S19.1.
	CB#2	63.17	EXISTING	REPLACE FRAME AND COVER WITH SURFACE STYLE PER CITY DETAIL S19.1.
250mm PVC STORM SEWER (PARK SERVICE)	CAP AT PROPERTY LINE	63.91	62.41	CAP SERVICE AT PROPERTY LINE FOR FUTURE CONNECTION
	WATERMAIN CROSSING	63.82	62.35	STM LATERAL CROSSES OVER 200mm WATERMAIN. TOP OF WM = 61.18m (CLEARANCE = 1170mm)
	SANITARY CROSSING	63.85	62.33	STM LATERAL CROSSES OVER 250mm SANITARY SEWER. SEWER INVERT = 61.28m (CLEARANCE = 800mm)
	CONNECTION TO CITY SEWER	63.87	62.32	CONNECT TO 900mm STORM SEWER AS PER CITY DWG S11. CITY SEWER INVERT 61.88m.
300mm PVC STORM SEWER (BUILDING SERVICE)	STORM CISTERN CONNECTION	63.90	59.58	CONNECTION TO BUILDING SERVICES: SEE MECHANICAL
	MONITORING MANHOLE	63.44	59.57	NEW 1200mm DIAMETER MANHOLE PER OPSD 701.010. FRAME/COVER PER S24.1 & S25.
	WATERMAIN CROSSING	63.23	59.51	STM LATERAL CROSSES UNDER 150mm WATERMAIN. BOTTOM OF WM = 60.75m (CLEARANCE = 960mm)
	SANITARY CROSSING	63.16	59.48	STM LATERAL CROSSES UNDER 375mm SANITARY SEWER. SEWER INVERT = 60.39m (CLEARANCE = 610mm)
	CONNECTION TO CITY SEWER	63.12	59.46	CONNECT TO 1200mm STORM SEWER AS PER CITY DWG S11. CITY SEWER INVERT 58.76m.

PRIVATE WATERMAIN TABLE

	STATION	DESCRIPTION	TOP OF PIPE ELEVATION	GROUND ELEVATION	COMMENTS
150mm PVC DR18 WATER SERVICE (NORTH SIDE)	1+00.0	BUILDING/PARKING GARAGE CONNECTION	SEE MECHANICAL	63.99	CONNECTION TO BUILDING SERVICES: SEE MECHANICAL
	1+02.3	VALVE & VALVE BOX	61.48	63.88	VALVE AND VALVE BOX PER W24
	1+04.7	STORM CROSSING	60.67	63.84	WATERMAI?N CROSSES UNDER 1200mm STORM SEWER PER W25. STORM INVERT= 61.32m (BARREL TO BARREL CLEARANCE = 500mm)
	1+09.9	CONNECTION TO 203mmØ WM.	61.01	63.66	CONNECT TO CITY WATERMAIN AS PER W33
150mm PVC DR18 WATER SERVICE (EAST SIDE)	2+00.0	BUILDING/PARKING GARAGE CONNECTION	SEE MECHANICAL	63.99	CONNECTION TO BUILDING SERVICES: SEE MECHANICAL
	2+02.6	VALVE & VALVE BOX	61.39	63.89	VALVE AND VALVE BOX PER W24
	2+04.9	CONNECTION TO 203mmØ WM.	60.57	63.43	CONNECT TO CITY WATERMAIN AS PER W33
50mm PEX WATER SERVICE (PARK)	3+00.0	PARK CONNECTION	SEE MECHANICAL / LANDSCAPE	64.00	NEW PARK WATER METER CHAMBER PER W31.1
	3+00.8	VALVE & VALVE BOX	61.63	64.03	VALVE AND VALVE BOX PER W24
	3+06.1	CONNECTION TO 203mmØ WM.	61.18	63.82	CONNECT TO CITY WATERMAIN AS PER W33

SANITARY INVERT SCHEDULE

	STRUCTURE	GROUND	INVERT	COMMENTS
250mm PVC	BUILDING/PARKING GARAGE CONNECTION	63.48	61.19	CONNECTION TO BUILDING SERVICES: SEE MECHANICAL FOR MONITORING PORT IN PARKING GARAGE
	MONITORING PORT	63.45	61.16	NEW 250mm DIAMETER MANHOLE PER CITY DETAIL S18.1. STEEL COVER REQUIRED.
	WATERMAIN CROSSING	62.82	60.99	SAN LATERAL CROSSES OVER 203mm WATERMAIN. TOP OF WM = 60.42m (CLEARANCE = 570mm)
	CONNECTION TO MAIN SEWER	62.88	60.92	CONNECT TO 250mm SANITARY SEWER AT NEW 1200mm DIAMETER MANHOLE. SEWER INVERT 61.00. NEW MANHOLE PER OPSD 701.010. FRAME AND COVER PER S24 & S25.
150mm PVC	CAP AT PROPERTY LINE	63.91	61.50	CAP NEW SERVICE AT PROPERTY LINE FOR FUTURE CONNECTION.
	WATERMAIN CROSSING	63.72	61.40	SAN LATERAL CROSSES OVER 203mm WATERMAIN. TOP OF WM = 61.15m (CLEARANCE = 250mm). NOTE SEWER SERVICE PIPE SECTION TO BE PLACED TO AVOID JOINT NEAR WATER CROSSING.
	CONNECTION TO MAIN SEWER	63.76	61.36	CONNECT TO 250mm SANITARY SEWER PER S11.1. SEWER INVERT 61.26.

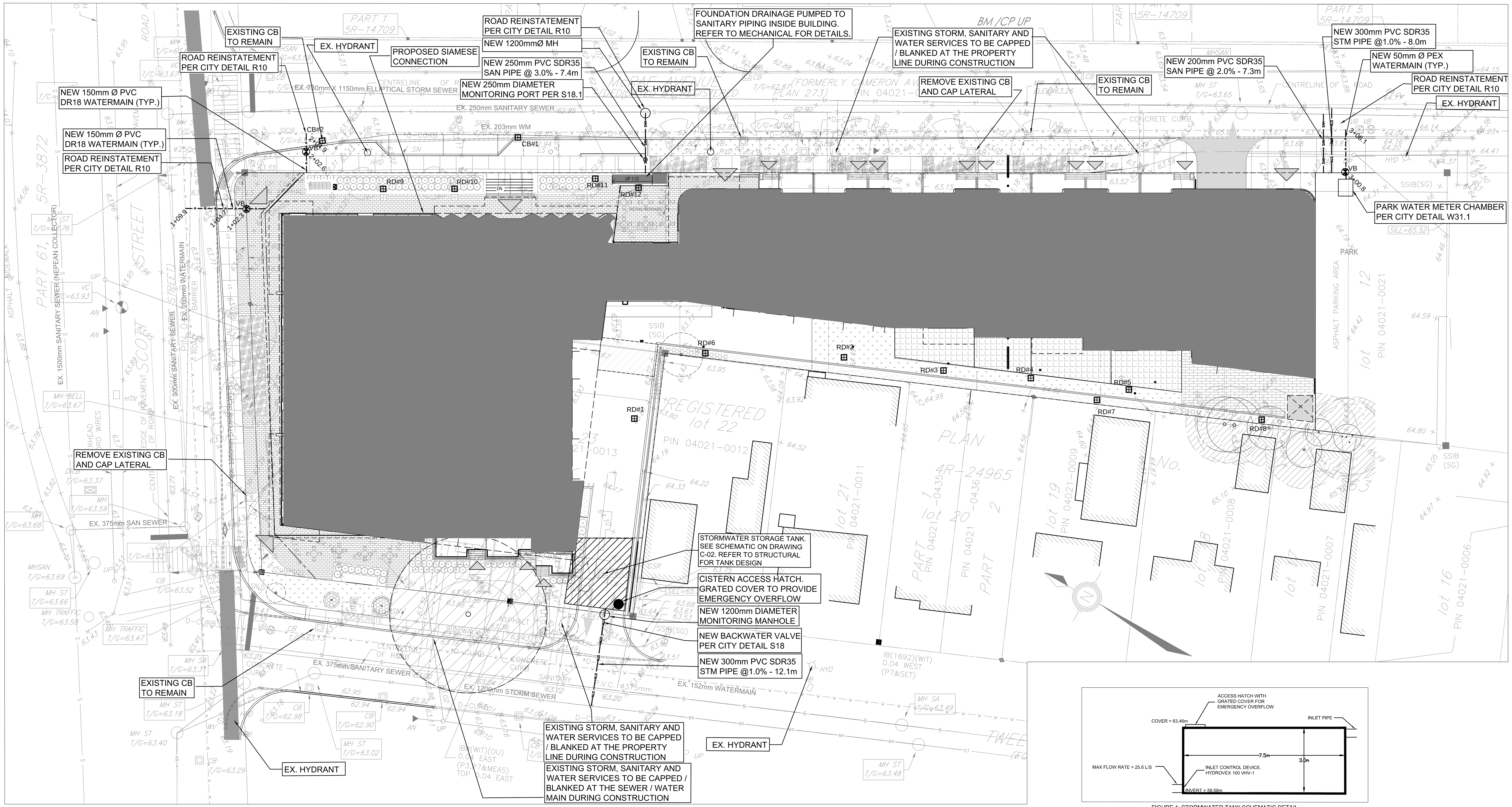

FIGURE 1: STORMWATER TANK SCHEMATIC DETAIL

NOT FOR CONSTRUCTION - SUBJECT TO CHANGE PENDING OUTSTANDING APPROVALS

ARCHITECTURE DE PAYSAGE Landscape Architect
James B. Lennox & Associates Inc.
3332 Carling Ave.
Ottawa, Ontario K2H 5A8
T 613 722 5168

CIVIL Civil
R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1844

ARPENTEUR Surveyor
Stantec Geomatics Ltd.
400 - 1331 Clyde Avenue
Ottawa, ON K2C 3G4
T 613 722 4420

URBANISTE Urban Planner
Fotenn Planning + Design
396 Cooper Street - Suite 300
Ottawa, ON, K2P 2H7
T 613 730 5709

ARCHITECTES Architect
NEUF architect(e)s SENCRL
630, boul. René-Lévesque O. 32e étages, Montréal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

SCEAU / Seal

LICENSED PROFESSIONAL ENGINEER
APR 30, 2021
N.S.R. RODGERS
100163412
PROVINCE OF ONTARIO

R.V. Anderson Associates Limited
engineering • environment • infrastructure

CLIENT Client

GWL REALTY ADVISORS

OUVRAGE Project

320 MCRAE

EMPLACEMENT Location: **320 MCRAE**
NO PROJET No.: **194453**

NO	REVISION	DATE
1	QA SUBMISSION	02/24/2020
2	FOR COORDINATION	02/26/2020
3	SITE PLAN SUBMISSION	03/20/2020
4	SITE PLAN RESUBMISSION	09/23/2020
5	EXCAVATION PERMIT	09/25/2020
6	FOR COORDINATION	12/22/2020
7	BUILDING PERMIT	01/04/2021
8	SITE PLAN RESUBMISSION	04/30/2021

DESSINÉ PAR Drawn by: **NR**
VÉRIFIÉ PAR Checked by: **TMK**
DATE: **01/04/21**
ÉCHELLE Scale: **1:250**

TITRE DU DESSIN Drawing Title

SITE SERVICING PLAN

RÉVISION Revision
NO. DESSIN Dwg Number: **C01**

[redacted]

D07-12-20-0035

#18143

NOTES:

1. THIS DRAWING IS TO BE READ IN CONJUNCTION WITH ARCHITECTURAL DRAWINGS BY NEUF ARCHITECTES SENCRL. REFER TO ARCHITECTURAL PLANS FOR DIMENSIONS.
2. ALL WATERMAINS TO BE INSULATED IF LESS THAN 2.4 METERS COVER AS PER CITY OF OTTAWA STANDARD DETAIL W22. AT ANY PROXIMITY OF SEWER MANHOLES, INSULATE WATERMAIN AS PER CITY DETAIL W23.
3. SEWERS ARE TO MAINTAIN 500mm BARRELL TO BARRELL CLEARANCE ABOVE AND 250mm BARRELL TO BARRELL CLEARANCE BELOW WATERMAINS AT ALL CROSSINGS (AS PER CITY DWGS W25 AND W25,2). IF 22° BENDS ARE USED ON THE WATER MAIN, THEY MUST BE ONE METER AWAY FROM THE SEWER.
4. THRUST BLOCKS TO BE AS PER CITY OF OTTAWA STANDARD DRAWINGS W25.3 AND W25.4. RESTRAINING AND RETAINING RINGS TO BE INSTALLED IN ACCORDANCE WITH CITY STANDARD DETAILS W25.5 AND W25.6.
5. TEMPORARY SUPPORT OF EXISTING UNDERGROUND UTILITIES IN ACCORDANCE WITH CITY STANDARD DETAIL W28.
6. WATERMAIN TRENCH AND BEDDING TO BE INSTALLED AS PER CITY DETAIL W17.
7. TAPPING VALVE SYSTEM CONNECTION TO CITY WATERMAIN BY CITY FORCES; EXCAVATION, BACKFILLING AND REINSTATEMENT BY CONTRACTOR.
8. CONTRACTOR SHALL BE RESPONSIBLE FOR LOCATION OF ALL EXISTING U/G AND OVERHEAD UTILITIES. VARIOUS UTILITIES CONCERNED TO BE GIVEN REQUIRED ADVANCE NOTICE PRIOR TO ANY DIGGING FOR STAKE OUT. THE OWNER AND CONSULTANT ASSUMES NO RESPONSIBILITY FOR THE ACCURACY OF THE LOCATION OF EXISTING UTILITIES AS INDICATED ON THIS DRAWING.
9. UTILITY INFORMATION WAS VERIFIED IN THE FIELD WHERE POSSIBLE. INDIVIDUAL COMPANIES SHOULD BE CONTACTED BY THE CONTRACTOR PRIOR TO COMMENCEMENT FOR CONFIRMATION OF EXISTENCE AND LOCATION OF UTILITIES.
10. WATER SERVICE, STORM SEWERS AND APPURTENANCE TO COMPLY WITH THE REQUIREMENTS OF THE LATEST REVISION OF THE ONTARIO PLUMBING CODE AND APPLICABLE CITY OF OTTAWA ENGINEERING STANDARDS.
11. ALL SANITARY SEWERS TO BE INSULATED IF LESS THAN 2.0 METERS COVER. ALL STORM SEWERS TO BE INSULATED IF LESS THAN 2.0 METERS COVER. INSULATE AS PER CITY OF OTTAWA STANDARD DETAIL W22. ALL BUILDING CONNECTIONS TO HAVE SUFFICIENT COVER OR INSULATION IS REQUIRED.
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15. PIPE BEDDING SHALL BE GRANULAR "A" AS PER CITY DETAIL S6, AND SHALL BE COMPACTED TO 95% SPD AND APPROVED SELECT NATIVE BACK FILL COMPACTED TO 95% SPD.
16. DRAWINGS TO BE READ IN CONJUNCTION WITH CONTRACT SPECIFICATIONS,
17. GRANULAR LAYERS BENEATH NEW ASPHALT SURFACES ON PROPERTY SHALL BE PLACED AT A THICKNESS NOT EXCEEDING 300mm. THE GRANULAR 'A' AND GRANULAR 'B' TYPE II IS TO BE COMPACTED TO A MINIMUM OF 100% SPMDD USING SUITABLE VIBRATORY EQUIPMENT.
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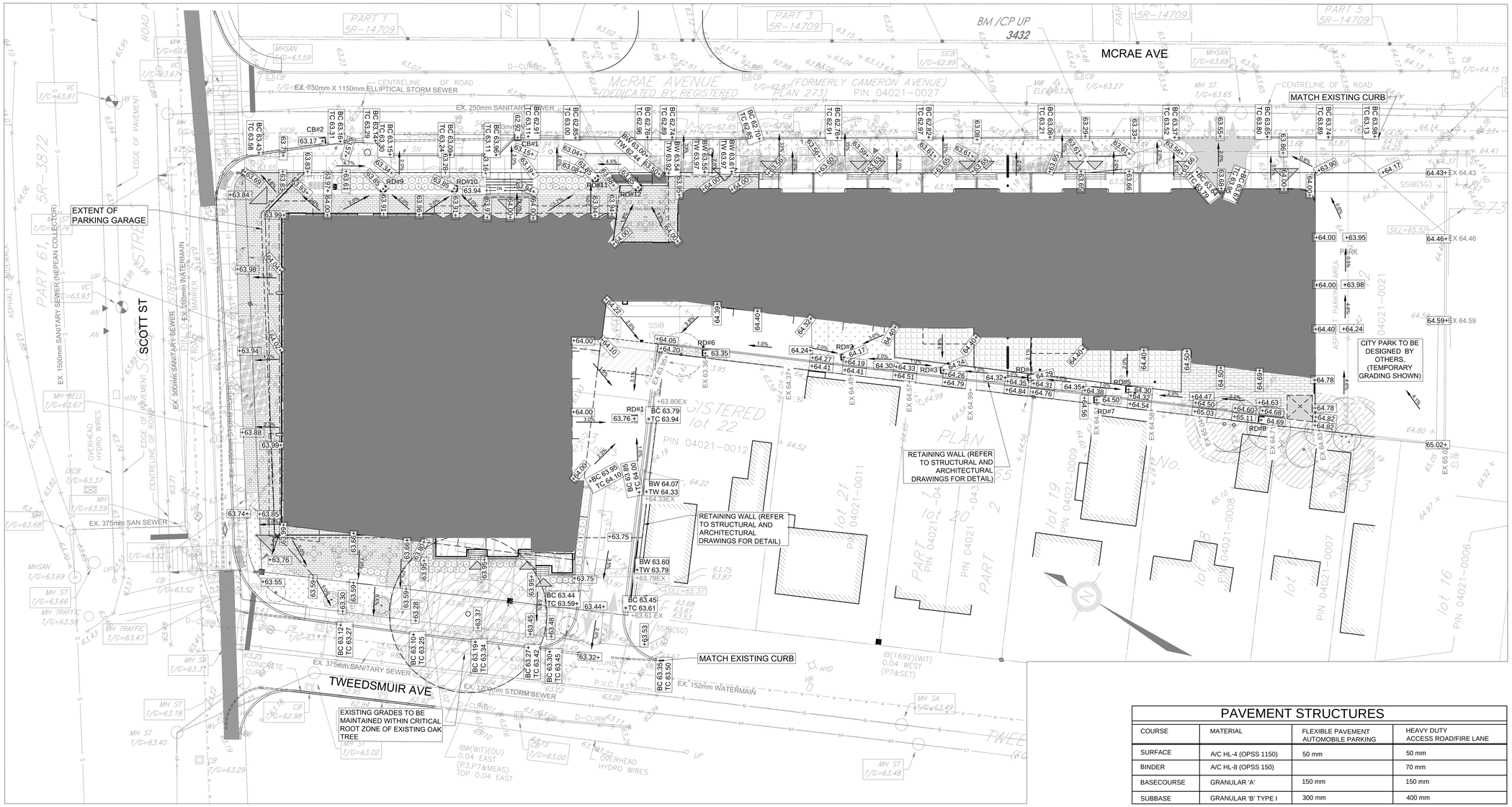

PAVEMENT STRUCTURES

COURSE	MATERIAL	FLEXIBLE PAVEMENT AUTOMOBILE PARKING	HEAVY DUTY ACCESS ROAD/FIRE LANE
SURFACE	A/C HL-4 (OPSS 1150)	50 mm	50 mm
BINDER	A/C HL-8 (OPSS 150)		70 mm
BASECOURSE	GRANULAR 'A'	150 mm	150 mm
SUBBASE	GRANULAR 'B' TYPE I	300 mm	400 mm

*NOTE: FOR DETAILED PAVEMENT STRUCTURE SPECIFICATIONS AND GUIDELINES, REFER TO THE GEOTECHNICAL INVESTIGATION, PROJECT FILE NO. 230236.004, DATED 28 NOVEMBER 2018, PREPARED BY PINCHIN LTD.

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ARCHITECTURE DE PAYSAGE Landscape Architect
James B. Lennox & Associates Inc.
3332 Carling Ave.
Ottawa, Ontario K2H 5A8
T 613 722 5168

CIVIL Civil
R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1844

ARPENTEUR Surveyor
Stantec Geomatics Ltd.
400 - 1331 Clyde Avenue
Ottawa, ON K2C 3G4
T 613 722 4420

URBANISTE Urban Planner
Fotenn Planning + Design
396 Cooper Street - Suite 300
Ottawa, ON, K2P 2H7
T 613 730 5709

ARCHITECTES Architect
NEUF architect(e)s SENCRL
630, boul. René-Lévesque O. 32e étages, Montreal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

SCEAU / Seal

LICENSED PROFESSIONAL ENGINEER
N. S. R. RODGERS
100163412
PROVINCE OF ONTARIO
Apr 30, 2021

arva
R.V. Anderson Associates Limited
engineering • environment • infrastructure

CLIENT Client
GWL REALTY ADVISORS

OUVRAGE Project
320 MCRAE

EMPLACEMENT Location
320 MCRAE

NO PROJET No.
194453

NO	REVISION	DATE
1	QA SUBMISSION	02/24/2020
2	FOR COORDINATION	02/26/2020
3	SITE PLAN SUBMISSION	03/20/2020
4	SITE PLAN RESUBMISSION	09/23/2020
5	EXCAVATION PERMIT	09/25/2020
6	FOR COORDINATION	12/22/2020
7	BUILDING PERMIT	01/04/2021
8	SITE PLAN RESUBMISSION	04/30/2021

DESSINÉ PAR Drawn by **NR**
VÉRIFIÉ PAR Checked by **TMK**
DATE **01/04/21**
ÉCHELLE Scale **1:250**

TITRE DU DESSIN Drawing Title
SITE GRADING AND STORM WATER MANAGEMENT PLAN

RÉVISION Revision
NO. DESSIN Dwg Number **C02**

D07-12-20-0035
#18143

EROSION AND SEDIMENT CONTROL NOTES:

GENERAL

THE CONTRACTOR ACKNOWLEDGES THAT SURFACE EROSION AND SEDIMENT RUNOFF RESULTING FROM HIS CONSTRUCTION OPERATIONS HAS POTENTIAL TO CAUSE A DETRIMENTAL IMPACT TO ANY DOWNSTREAM WATERCOURSE OR SEWER, AND THAT ALL CONSTRUCTION OPERATIONS THAT MAY IMPACT UPON WATER QUALITY SHALL BE CARRIED OUT IN A MANNER THAT STRICTLY MEETS THE REQUIREMENTS OF ALL APPLICABLE LEGISLATION AND REGULATIONS.

AS SUCH, THE CONTRACTOR SHALL BE RESPONSIBLE FOR CARRYING OUT HIS OPERATIONS, AND SUPPLYING AND INSTALLING ANY APPROPRIATE CONTROL MEASURES, SO AS TO PREVENT SEDIMENT LADEN RUNOFF FROM ENTERING ANY SEWER OR WATERCOURSE WITHIN OR DOWNSTREAM OF THE WORKING AREA.

THE CONTRACTOR ACKNOWLEDGES THAT NO ONE MEASURE IS LIKELY TO BE 100% EFFECTIVE FOR EROSION PROTECTION AND CONTROLLING SEDIMENT RUNOFF AND DISCHARGES FROM THE SITE. THEREFORE, WHERE NECESSARY THE CONTRACTOR SHALL IMPLEMENT SEQUENTIAL MEASURES ARRANGED IN SUCH A MANNER AS TO MITIGATE SEDIMENT RELEASE FROM THE CONSTRUCTION OPERATIONS AND ACHIEVE SPECIFIC MAXIMUM PERMITTED CRITERIA WHERE APPLICABLE. SUGGESTED ON-SITE MEASURES MAY INCLUDE, BUT SHALL NOT BE LIMITED TO, THE FOLLOWING METHODS: SEDIMENT PONDS, FILTER BAGS, PUMP FILTERS, SETTLING TANKS, SILT FENCES, STRAW BALES, FILTER CLOTHS, CATCH BASIN FILTERS, CHECK DAMS AND/OR BERMS, OR OTHER RECOGNIZED TECHNOLOGIES AND METHODS AVAILABLE AT THE TIME OF CONSTRUCTION. SPECIFIC MEASURES SHALL BE INSTALLED IN ACCORDANCE WITH THE REQUIREMENTS OF OPSS 805 WHERE APPROPRIATE, OR IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS.

WHERE, IN THE OPINION OF THE CONTRACT ADMINISTRATOR OR REGULATORY AGENCY, THE INSTALLED CONTROL MEASURES FAIL TO PERFORM ADEQUATELY, THE CONTRACTOR SHALL SUPPLY AND INSTALL ADDITIONAL OR ALTERNATIVE MEASURES AS DIRECTED BY THE CONTRACT ADMINISTRATOR OR REGULATORY AGENCY. AS SUCH, THE CONTRACTOR SHALL HAVE ADDITIONAL CONTROL MATERIALS ON SITE AT ALL TIMES WHICH ARE EASILY ACCESSIBLE AND MAY BE IMPLEMENTED BY HIM AT A MOMENT'S NOTICE.

BEFORE COMMENCING THE WORK, THE CONTRACTOR SHALL SUBMIT TO THE CONTRACT ADMINISTRATOR SIX COPIES OF A DETAILED EROSION AND SEDIMENT CONTROL PLAN (ESCP). THE ESCP WILL CONSIST OF A WRITTEN DESCRIPTION AND DETAILED DRAWINGS INDICATING THE ON-SITE ACTIVITIES AND MEASURES TO BE USED TO CONTROL EROSION AND SEDIMENT MOVEMENT FOR EACH STEP OF THE WORK.

CONTRACTOR'S RESPONSIBILITIES

THE CONTRACTOR IS RESPONSILBLE FOR MAINTAINING ALL ESC MEASURES IN WORKING CONDITION AT ALL TIMES, TO THE SATISFACTORY OF THE CONTRACT ADMINISTRATOR AND CITY. THE CONTRACTOR SHALL ROUTINELY INSPECT ALL ESC MEASURES AT A MINIMUM ONCE A WEEK AND AFTER EACH RAINFALL EVENT GREATER THAN 10MM TO ENSURE THAT ESC MEASURES ARE IIN PROPER WORKING CONDITIONS. ANY DAMAGES MUST BE REPAIRED WITHING 24 HOURS.

THE CONTRACTOR IS RESPONSIBLE TO KEEP THE ROADS FREE AND CLEAN FROM MUD OR DEBRIS. ROADS TO BE INSPECTED DAILY AND CLEANED AS REQUIRED.

THE CONTRACTOR SHALL ENSURE THAT ALL WORKERS, INCLUDING SUB-CONTRACTORS, IN THE WORKING AREA ARE AWARE OF THE IMPORTANCE OF THE EROSION AND SEDIMENT CONTROL MEASURES AND INFORMED OF THE CONSEQUENCES OF THE FAILURE TO COMPLY WITH THE REQUIREMENTS OF ALL REGULATORY AGENCIES AND THE SPECIFICATIONS DETAILED HEREIN.

THE CONTRACTOR SHALL PERIODICALLY, AND WHEN REQUESTED BY THE CONTRACT ADMINISTRATOR, CLEAN OUT ACCUMULATED SEDIMENT DEPOSITS AS REQUIRED AT THE SEDIMENT CONTROL DEVICES, INCLUDING THOSE DEPOSITS THAT MAY ORIGINATE FROM OUTSIDE THE CONSTRUCTION AREA. ACCUMULATED SEDIMENT SHALL BE REMOVED IN SUCH A MANNER THAT PREVENTS THE DEPOSITION OF THIS MATERIAL INTO ANY SEWER OR WATERCOURSE AND AVOIDS DAMAGE TO THE CONTROL MEASURE. THE SEDIMENT SHALL BE REMOVED FROM THE SITE AT THE CONTRACTOR'S EXPENSE AND MANAGED IN COMPLIANCE WITH THE REQUIREMENTS FOR EXCESS EARTH MATERIAL, AS SPECIFIED ELSEWHERE IN THE CONTRACT.

THE CONTRACTOR SHALL IMMEDIATELY REPORT TO THE CONTRACT ADMINISTRATOR ANY ACCIDENTAL DISCHARGES OF SEDIMENT MATERIAL INTO EITHER THE WATERCOURSE OR THE STORM SEWER SYSTEM. FAILURE TO REPORT WILL BE CONSTITUTE A BREACH OF THIS SPECIFICATION AND THE CONTRACTOR MAY ALSO BE SUBJECT TO THE PENALTIES IMPOSED BY ANY APPLICABLE REGULATORY AGENCY. APPROPRIATE RESPONSE MEASURES, INCLUDING ANY REPAIRS TO EXISTING CONTROL MEASURES OR THE IMPLEMENTATION OF ADDITIONAL CONTROL MEASURES, SHALL BE CARRIED OUT BY THE CONTRACTOR WITHOUT DELAY.

THE SEDIMENT CONTROL MEASURES SHALL ONLY BE REMOVED WHEN, IN THE OPINION OF THE CONTRACT ADMINISTRATOR, THE MEASURE OR MEASURES, IS NO LONGER REQUIRED. NO CONTROL MEASURE MAY BE PERMANENTLY REMOVED WITHOUT PRIOR AUTHORIZATION FROM THE CONTRACT ADMINISTRATOR. ALL SEDIMENT AND EROSION CONTROL MEASURES SHALL BE REMOVED IN A MANNER THAT AVOIDS THE ENTRY OF ANY EQUIPMENT, OTHER THAN HAND-HELD EQUIPMENT, INTO ANY WATERCOURSE, AND PREVENTS THE RELEASE OF ANY SEDIMENT OR DEBRIS INTO ANY SEWER OR WATERCOURSE WITHIN OR DOWNSTREAM OF THE WORKING AREA. ALL ACCUMULATED SEDIMENT SHALL BE REMOVED FROM THE WORKING AREA AT THE CONTRACTOR'S EXPENSE AND MANAGED IN COMPLIANCE WITH THE REQUIREMENTS FOR EXCESS EARTH MATERIAL, AS SPECIFIED ELSEWHERE IN THE CONTRACT.

CONTRACTOR'S RESPONSIBILITIES CONTINUED

WHERE, IN THE OPINION OF EITHER THE CONTRACT ADMINISTRATOR OR A REGULATORY AGENCY, ANY OF THE TERMS SPECIFIED HEREIN HAVE NOT BEEN COMPLIED WITH OR PERFORMED IN A SUITABLE MANNER, OR AT ALL, THE CONTRACT ADMINISTRATOR OR REGULATORY AGENCY HAS THE RIGHT TO IMMEDIATELY WITHDRAW ITS PERMISSION TO CONTINUE THE WORK BUT MAY RENEW ITS PERMISSION UPON BEING SATISFIED THAT THE DEFAULTS OR DEFICIENCIES IN THE PERFORMANCE OF THIS SPECIFICATION BY THE CONTRACTOR HAVE BEEN REMEDIED.

ALL STOCKPILES SHALL BE SURROUNDED WITH SEDIMENT CONTROL FENCING.

THE CONTRACTOR IS RESPONSIBLE TO IMPLEMENT DUST CONTROL MEASURES AND CONSTRUCTION PRACTICE GUIDELINES AS APPROVED BY THE CITY.

ALL DISTURBED GROUND LEFT INACTIVE FOR OVER 14 DAYS SHALL BE STABILIZED BY VEGETATIVE OR NON VEGETATIVE METHODS TO THE SATISFACTION OF THE CONTRACT ADMINISTRATOR.

ALL SOIL STOCKPILES LEFT INACTIVE FOR OVER 14 DAYS SHALL BE STABILIZED BY VEGETATIVE OR NON VEGETATIVE METHODS TO THE SATISFACTION OF THE CONTRACT ADMINISTRATOR.

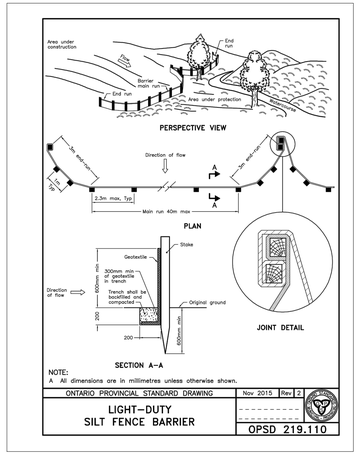
SEDIMENT TO BE REMOVED FROM SITE BEFORE IT HAS ACCUMULATED TO ONE-HALF OF THE ABOVE GROUND HEIGHT OF PERIMITER FENCING.

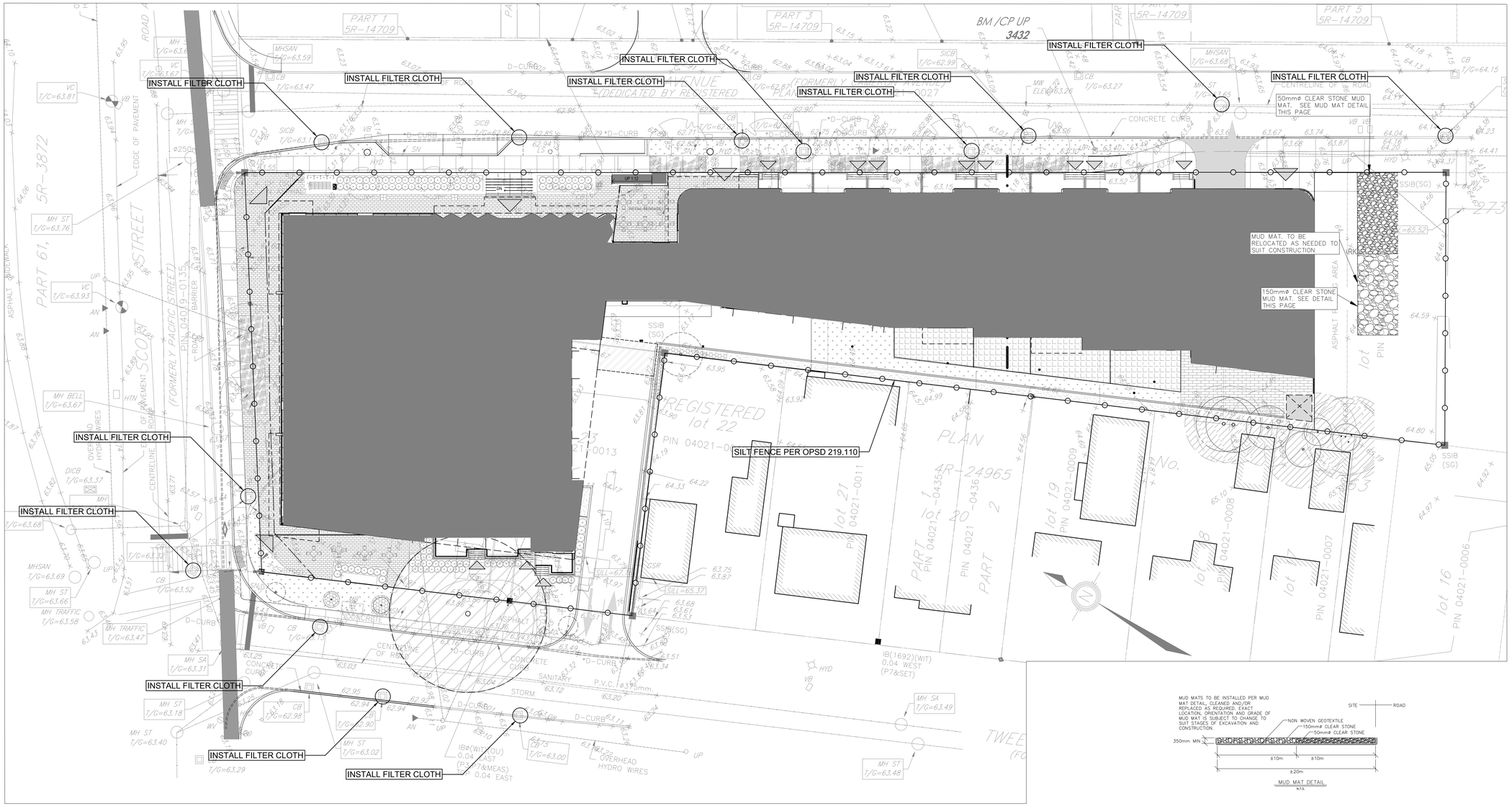
SITE SPECIFIC NOTES:

EROSION AND SEDIMENT CONTROL MEASURES (IN ACCORDANCE WITH THE REQUIREMENTS OF OPSS 805 - NOVEMBER 2018 FOR TEMPORARY MEASURES) CONSISTING OF BOTH PERMANENT AND TEMPORARY MEASURES SHALL BE IMPLEMENTED PRIOR TO THE COMMENCEMENT OF CONSTRUCTION ACTIVITIES TO ENSURE THAT SEDIMENT IS CONTAINED WITHIN THE SITE,. FILTER FABRIC SHALL BE INSTALLED UNDER THE FRAME OF ALL PROPOSED AND EXISTING CATCHBASINS AND STORM MANHOLES IMMEDIATELY ADJACENT TO ANY DISTURBED AREAS PRIOR TO CONSTRUCTION TO PREVENT SEDIMENT FROM ENTERING INTO THE STORM SEWER SYSTEM. THE FILTER FABRIC SHALL REMAIN IN-PLACE FOR THE DURATION OF CONSTRUCTION ACTIVITIES AND SHALL NOT BE REMOVED UNTIL SUCH TIME AS THE LANDSCAPING HAS BEEN ESTABLISHED AND UPON AUTHORIZATION BY THE ENGINEER. LIGHT DUTY SEDIMENT FENCINGPER OPSD 219.110 SHALL ALSO BE PLACED AROUND THE PERIMETER OF THE SITE FOR THE DURATION OF THE CONSTRUCTION.

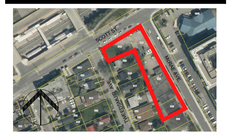
THE CONTRACTOR ACKNOWLEDGES THAT FAILURE TO IMPLEMENT APPROPRIATE EROSION AND SEDIMENT CONTROL MEASURES MAY BE SUBJECT TO PENALTIES IMPOSED BY ANY APPLICABLE REGULATORY AGENCY.

SEDIMENT AND EROSION CONTROL MEASUREMENTS MAY BE MODIFIED IN THE FIELD AT THE DISCRETION OF THE CITY OF OTTAWA SITE INSPECTOR OR CONSERVATION AUTHORITY.





NOT FOR CONSTRUCTION - SUBJECT TO CHANGE PENDING OUTSTANDING APPROVALS



ARCHITECTURE DE PAYSAGE Landscape Architect
James B. Lennox & Associates Inc.
3332 Carling Ave.
Ottawa, Ontario K2H 5A8
T 613 722 5168

CIVIL Civil
R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1844

ARPENTEUR Surveyor
Stantec Geomatics Ltd.
400 - 1331 Clyde Avenue
Ottawa, ON K2C 3G4
T 613 722 4420

URBANISTE Urban Planner
Fotenn Planning + Design
396 Cooper Street - Suite 300
Ottawa, ON, K2P 2H7
T 613 730 5709

ARCHITECTES Architect
NEUF architect(e)s SENCRL
630, boul. René-Lévesque O. 32e étages, Montreal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

SCEAU / Seal

LICENSED PROFESSIONAL ENGINEER
N. S. R. RODGERS
100163412
PROVINCE OF ONTARIO
Apr 30, 2021

arva
R.V. Anderson Associates Limited
engineering • environment • infrastructure

CLIENT Client
GWL REALTY ADVISORS

OUVRAGE Project
320 MCRAE

EMPLACEMENT Location
320 MCRAE

NO PROJET No.
194453

NO	REVISION	DATE
1	QA SUBMISSION	02/24/2020
2	FOR COORDINATION	02/26/2020
3	SITE PLAN SUBMISSION	03/20/2020
4	SITE PLAN RESUBMISSION	09/23/2020
5	EXCAVATION PERMIT	09/25/2020
6	FOR COORDINATION	12/22/2020
7	BUILDING PERMIT	01/04/2021
8	SITE PLAN RESUBMISSION	04/30/2021

DESSINÉ PAR Drawn by
NR

VÉRIFIÉ PAR Checked by
TMK

DATE
01/04/21

ÉCHELLE Scale
1:250

TITRE DU DESSIN Drawing Title
SITE EROSION AND SEDIMENT CONTROL PLAN

RÉVISION Revision
[redacted]

NO. DESSIN Dwg Number
C03

D07-12-20-0035

#18143

APPENDIX 2

Stormwater Design Calculations & Correspondence

320 McRae

CAPTURED FLOW
5 yr Storm Post-Development Flow

100 yr Average Discharge	**12.80 L/s**
(based on 20 minute time of concentration)	

Elapsed time (min)	Elapsed time (s)	Intensity (mm/hr)	Intensity (mm/s)	Acc Depth (mm)	C	Area (m^2)	Flow (l/s)	Discharge (l/s)	Storage flow (l/s)	Storage volume (m^3)	
0	0	0.00	0.0000	0.00	0.88	4374	0.00	0.00	0.00	0.00	
5	300	141.18	0.0392	11.76	0.88	4374	75.47	12.80	62.67	18.80	
10	600	104.19	0.0289	20.45	0.88	4374	111.40	12.80	98.60	59.16	
15	900	83.56	0.0232	27.41	0.88	4374	89.34	12.80	76.54	68.89	
20	1200	70.25	0.0195	33.26	0.88	4374	75.11	12.80	62.31	74.77	
25	1500	60.90	0.0169	38.34	0.88	4374	65.11	12.80	52.31	78.47	
30	1800	53.93	0.0150	42.83	0.88	4374	57.66	12.80	44.86	80.75	
35	2100	48.52	0.0135	46.88	0.88	4374	51.87	12.80	39.07	82.06	
40	2400	44.18	0.0123	50.56	0.88	4374	47.24	12.80	34.44	82.66	
45	2700	40.63	0.0113	53.94	0.88	4374	43.44	12.80	30.64	82.73	-peak storage
50	3000	37.65	0.0105	57.08	0.88	4374	40.26	12.80	27.46	82.38	
55	3300	35.12	0.0098	60.01	0.88	4374	37.55	12.80	24.75	81.69	
60	3600	32.94	0.0092	62.75	0.88	4374	35.22	12.80	22.42	80.72	
65	3900	31.04	0.0086	65.34	0.88	4374	33.19	12.80	20.39	79.53	
70	4200	29.37	0.0082	67.79	0.88	4374	31.40	12.80	18.60	78.14	
75	4500	27.89	0.0077	70.11	0.88	4374	29.82	12.80	17.02	76.58	
80	4800	26.56	0.0074	72.33	0.88	4374	28.40	12.80	15.60	74.88	
85	5100	25.37	0.0070	74.44	0.88	4374	27.12	12.80	14.32	73.05	
90	5400	24.29	0.0067	76.46	0.88	4374	25.97	12.80	13.17	71.11	
95	5700	23.31	0.0065	78.41	0.88	4374	24.92	12.80	12.12	69.07	
100	6000	22.41	0.0062	80.27	0.88	4374	23.96	12.80	11.16	66.95	
105	6300	21.58	0.0060	82.07	0.88	4374	23.08	12.80	10.28	64.74	
110	6600	20.82	0.0058	83.81	0.88	4374	22.26	12.80	9.46	62.46	
115	6900	20.12	0.0056	85.48	0.88	4374	21.51	12.80	8.71	60.11	
120	7200	19.47	0.0054	87.11	0.88	4374	20.81	12.80	8.01	57.71	
125	7500	18.86	0.0052	88.68	0.88	4374	20.17	12.80	7.37	55.24	
130	7800	18.29	0.0051	90.20	0.88	4374	19.56	12.80	6.76	52.73	
135	8100	17.76	0.0049	91.68	0.88	4374	18.99	12.80	6.19	50.17	
140	8400	17.27	0.0048	93.12	0.88	4374	18.46	12.80	5.66	47.57	
145	8700	16.80	0.0047	94.52	0.88	4374	17.96	12.80	5.16	44.93	
150	9000	16.36	0.0045	95.89	0.88	4374	17.49	12.80	4.69	42.25	
155	9300	15.95	0.0044	97.22	0.88	4374	17.05	12.80	4.25	39.53	
160	9600	15.56	0.0043	98.51	0.88	4374	16.63	12.80	3.83	36.79	
165	9900	15.18	0.0042	99.78	0.88	4374	16.23	12.80	3.43	34.01	
170	10200	14.83	0.0041	101.01	0.88	4374	15.86	12.80	3.06	31.20	
175	10500	14.50	0.0040	102.22	0.88	4374	15.50	12.80	2.70	28.36	
180	10800	14.18	0.0039	103.40	0.88	4374	15.16	12.80	2.36	25.50	

Flow Calculations:
For 5m (300s) interval
t/600*A*C*I
(300)/600*1076*0.95*0.0392=20.04

320 McRae

CAPTURED FLOW
100 yr Storm Post-Development Flow

100 yr Average Discharge	**12.80 L/s**
(based on 20 minute time of concentration)	

Elapsed time (min)	(s)	Intensity (mm/hr)	(mm/s)	Acc Depth (mm)	C	Area (m^2)	Flow (l/s)	Discharge (l/s)	Storage flow (l/s)	Storage volume (m^3)	
0	0	0.00	0.0000	0.00	0.94	4374	0.00	0.00	0.00	0.00	
5	300	242.70	0.0674	20.23	0.94	4374	138.60	12.80	125.80	37.74	
10	600	178.56	0.0496	35.11	0.94	4374	203.93	12.80	191.13	114.68	
15	900	142.89	0.0397	47.01	0.94	4374	163.20	12.80	150.40	135.36	
20	1200	119.95	0.0333	57.01	0.94	4374	137.00	12.80	124.20	149.03	
25	1500	103.85	0.0288	65.66	0.94	4374	118.60	12.80	105.80	158.71	
30	1800	91.87	0.0255	73.32	0.94	4374	104.92	12.80	92.12	165.82	
35	2100	82.58	0.0229	80.20	0.94	4374	94.31	12.80	81.51	171.18	
40	2400	75.15	0.0209	86.46	0.94	4374	85.82	12.80	73.02	175.26	
45	2700	69.05	0.0192	92.22	0.94	4374	78.86	12.80	66.06	178.37	
50	3000	63.95	0.0178	97.55	0.94	4374	73.04	12.80	60.24	180.73	
55	3300	59.62	0.0166	102.51	0.94	4374	68.10	12.80	55.30	182.48	
60	3600	55.89	0.0155	107.17	0.94	4374	63.84	12.80	51.04	183.73	
65	3900	52.65	0.0146	111.56	0.94	4374	60.13	12.80	47.33	184.58	
70	4200	49.79	0.0138	115.71	0.94	4374	56.86	12.80	44.06	185.07	
75	4500	47.26	0.0131	119.65	0.94	4374	53.97	12.80	41.17	185.27	-peak storage
80	4800	44.99	0.0125	123.40	0.94	4374	51.38	12.80	38.58	185.20	
85	5100	42.95	0.0119	126.98	0.94	4374	49.06	12.80	36.26	184.91	
90	5400	41.11	0.0114	130.40	0.94	4374	46.95	12.80	34.15	184.43	
95	5700	39.43	0.0110	133.69	0.94	4374	45.04	12.80	32.24	183.76	
100	6000	37.90	0.0105	136.85	0.94	4374	43.29	12.80	30.49	182.93	
105	6300	36.50	0.0101	139.89	0.94	4374	41.68	12.80	28.88	181.97	
110	6600	35.20	0.0098	142.82	0.94	4374	40.20	12.80	27.40	180.87	
115	6900	34.01	0.0094	145.65	0.94	4374	38.84	12.80	26.04	179.66	
120	7200	32.89	0.0091	148.40	0.94	4374	37.57	12.80	24.77	178.34	
125	7500	31.86	0.0089	151.05	0.94	4374	36.39	12.80	23.59	176.92	
130	7800	30.90	0.0086	153.63	0.94	4374	35.29	12.80	22.49	175.41	
135	8100	30.00	0.0083	156.13	0.94	4374	34.26	12.80	21.46	173.82	
140	8400	29.15	0.0081	158.56	0.94	4374	33.29	12.80	20.49	172.15	
145	8700	28.36	0.0079	160.92	0.94	4374	32.39	12.80	19.59	170.41	
150	9000	27.61	0.0077	163.22	0.94	4374	31.53	12.80	18.73	168.61	
155	9300	26.91	0.0075	165.46	0.94	4374	30.73	12.80	17.93	166.74	
160	9600	26.24	0.0073	167.65	0.94	4374	29.97	12.80	17.17	164.81	
165	9900	25.61	0.0071	169.78	0.94	4374	29.25	12.80	16.45	162.83	
170	10200	25.01	0.0069	171.87	0.94	4374	28.56	12.80	15.76	160.80	
175	10500	24.44	0.0068	173.90	0.94	4374	27.92	12.80	15.12	158.72	
180	10800	23.90	0.0066	175.90	0.94	4374	27.30	12.80	14.50	156.59	
185	11100	23.39	0.0065	177.84	0.94	4374	26.71	12.80	13.91	154.42	
190	11400	22.90	0.0064	179.75	0.94	4374	26.15	12.80	13.35	152.21	
195	11700	22.43	0.0062	181.62	0.94	4374	25.62	12.80	12.82	149.96	
200	12000	21.98	0.0061	183.45	0.94	4374	25.11	12.80	12.31	147.67	
205	12300	21.55	0.0060	185.25	0.94	4374	24.62	12.80	11.82	145.35	
210	12600	21.14	0.0059	187.01	0.94	4374	24.15	12.80	11.35	143.00	
215	12900	20.75	0.0058	188.74	0.94	4374	23.70	12.80	10.90	140.61	
220	13200	20.37	0.0057	190.44	0.94	4374	23.27	12.80	10.47	138.20	
225	13500	20.01	0.0056	192.11	0.94	4374	22.86	12.80	10.06	135.75	
230	13800	19.66	0.0055	193.75	0.94	4374	22.46	12.80	9.66	133.28	

Flow Calculations:
For 5m (300s) interval
t/600*A*C*I
(300)/600*1076*1*0.0674=36.27

Nathaniel Rodgers

From: Wu, John <John.Wu@ottawa.ca>
Sent: Monday, December 16, 2019 2:36 PM
To: Elizabeth Rodgers
Subject: RE: 320 McRae - Pre-Consultation - SWM Criteria & Site Servicing Constraints

Follow Up Flag: Follow up
Flag Status: Flagged

C 0.5 5year's to restrict up to 100 years' storm, Tc 20 minutes

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: December 16, 2019 2:26 PM
To: Wu, John <John.Wu@ottawa.ca>
Cc: Jaime Posen <posen@fotenn.com>
Subject: 320 McRae - Pre-Consultation - SWM Criteria & Site Servicing Constraints

Hi John,

Further to the pre-consultation meeting for the 320 McRae development (which took place on May 9, 2019), it is my understanding that you are the engineering department contact from the pre-application consultation.
I'm looking for information on servicing and stormwater management constraints for the site.

RVA will be preparing the Site Servicing and Stormwater Management design. Can you please provide the stormwater management criteria for the site and any servicing restrictions?

Thanks,
Beth Rodgers (Hamley), P.Eng.
Project Engineer

R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1284 x 3226
website | **facebook** | **twitter** | **linkedin**

Note that my email address has recently changed to ERodgers@rvanderson.com
Please update your records accordingly.

Elizabeth Rodgers

From: Jamie Batchelor <jamie.batchelor@rvca.ca>
Sent: January 28, 2020 8:53 AM
To: Elizabeth Rodgers
Cc: Nathaniel Rodgers
Subject: RE: 320 McRae - WQ Control Requirements

[CAUTION EXTERNAL EMAIL] Make Sure that it is legitimate before Replying or Clicking on any links

Good Morning Beth,

Based on the site plan provided there are no surface parking spaces and the development will be primarily rooftop area receiving rainwater. Roofs and landscaped areas, for the purpose of protecting surface water quality and aquatic habitat, are deemed as clean. The RVCA therefore accepts that the stormwater runoff from the site does not require any additional quality control measures save and except best management practices.

Jamie Batchelor, MCIP, RPP
Planner, ext. 1191
Jamie.batchelor@rvca.ca

Rideau Valley Conservation Authority
3889 Rideau Valley Drive
PO Box 599, Manotick ON K4M 1A5
T 613-692-3571 | 1-800-267-3504 F 613-692-0831 | www.rvca.ca

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: Friday, January 24, 2020 2:16 PM
To: Jamie Batchelor <jamie.batchelor@rvca.ca>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: RE: 320 McRae - WQ Control Requirements

Hi Jamie,

Please find attached the draft site plan for 320 McRae. There are no proposed surface parking spaces within the site; however, there is an access laneway (for garbage trucks, moving trucks, etc) within the site. There is also a ramp to the underground parking garage.

Let me know if you need more info or would like to discuss.

Thanks,

Beth Rodgers (Hamley), P.Eng.
Associate, Project Engineer

R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1284 x 3226
website | **facebook** | **twitter** | **linkedin**

Note that my email address has recently changed to ERodgers@rvanderson.com
Please update your records accordingly.

From: Jamie Batchelor <jamie.batchelor@rvca.ca>
Sent: January 22, 2020 2:09 PM
To: Elizabeth Rodgers <erodgers@rvanderson.com>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: RE: 320 McRae - WQ Control Requirements

Good Afternoon Beth,

Can you provide me with a site plan? I need to know hoe many surface parking spaces are being provided. Thanks.

Jamie Batchelor, MCIP, RPP
Planner, ext. 1191
Jamie.batchelor@rvca.ca

Rideau Valley Conservation Authority
3889 Rideau Valley Drive
PO Box 599, Manotick ON K4M 1A5
T 613-692-3571 | 1-800-267-3504 F 613-692-0831 | www.rvca.ca

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: Tuesday, January 21, 2020 10:08 AM
To: Jamie Batchelor <jamie.batchelor@rvca.ca>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: 320 McRae - WQ Control Requirements

Hi Jamie,

We're working on redevelopment of another site in Ottawa, at 320 McRae Ave. The sanitary and storm sewers in this area are separated; therefore, we will have separate laterals for stormwater and sanitary discharge from the site. Can you please provide stormwater quality control requirements for this site?

If you need more information on the project, please let me know.

Thanks,
Beth Rodgers (Hamley), P.Eng.
Associate, Project Engineer

R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1284 x 3226
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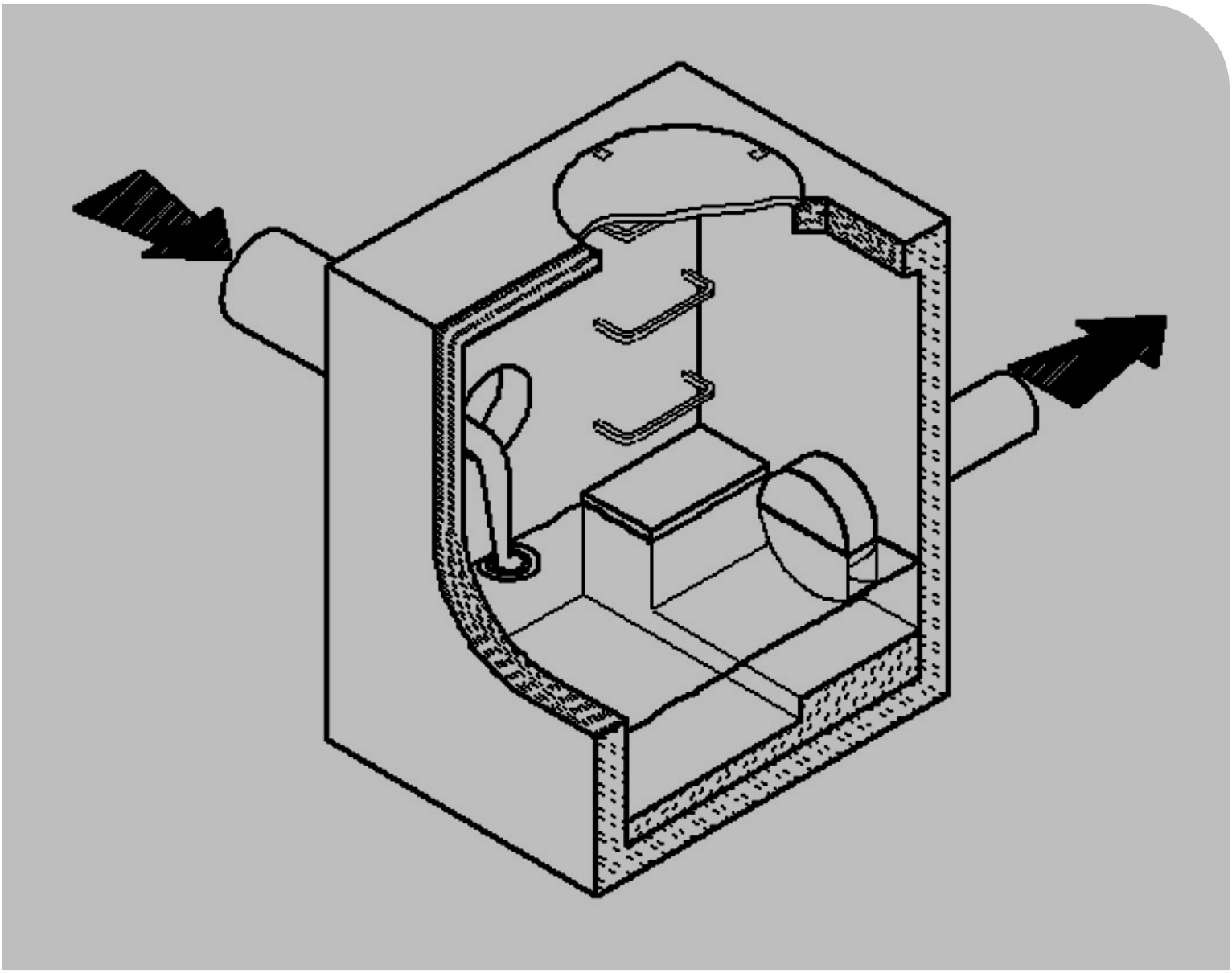


VEOLIA

HYDROVEX® VHV/SVHV
Vertical Vortex Flow Regulator
CSO, SSO, Stormwater Management

WATER TECHNOLOGIES

HYDROVEX® VHV / SVHV Vertical Vortex Flow Regulator

Application

One of the major problems of urban wet weather flow management is the runoff generated by heavy rainfall. During a storm event, uncontrolled flows may overload the drainage system and cause flooding. Wear and deterioration on the network are increased dramatically as a result of increased flow velocities. In a combined sewer system, the wastewater treatment plant will experience a significant increase in flows during storms, thereby losing its treatment efficiency. A simple means of managing excessive storm water runoff is to control the flows at their point of origin, the manhole.

The HYDROVEX® VHV / SVHV line of vortex flow regulators is ideal for point source control of low to medium stormwater flows in manholes, catch basins and other retention structures. The HYDROVEX® VHV / SVHV design is based on the fluid mechanics principle of the forced vortex. The discharge is controlled by an air-filled vortex which reduces the effective water passage area without physically reducing orifice size. This effect grants precise flow regulation without the use of moving parts or electricity, and allows for larger inlet and outlet openings compared to the basic orifice. Although the concept is quite simple, many years of research and testing have been invested to optimize the performance of our vortex technology.

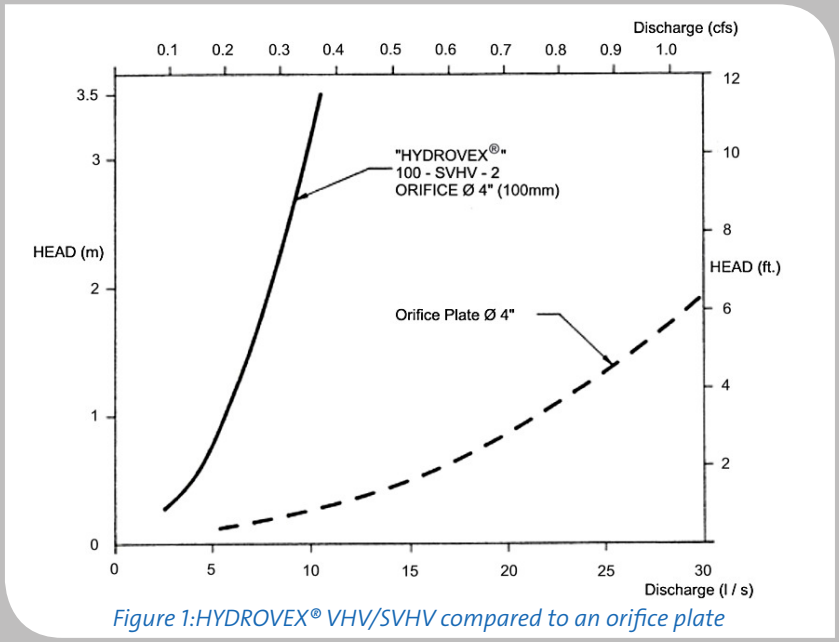


Figure 1:HYDROVEX® VHV/SVHV compared to an orifice plate

Vortex valves have openings typically 4 to 6 times larger than an orifice plate for the same design. Larger opening sizes decrease the chance of blockage caused by sediments and debris found in storm water flows. Figure 1 shows the discharge curve of a vortex regulator compared to an equally sized orifice plate. For an identical opening size, the flow is approximately four times smaller than the orifice plate for the same upstream water pressure.

Advantages

- Large inlet/outlet openings reduce the chance of clogging
- Openings typically 4-6 times larger than the basic orifice (Figure 1)
- Outlet orifice always equal or larger than inlet
- Ideal for precise control of low to medium stormwater flow applications
- Submerged inlet for floatables control
- No moving parts or electricity required
- Durable and robust stainless steel construction
- Minimal maintenance
- Easy to install

Selection

Selecting a VHV/SVHV regulator is easily achieved using Figure 3. Each selection is made using the maximum allowable flow rate and the maximum allowable upstream water pressure (head). The area in which the design point falls will designate the required model. The maximum design head is defined as the difference between the maximum upstream water level and the invert of the outlet pipe. All selections should be verified by a John Meunier Inc. representative prior to fabrication.

Design example:

- Maximum discharge: 6 L/s (0.2 cfs)*
- Maximum design head 2m (6.56 ft.)**
- Using Figure 3, model 75 VHV-1 is selected

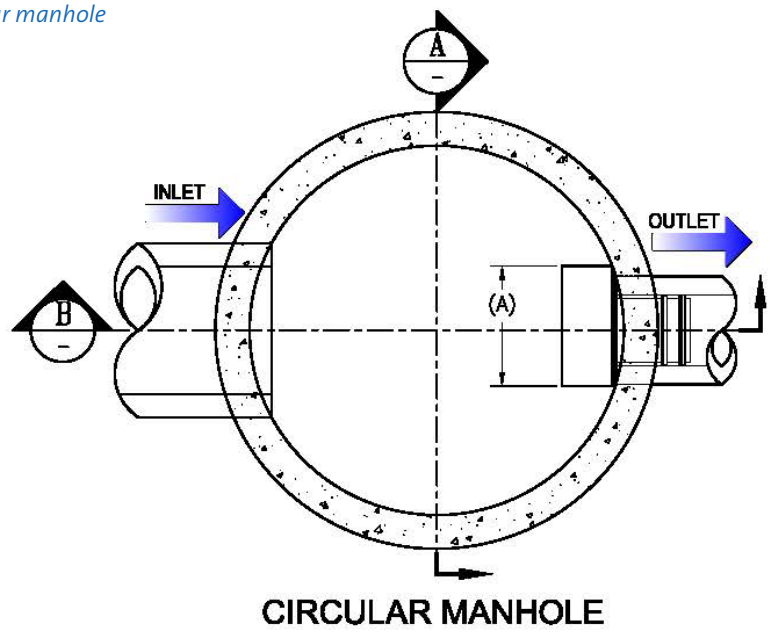
**The selection chart provided assumes free flowing downstream conditions. Should the outlet pipe be >80% full at design flow, a larger pipe diameter should be used. In the above example, the minimum outlet pipe diameter and slope would be 150mm (6in), 0.3%.*

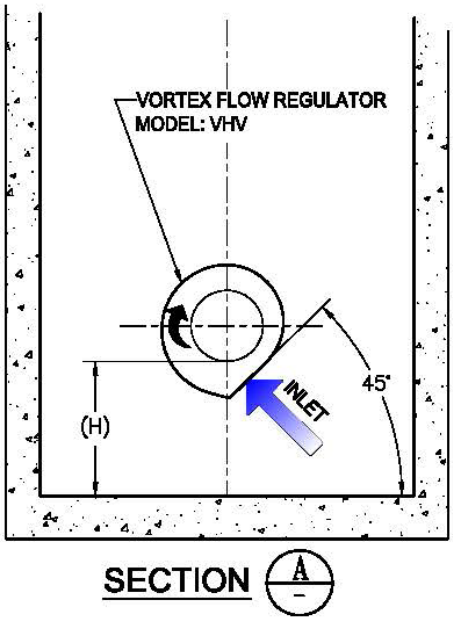
***The design head is defined as the difference between the maximum upstream water level and the outlet pipe invert.*

The HYDROVEX® VHV / SVHV vortex flow regulators can be installed in circular or square manholes. The table below lists the minimum dimensions and clearances required for each regulator model. It is imperative to respect the minimum clearances shown to ensure ease of installation and proper functioning of the regulator.

Model	Regulator Diameter A (mm) [in]	CIRCULAR Minimum Manhole Diameter B (mm) [in]	SQUARE Minimum Chamber Width B (mm) [in]	Minimum Outlet Pipe Diameter C (mm) [in]	Minimum Clearance H (mm) [in]
25 SVHV-1	125 [5]	600 [24]	600 [24]	150 [6]	150 [6]
32 SVHV-1	150 [6]	600 [24]	600 [24]	150 [6]	150 [6]
40 SVHV-1	200 [8]	600 [24]	600 [24]	150 [6]	150 [6]
50 VHV-1	150 [6]	600 [24]	600 [24]	150 [6]	150 [6]
75 VHV-1	250 [10]	600 [24]	600 [24]	150 [6]	150 [6]
100 VHV-1	325 [13]	900 [36]	600 [24]	150 [6]	200 [8]
125 VHV-2	275 [11]	900 [36]	600 [24]	150 [6]	200 [8]
150 VHV-2	350 [14]	900 [36]	600 [24]	150 [6]	225 [9]
200 VHV-2	450 [18]	1200 [48]	900 [36]	200 [8]	300 [12]
250 VHV-2	575 [23]	1200 [48]	900 [36]	250 [10]	350 [14]
300 VHV-2	675 [27]	1600 [64]	1200 [48]	250 [10]	400 [16]
350 VHV-2	800 [32]	1800 [72]	1200 [48]	300 [12]	500 [20]

Figure 2a: Minimum dimensions and clearances, circular manhole





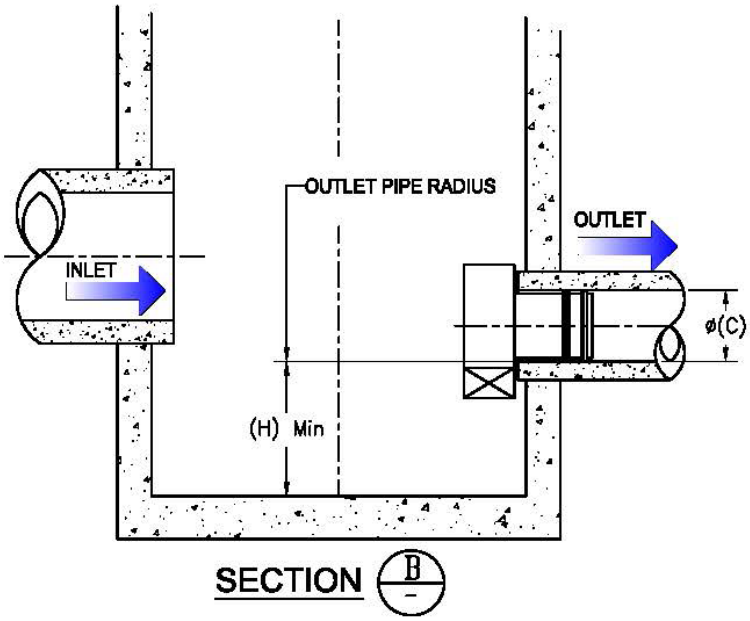
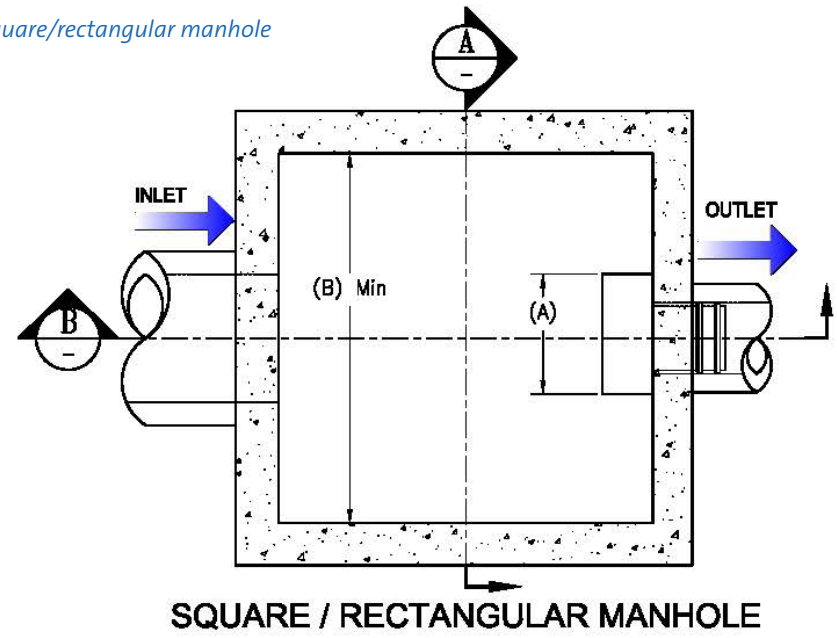
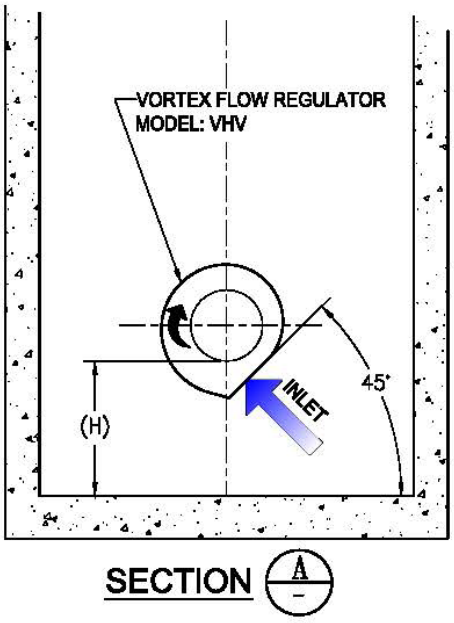
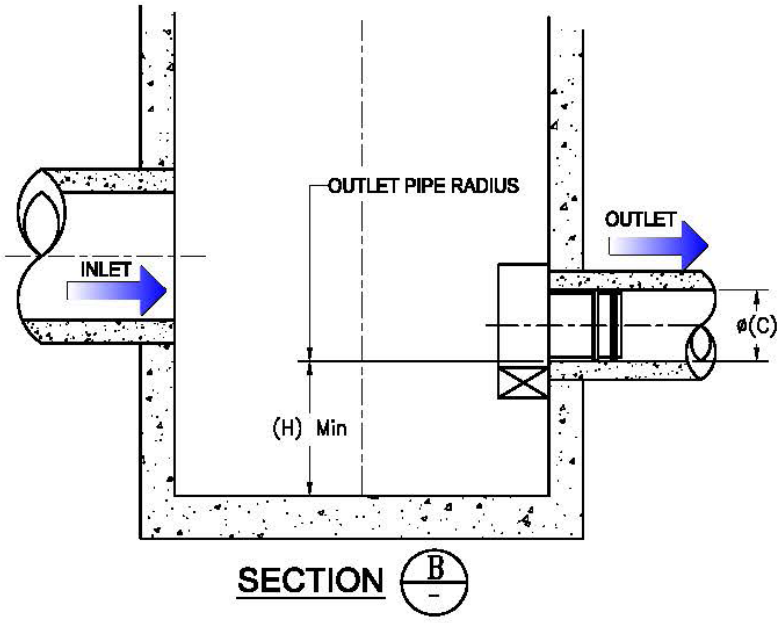


Figure 2b: Minimum dimensions and clearances, square/rectangular manhole







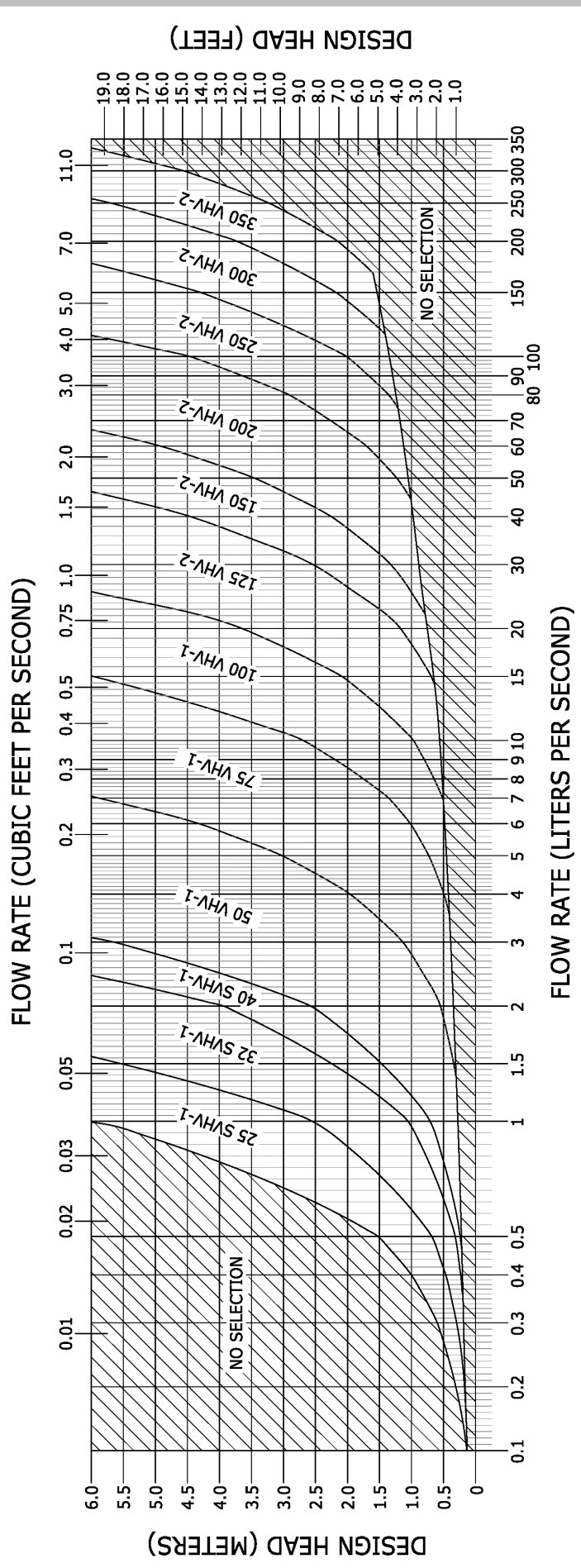


Figure 3 : HYDROVEX® VHV/SVHV Selection Chart

Options

A variety of options are available for the HYDROVEX® VHV / SVHV vortex flow regulators, including:

- Type O: extended inlet for odor control
- FV-VHV: sliding plate mounted
- Gooseneck: for shallow or no sump installations
- Vent: for low slope applications

DT: roof drainage applications

Specifications

In order to specify a HYDROVEX® VHV/SVHV flow regulator, the following parameters must be clearly indicated:

- Model number, ex: 75-VHV-1
- Outlet pipe diameter and type, ex: ø 150mm [6”], SDR 35
- Design discharge rate, ex: 6.0 L/s [0.21 CFS]
- Design head, ex: 2.0 m [6.56 ft] *
- Manhole diameter, ex: ø 900 mm [ø 36”]
- Minimum clearance “H”, ex: 150 mm [6 in]
- Construction material type (304 stainless steel standard)

**The design head is defined as the difference between the maximum upstream water level and the outlet pipe invert.*

Installation

The installation of a HYDROVEX® VHV/SVHV flow regulator can be accomplished quickly and does not require any special tools. The sleeve of the vortex flow regulator is simply inserted into the outlet pipe of the manhole and the unit is then secured to the concrete wall using the supplied anchor.

Maintenance

HYDROVEX® regulators are designed to minimize maintenance requirements. We recommend a periodic visual inspection in order to ensure that the unit is free of debris. The manhole sump beneath the unit should be inspected and cleaned with a vacuum truck periodically to remove accumulated sediments.

Guaranty

The HYDROVEX® line of VHV / SVHV regulators are guaranteed against both design and manufacturing defects for a period of 5 years after sale. The unit will be modified or replaced should it be found to be defective within the guarantee period.

Resourcing the world

Veolia Water Technologies

4105 Sartelon • Saint-Laurent, Quebec • H4S 2B3 Canada

T.: 514-334-7230 • F.: 514-334-5070 • Sales Direct Line: 1-855-564-3747

cso@veolia.com • www.veoliawatertechnologies.ca

APPENDIX 3

Site Servicing Calculations & Correspondence

W/WW Demand Calculations **320 McRae Avenue**

Table 4.1 Per Unit Populations	
Unit Type	**Persons Per Unit**
Townhouse (row)	2.7
Apartments:	
Bachelor	1.4
1 Bedroom	1.4
2 Bedroom	2.1
3 Bedroom	3.1
Avg Apt.	1.8

WATER

Demand Type	Average Day Dema	P.F. (Max Day Demand)	P.F. (Max Hour Demand)	Units
Residential	350	2.5	2.2	L/c/d
Shopping Center	25000	1.5	1.8	L/gross ha/d

WASTEWATER

Demand Type	Average Day Dema	P.F. (Max. Daily Demand)	Units
Residential	280	See Table Below	L/c/d
Shopping Center	28000	1	L/gross ha/d

Type	Units/ Area (ha)	Population	Peak Factor (P.F.)
Bachelor	58	81.2	
1 Bedroom	186	260.4	
1 Bedroom+	0	0	
2 Bedroom	82	172.2	
2 Bedroom+	0	0	
3 Bedroom	10	31	
Townhouse	11	29.7	
Total Proposed	347	574.5	3.35
Ex Single Family Hom	2	6.8	3.74

Water Wastewater Demand Calculations **320 McRae Ave**

NEW DEVELOPMENT - 320 McRae Ave

			WATER						WASTEWATER			
Type	**Units/ Area (ha)**	**Population**	**Avg. Daily Flow (L/d)**	**Avg. Daily Flow(L/s)**	**Max Daily Flow (L/d)**	**Max Daily Flow (L/s)**	**Max Hourly Flow (L/d)**	**Max Hourly Flow (L/s)**	**Avg. Daily Flow (L/d)**	**Avg. Daily Flow(L/s)**	**Max Daily Flow (L/d)**	**Max Daily Flow (L/s)**
Bachelor	58	81.2	28420	0.33	71050	0.82	156310	1.81	22736	0.26	0.00	0.00
1 Bedroom	186	260.4	91140	1.05	227850	2.64	501270	5.80	72912	0.84	0.00	0.00
1 Bedroom+	0	0	0	0.00	0	0.00	0	0.00	0	0.00	0.00	0.00
2 Bedroom	82	172.2	60270	0.70	150675	1.74	331485	3.84	48216	0.56	0.00	0.00
2 Bedroom+	0	0	0	0.00	0	0.00	0	0.00	0	0.00	0.00	0.00
3 Bedroom	10	31	10850	0.13	27125	0.31	59675	0.69	8680	0.10	0.00	0.00
Townhouse	11	29.7	10395	0.12	25987.5	0.30	57172.5	0.66	8316	0.10	0.00	0.00
Commercial	0.0882		2205	0.03	3307.5	0.04	5953.5	0.07	2469.6	0.03	2469.60	0.03
Total Residential		574.5	**201075**	**2.33**	**502687.5**	**5.82**	**1105912.5**	**12.80**	**160860**	**1.86**	**539516.5754**	**6.24**
Total		574.5	**203280**	**2.35**	**505995**	**5.86**	**1111866**	**12.87**	**163329.6**	**1.89**	**541986.18**	**6.27**

EXISTING - entire site

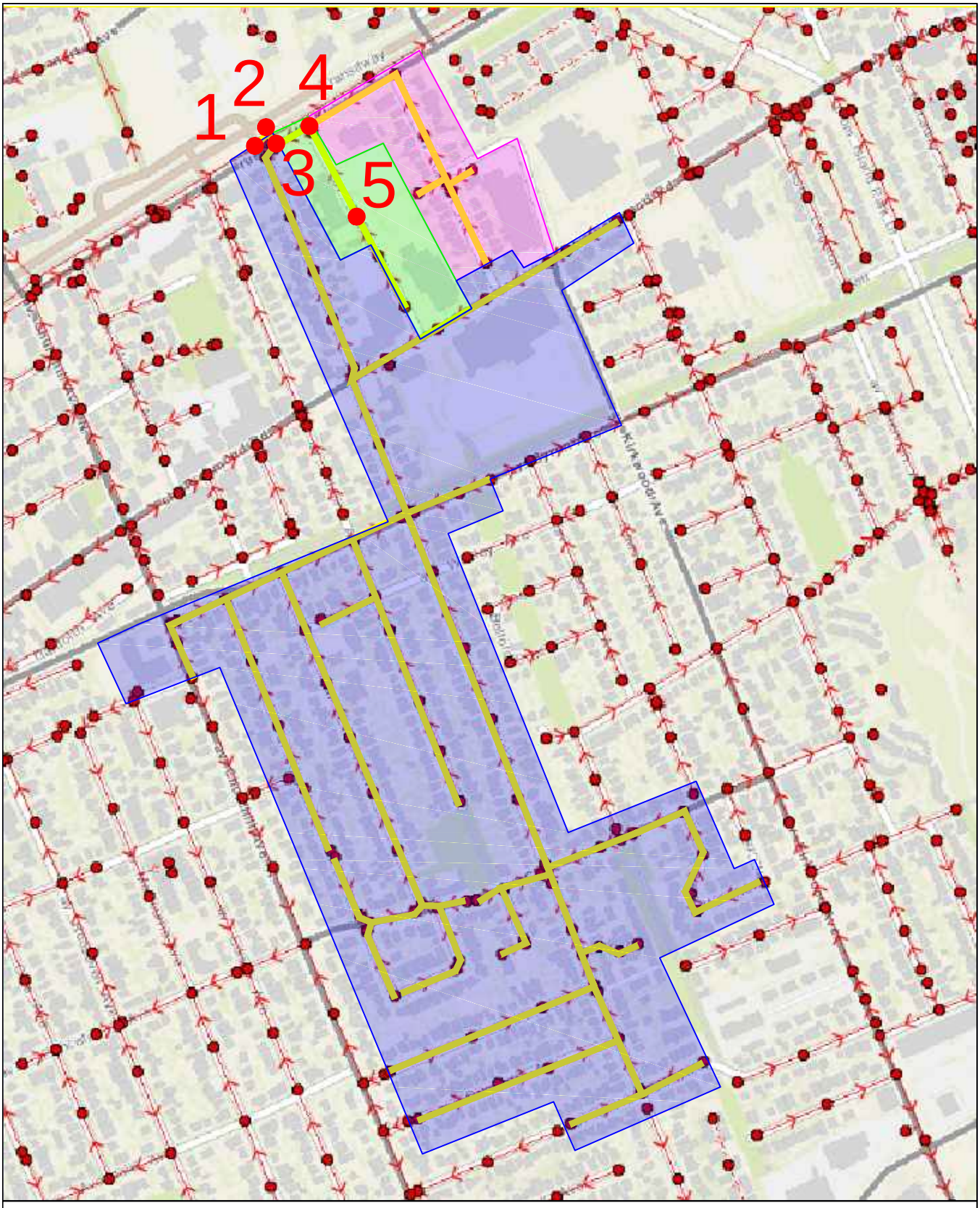
			WATER						WASTEWATER			
Type	**Units/ Area (ha)**	**Population**	**Avg. Daily Flow (L/d)**	**Avg. Daily Flow(L/s)**	**Max Daily Flow (L/d)**	**Max Daily Flow (L/s)**	**Max Hourly Flow (L/d)**	**Max Hourly Flow (L/s)**	**Avg. Daily Flow (L/d)**	**Avg. Daily Flow(L/s)**	**Max Daily Flow (L/d)**	**Max Daily Flow (L/s)**
Single family home	2	6.8	2380	0.03	5950	0.07	13090	0.15	1904	0.02	7127.51	0.08
Commercial	0.117		2925	0.03	4387.5	0.05	7897.5	0.09	3276	0.04	3276.00	0.04
Total		6.8	**5305**	**0.06**	**10337.5**	**0.12**	**20987.5**	**0.24**	**5180**	**0.06**	**10403.51**	**0.12**

320 McRae - Sewer Capacity Analysis Table

Location			Residential Area and Population										Commercial		Institutional		Industrial			Infiltration Flow				Pipe Data				
ID	Up	Down	Area	Number of Units				Population	Cumulative		Peak Factor	Q	Area	Accumulated	Area	Accumulated	Area	Accumulated	QCII	Total Area	Accumulated	Infiltration	Total Flow	Diameter	Slope	Length	Qcap	Q / Qcap
				Singles	Semis	Townhouse	Appartments																					
			ha	ea	ea	ea	ea		ha			L/s	ha	ha	ha	ha	ha	ha	L/s	ha	ha	L/s	L/s	m	m/m	m	L/s	
A	5	4	0.05	1			200	363.4	0.05	363.4	3.433287	4.043332	2.06	2.06		0		0	0.667593	2.11	2.11	0.7385	5.449424	0.25	0.00788	10.2	54.79	0.09946
B	4	3	3.85	30			252	555.6	3.9	919	3.258682	9.705138	0.35	2.41	0.28	0.28	1.29	1.29	4.268461	7.88	9.99	3.4965	17.4701	0.3	0.00434	50.7	66.31	0.263461
C	3	2	33.81	465	14	61	336	2388.3	37.71	3307.3	2.924862	31.34897	5.38	7.79	1.33	1.61	0.14	1.43	6.811632	48.54	58.53	20.4855	58.6461	0.375	1	14.5	182.9	0.320646
D/S	2	1	0					0	37.71	3307.3	2.924862	31.34897		7.79		1.61		1.43	6.811632	0	58.53	20.4855	58.6461	0.375	1	5.9	182.9	0.320646
		Population Per Unit		3.4	2.7	2.7	1.8																					

Design Parameters

Average Daily Residential Flow	280 L/p/d	**Peak Factor Residential**	2 to 4	**Infiltration**	0.35 L/s/ha
Average Daily Commercial Flow	28000 L/ha/d	**Peak Factor Commercial**	1		
Average Daily Institutional Flow	28000 L/ha/d	**Peak Factor Institutional**	1		
Average Daily Industrial Flow	35000 L/ha/d	**Peak Factor Industiral**	App 4-B		



SANITARY SEWER CATCHMENT AREA

CATCHMENT AREA A

CATCHMENT AREA B

CATCHMENT AREA C

SEWER WITHIN CATCHMENT

1 MAINTENANCE HOLE

Fire Flow Analysis - FUS Method
320 McRae Avenue

320 McRAE AVENUE FIRE DEMAND CALCULATIONS

			TOTAL
A	Coefficient for type of construction:		0.6
B	Total Floor Area (excl. basement)	m^2	8,039
C	Height in Stories		26
D	Fire Flow Required	L/min	12,000
E	15% Reduction for Occupancy Charge	L/min	-1,800
	Fire Flow Required	L/min	10,200
F	50% Reduction for Automatic Sprinklers	L/min	-5,100
G	Charge for Building Separation		
	North: Nearest Building	120	5%
	West: Nearest Building	5	20%
	South: Nearest Building	31	5%
	East: Nearest Building	33	5%
	Charge for Building Separation	L/min	3,570
H	Fire Flow Required	L/min	9,000
	Fire Flow Required	L/s	150

Elizabeth Rodgers

From: Wu, John <John.Wu@ottawa.ca>
Sent: February 12, 2020 3:07 PM
To: Elizabeth Rodgers
Subject: RE: 320 McRae- Water Boundary Conditions

Follow Up Flag: Follow up
Flag Status: Flagged

The following are boundary conditions, HGL, for hydraulic analysis at 320 McRae (zone 1W) assumed to be connected to the 203mm on Scott and/or 203mm on McRae (see attached PDF for location).

Minimum HGL = 108.5m

Maximum HGL = 115.5m

MaxDay + FireFlow (150 L/s) = 106.0m, Scott connection

MaxDay + FireFlow (150 L/s) = 103.0m, McRae connection

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.

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: February 11, 2020 5:37 PM
To: Wu, John <John.Wu@ottawa.ca>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: RE: 320 McRae- Water Boundary Conditions

Hi John,

I believe the building will fall under the ISO Construction class 5 (modified fire resistive), so a coefficient of 0.6 would apply for the building. I've redone the FUS calculations accordingly (attached).

Based on updated calculations using the FUS method, the amount of fire flow required is 150 L/s.

Can you please provide updated results from the distribution model using this fire flow?

Thanks,
Beth

From: Wu, John <John.Wu@ottawa.ca>
Sent: February 6, 2020 2:01 PM
To: Elizabeth Rodgers <erodgers@rvanderson.com>
Subject: RE: 320 McRae- Water Boundary Conditions

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

The following are boundary conditions, HGL, for hydraulic analysis at 320 McRae (zone 1W) assumed to be connected to the 203mm on Scott and/or 203mm on McRae (see attached PDF for location).

Minimum HGL = 108.5m

Maximum HGL = 115.5m

MaxDay + FireFlow (317 L/s) = 92.0m, Scott connection

MaxDay + FireFlow (317 L/s) = 83.0m, McRae connection

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

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

Please note, even the fire flow is available for 317, you need more than two fire hydrant to get that amount of fire flow.

John

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: February 4, 2020 10:13 AM
To: Wu, John <John.Wu@ottawa.ca>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: RE: 320 McRae- Water Boundary Conditions

Hi John,

According to the structural engineer, the building is not considered fire-resistive construction. Therefore, I've used the non-combustible coefficient of 0.8 and total floor area as per FUS method.
Amount of fire flow required: 317 L/s.

Please review and provide boundary conditions.

Beth Rodgers (Hamley), P.Eng.
Associate, Project Engineer

R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1284 x 3226
website | **facebook** | **twitter** | **linkedin**

Please consider the environment before printing this email.

Note that my email address has recently changed to ERodgers@rvanderson.com
Please update your records accordingly.

From: Wu, John <John.Wu@ottawa.ca>
Sent: January 28, 2020 10:49 AM
To: Elizabeth Rodgers <erodgers@rvanderson.com>
Subject: RE: 320 McRae- Water Boundary Conditions

Go read the area in FUS 1999, use 0.6 for the construction method. Do not use the whole area of the building.
The occupancy can be 15%.

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: January 28, 2020 10:45 AM
To: Wu, John <John.Wu@ottawa.ca>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: RE: 320 McRae- Water Boundary Conditions

Hi John,
Please find attached for the FUS calculation sheet for 320 McRae, for your review.

Thanks,
Beth Rodgers (Hamley), P.Eng.
Associate, Project Engineer

R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1284 x 3226
website | **facebook** | **twitter** | **linkedin**

Note that my email address has recently changed to ERodgers@rvanderson.com
Please update your records accordingly.

From: Wu, John <John.Wu@ottawa.ca>
Sent: January 28, 2020 10:37 AM
To: Elizabeth Rodgers <erodgers@rvanderson.com>
Subject: RE: 320 McRae- Water Boundary Conditions

Your fire flow it too large, please send us the FUS calculation sheet for review.

From: Elizabeth Rodgers <erodgers@rvanderson.com>
Sent: January 27, 2020 4:48 PM
To: Wu, John <John.Wu@ottawa.ca>
Cc: Nathaniel Rodgers <nrodgers@rvanderson.com>
Subject: 320 McRae- Water Boundary Conditions

Hi John,

For the private development project located at 320 McRae Ave, I would like to request water boundary conditions. If you are not the appropriate contact, please forward this to the appropriate person
.
Can you please provide water boundary conditions for this area, based on the information below?

1. Type of development – Residential condo building with commercial/retail space on the main floor.
2. Location of service – See attached figures of location. Water service to be from either McRae Ave or Scott Street (between McRae and Tweedsmuir).
3. Amount of fire flow required: 367 L/s (calculated per FUS method)
4. Average daily demand: 2.16 L/s

5. Maximum daily demand: 5.38 L/s.
6. Maximum hourly daily demand: 11.83 L/s

Attachment 1 – Draft Site Plan
Attachment 2 – General Location Plan

Please let me know if you need more information.

Thanks,
Beth Rodgers (Hamley), P.Eng.
Associate, Project Engineer

R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1284 x 3226
website | **facebook** | **twitter** | **linkedin**

Please consider the environment before printing this email.

Note that my email address has recently changed to ERodgers@rvanderson.com
Please update your records accordingly.

Nathaniel Rodgers

From: Wes Tabaczuk <wtabaczuk@Pinchin.com>
Sent: Wednesday, August 5, 2020 12:07 PM
To: Nathaniel Rodgers; Jalali, Farzi; Christine Wilson
Cc: Scott Mather; Burke, Ashley; Hanna, Andrew; Devon Heard; Hugo Gagnon; Jaime Posen; Elizabeth Rodgers
Subject: RE: 320 McRae - SPA Comments

Good Afternoon,

The long term steady state rate that is required to keep the entire foundation area dewatered under prolonged pumping is 65,000 L/day.

Let me know if you have any other questions.

Thanks,

Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services
Pinchin Ltd. | T: 613.592.3387 ext. 1829 | C:613.853.2211

From: Nathaniel Rodgers <nrodgers@rvanderson.com>
Sent: Wednesday, July 29, 2020 12:07 PM
To: Wes Tabaczuk <wtabaczuk@Pinchin.com>; Jalali, Farzi <Farzaneh.Jalali@gwlra.com>; Christine Wilson <cwilson@Pinchin.com>
Cc: Scott Mather <smather@Pinchin.com>; Burke, Ashley <Ashley.Burke@gwlra.com>; Hanna, Andrew <andrew.hanna@gwlra.com>; Devon Heard <dheard@neufarchitectes.com>; Hugo Gagnon <hgagnon@neufarchitectes.com>; Jaime Posen <posen@fotenn.com>; Elizabeth Rodgers <erodgers@rvanderson.com>
Subject: RE: 320 McRae - SPA Comments

Hi All,

Please note that the post construction flow rates per #17 below are required to address the comment #18 for the site servicing report. Thus this flow rate will need to be quantified before we will be able to address the comments to resubmit for SPA. This is required for the sewer capacity calculations that the City requested. Please provide when available so we can complete our response to the SPA comments.

Regards,

Qdwh#Jrgjhuv/#SIHqj#
Dvvrfldwh/#Surmfw#Pdqdjhu#

S=+946,#559045;7#n{w#6557##
#
UIY1#Dqghuvrq#Dvvrfldwhv#Dlp lwhg##

APPENDIX 4

Geotechnical and Environmental Reports and Correspondence

FINAL

Geotechnical Investigation – Proposed Commercial/Residential Development

320 McRae Avenue, 1976 Scott Street, and 311 & 315 Tweedsmuir Avenue
Ottawa, Ontario

Prepared for:

1213763 Ontario Inc.
33 Yonge Street, Suite 1000
Toronto, ON M5E 1G4

Attn: Mr. Andrew Hanna

April 29, 2020

Pinchin File: 230236.004

Issued to:	**1213763 Ontario Inc.**
Contact:	**Mr. Andrew Hanna**
Issued on:	**April 29, 2020**
Pinchin file:	**230236.004**
Issuing Office:	**1 Hines Road, Suite 200, Kanata, ON K2K 3C7**
Primary Contact:	**Wesley Tabaczuk, P.Eng.** **Project Manager**

Author: Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services
613.592.3387 ext.1829
wtabaczuk@pinchin.com

Reviewer: Vanessa Marshall, M.Eng., P. Eng.
National Practice Leader, Geotechnical Services
519.746.4210 ext. 3756
vmarshall@pinchin.com

Reviewer: Robert Daciw, B.Sc., P.Geo.
Director, Real Estate Due Diligence
905.363.1460
rdaciw@pinchin.com

TABLE OF CONTENTS

FIGURES

APPENDICES

1.0 INTRODUCTION

Pinchin Ltd. (Pinchin) was retained by 1213763 Ontario Inc. (Client) to conduct a Geotechnical Investigation and provide subsequent geotechnical design recommendations for the proposed commercial/residential development to be located at 320 McRae Avenue, 1976 Scott Street, and 311 & 315 Tweedsmuir Avenue Ottawa, Ontario (Site). The Site location is shown on Figure 1.

The Client provided proposed future development plans for the Site, which include a 25-storey mixed-use commercial and residential building located on the north portion of the Site, and a four-storey commercial and residential building located on the south portion of the Site. The proposed development will also reportedly include a two-level underground parking garage (UPG) which will be located beneath both buildings as well as the remainder of the Site footprint. At the time of this report the depth to the underside of the footings for the UPGs is unknown; as such, for the purpose of this report, Pinchin has assumed an approximate depth of 4.0 metres below the existing ground surface (mbgs) per level of UPG. Therefore, the depths to the underside of the footings for the two level UPG are approximately 8 mbgs.

Pinchin's geotechnical comments and recommendations are based on the results of the Geotechnical Investigation and our understanding of the project scope.

The purpose of the Geotechnical Investigation was to delineate the subsurface conditions and soil engineering characteristics by advancing a total of nine sampled boreholes (Boreholes BH4 to BH12) at the Site. The information gathered from the Geotechnical Investigation will allow Pinchin to provide geotechnical design recommendations for the proposed development. It is noted that Pinchin completed a Phase II Environmental Site Assessment (ESA) in conjunction with the geotechnical field investigation; as such, the information obtained from the Phase II ESA was also used to aid in providing geotechnical design recommendations. A copy of the Phase II ESA monitoring well logs are included in Appendix II.

Based on a desk top review and the results of the Geotechnical Investigation, the following geotechnical data and engineering design recommendations are provided herein:

- A review of relevant area geology and Site background information;
- A detailed description of the observed soil, bedrock and groundwater conditions;
- Site preparation;
- Site service trench design;
- Open cut excavations;
- Anticipated groundwater management;

- Foundation design recommendations including bedrock bearing resistances at Ultimate Limit States (ULS) design;
- Potential total and differential settlements;
- Foundation frost protection and engineered fill specifications and installation;
- Seismic Site classification for seismic Site response;
- Underground parking garage design recommendations;
- Interior concrete floor slab-on-grade (including modulus of subgrade reaction); and
- Asphaltic concrete pavement structure design for parking areas and access roadways.

Abbreviations terminology and principle symbols commonly used throughout the report are enclosed in Appendix I.

2.0 SITE DESCRIPTION AND GEOLOGICAL SETTING

The Site is an L-shaped property which is bounded by Scott Street to the north, McRae Avenue to the east, Tweedsmuir Avenue to the northwest, single family residential dwellings to the southwest, and an asphalt surfaced parking area to the south. The Site is currently developed with a combination of single family residential dwellings, and a single storey multi-tenant commercial building. The Site is also complete with a combination of gravel and asphalt surfaced parking areas as well as areas of soft landscaping (i.e. grassed areas with trees).

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on sandy silt to silty sand textured till on Paleozoic terrain. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey Map 1972, published 1978).

3.0 GEOTECHNICAL FIELD INVESTIGATION AND METHODOLOGY

Pinchin completed a field investigation at the Site on November 2, 9, and 12, 2018 by advancing a total of nine sampled boreholes (Boreholes BH4 to BH12) throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 0.8 to 3.2 mbgs, where refusal was encountered on bedrock. In addition, a 3.0 m and a 19.8 m long bedrock core with NQ sized diamond bit core barrel were advanced at the base of Boreholes BH4 and BH12, respectively, to confirm the presence of bedrock and to evaluate the Rock Quality Designation (RQD). The approximate spatial locations of the boreholes advanced at the Site are shown on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater observations and measurements recorded are included on the appended borehole logs.

The bedrock cores were advanced in accordance with ASTM D2113. The bedrock types and RQD’s were evaluated immediately upon core retrieval.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples and rock cores as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

At the request of the Client, Pinchin retained the services of Geophysics GPR International Inc. (Geophysics GPR) to complete one shear wave velocity sounding at the Site in January 2020. The purpose of the shear wave velocity sounding was to determine Seismic Site Classification for the Site.

The field logging of the soil and groundwater conditions was performed to collect geotechnical engineering design information. The borehole logs include textural descriptions of the subsoil in accordance with a modified Unified Soil Classification System (USCS) and indicate the soil boundaries inferred from non-continuous sampling and observations made during the borehole advancement. These boundaries reflect approximate transition zones for the purpose of geotechnical design and should not be interpreted as exact planes of geological change. The modified USCS classification is explained in further detail in Appendix I. Details of the soil and groundwater conditions encountered within the boreholes are included on the Borehole Logs within Appendix II.

Select soil samples collected from the boreholes were submitted to a material testing laboratory to determine the grain size distribution of the soil, the results of which are provided in Appendix III. In addition, the collected samples were compared against previous geotechnical information from the area, for consistency and calibration of results.

4.0 SUBSURFACE CONDITIONS

4.1 Borehole Soil Stratigraphy and Bedrock Lithology

In general, the soil stratigraphy at the Site consists of either surficial organics, surficial asphalt, or granular fill material overlying bedrock to the maximum borehole refusal depth of approximately 3.2 mbgs. The surficial organic material is typically located on the northwest portion of the Site and was measured to be approximately 200 mm thick. The surficial asphalt is located in the various parking areas and driveways and was measured to range in thickness from 50 to 100 mm.

The granular fill material was encountered within all boreholes either at the surface or underlying the surficial organics and surficial asphalt materials. The granular fill material extended to the underlying bedrock surface at each location and was noted to range in soil matrix from gravelly sand containing trace to some silt to gravelly, silty sand. It is noted that trace brick pieces, trace glass, and bedrock fragments were encountered within the fill. The granular fill material was observed to typically range in thickness from approximately 0.8 to 1.7 m with the exception of the Borehole BH9 which was measured to be approximately 3.2 m thick. Based on uncorrected SPT “N” values of between 1 and 50 blows per 300 mm penetration of a split spoon sampler, the granular fill material had a variable very loose to dense relative density; however, with the exception of isolated pockets within select boreholes, the granular fill generally had a compact to dense relative density. The results of three particle size distribution analyses performed on samples of the fill material indicate that the samples contain 23 to 34% gravel, 40 to 58% sand, and 19 to 26% silt sized particles.

The bedrock cores recovered consisted of limestone rock, which was slightly weathered in the upper layers and transitioned to fresh in the deeper rock core. The bedrock was grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. The bedrock at the fracture locations was mostly sharp and angular, which indicates minor water migration. Natural fractures were closely to moderately spaced, and were generally found to occur in sets oriented at approximately 45 to 90º to the core axis. An approximate 20% wash return within the rock cores was observed. The wash return was grey to milky white in colour. The rock core recovery ranged from 60 to 100%, with an average RQD of 33% in the upper 3.0 m, and an average RQD of 84% below approximately 3.0 m. Based on the RQDs obtained, the bedrock is considered to be weathered and poor quality in the upper 3.0 m and unweathered and good quality below the upper 3.0 m.

4.2 Groundwater Conditions

Groundwater observations and measurements were obtained in the open boreholes at the completion of drilling and are summarized on the appended borehole logs. Groundwater was not observed within the boreholes advanced at the Site; however, groundwater measurements were obtained from the

groundwater monitoring wells which were installed as part of Pinchin's Phase II ESA. Groundwater was measured on November 13, 2018 at depths ranging from 4.4 to 6.1 mbgs. Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions.

5.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

5.1 General Information

The recommendations presented in the following sections of this report are based on the information available regarding the proposed construction, the results obtained from the geotechnical investigation, and Pinchin's experience with similar projects. Since the investigation only represents a portion of the subsurface conditions, it is possible that conditions may be encountered during construction that are substantially different than those encountered during the investigation. If these situations are encountered, adjustments to the design may be necessary. A qualified geotechnical engineer should be on-Site during the foundation preparation to ensure the subsurface conditions are the same/similar to what was observed during the investigation.

It is Pinchin's understanding that the proposed development is to consist of a 25-storey mixed-use commercial and residential building located on the north portion of the Site, and a four-storey commercial and residential building located on the south portion of the Site. The proposed development will also reportedly include a two-level underground parking garage (UPG) which will be located beneath both buildings as well as the remainder of the Site footprint. At this time the depth to the underside of the footings for the UPGs is unknown; as such, for the purpose of this report, Pinchin has assumed an approximate depth of 4.0 metres below the existing ground surface (mbgs) per level of UPG. Therefore, the depths to the underside of the footings for the two level UPG are approximately 8 mbgs

5.2 Site Preparation

Prior to Site preparation activities commencing, the existing building structures will need to be demolished and removed from the Site, including all foundations and service pipes.

Preparation of the Site for the proposed development will consist of removing all trees, vegetation, surficial and overburden materials down to the underlying bedrock surface. The existing inorganic granular fill material may be used to raise grades below soft landscaping areas only.

Prior to placing any fill material at the Site, the bedrock and/or subgrade soil should be inspected by a qualified geotechnical engineer, and loosened/soft pockets should be sub excavated. All fill material is to be installed in maximum 200 mm thick loose lifts, compacted to 98% of its Standard Proctor Maximum Dry Density (SPMDD), within plus 2 to minus 4 of the optimum moisture content.

A qualified geotechnical engineering technician should be on site to observe fill placement operations and perform field density tests at random locations throughout each lift, to indicate the specified compaction is being achieved.

5.3 Open Cut Excavations

It is anticipated that the excavations for the building foundations will extend to a depth of approximately 8.0 mbgs in order to accommodate the proposed levels of underground parking. As such, a portion of the bedrock will need to be removed to accommodate the underground levels.

Based on the subsurface information obtained from within the boreholes it is anticipated that the excavated material will consist of a combination of asphalt, organics, granular fill, bedrock fragments, and bedrock. Groundwater was measured to be located at depths ranging from approximately 4.4 to 6.1 mbgs.

Where workers must enter trench excavations deeper than 1.2 m, the trench excavations should be suitably sloped and/or braced in accordance with the Occupational Health and Safety Act (OHSA), Ontario Regulation 213/91, Construction Projects, July 1, 2011, Part III - Excavations, Section 226. Alternatively, the excavation walls may be supported by either closed shoring, bracing, or trench boxes complying with sections 235 to 239 and 241 under O. Reg. 231/91, s. 234(1). Steel sheet piles are not possible due to the shallow bedrock. The shoring system may be designed as full cantilevers or the lateral loads can be taken up to the installation of internal bracing of rakers or tie back soil anchors. The temporary shoring design must include appropriate factors of safety, and any possible surcharge loading must be taken into account.

Based on the OHSA, the in-situ soil may be classified as Type 3 soil above the groundwater table. Temporary excavations in these soils must be cut at an inclination of 1 horizontal to 1 vertical (H to V) or less from the base of the excavation.

The upper approximate 3.0 m of bedrock in this area is typically weathered and can usually be removed with mechanical equipment, such as a large excavator and hydraulic hammer (hoe ram) and where required, with line drilling on close centres. Often a hydraulic hammer can be utilized to create an initial opening for the excavator bucket to gain access of the layered rock. The bedrock is known to contain vertical joints and near horizontal bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected.

Depending on the ability of the mechanical equipment to advance through the bedrock, drilling and blasting may be required. It is often difficult to blast “neat” lines using conventional drilling and blasting procedures, as such, problems with “over break” are common. This may affect quantities claimed by the contractor for rock excavations, as well as the potential for off-site disposal of the blasted rock, if

necessary. Allowances should be made for over break conditions. Due consideration should also be given to controlled blasting procedures in order to prevent potential damage to the surrounding environment.

In addition, we recommend that a pre-blast survey of all neighbouring properties be undertaken prior to conducting drilling and blasting activities. The preconstruction survey will serve to protect the Client from claims unrelated to the construction activities in the development of this property.

Pinchin notes that, local contractors are familiar with excavating the local bedrock and have specialized knowledge and techniques for its removal. Depending on the block size and degree of weathering of the rock they may have a different approach than what is presented in the preceding paragraphs.

Construction slopes in intact bedrock should stand near vertical provided the “loose” rock is properly scaled off the face. Once the blasting is completed, if there are any permanent bedrock shear walls, they will have to be reviewed by a Rock Mechanics Specialist to determine if it is stable or if it needs reinforcing, such as rock bolting.

In addition to compliance with the OHSA, the excavation procedures must also be in compliance to any potential other regulatory authorities, such as federal and municipal safety standards.

5.4 Anticipated Groundwater Management

Groundwater measurements were obtained from the groundwater monitoring wells which were installed as part of Pinchin’s Phase II ESA. Groundwater was measured on November 13, 2018 at depths ranging from 4.4 to 6.1 mbgs and is located within the bedrock.

Moderate groundwater inflow through the overburden soil and bedrock face is expected where the excavations extend less than 0.50 m below the groundwater table. It is believed that this groundwater inflow can be controlled using a gravity dewatering system with perimeter interceptor ditches and high capacity pumps. For excavations extending more than 0.5 m below the stabilized groundwater table, a dewatering system installed by a specialist dewatering contractor may be required to either lower the groundwater level prior to excavation, or to maintain the groundwater level during construction. The design of the dewatering system should be left to the contractor’s discretion, and the system should meet a performance specification to maintain and control the groundwater at least 0.50 m below the excavation base.

Seasonal variations in the water table should be expected, with higher levels occurring during wet weather conditions in the spring and fall and lower levels occurring during dry weather conditions. If construction commences during wet periods (typically spring or fall), there is a greater potential that the groundwater elevation could be higher and/or perched groundwater may be present. Any potential

precipitation of perched groundwater should be able to be controlled from pumping from filtered sumps, and should be pumped away immediately (not allowed to pond).

Prior to commencing excavations, it is critical that all existing surface water and potential surface water is controlled and diverted away from the Site to prevent infiltration and subgrade softening. At no time should excavations be left open for a period of time that will expose them to precipitation and cause subgrade softening.

All collected water is to discharge a sufficient distance away from the excavation to prevent re-entry. Sediment control measures, such as a silt fence should be installed at the discharge point of the dewatering system. The utmost care should be taken to avoid any potential impacts on the environment.

It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. A Permit to Take Water or a submission to the Environmental Activity and Sector Registry (EASR) would be required if the daily water takings exceed 50,000 L/day. It is the responsibility of the contractor to make this application if required.

5.5 Site Servicing

5.5.1 Pipe Bedding and Cover Materials for Flexible and Rigid Pipes

The subgrade conditions beneath the Site services will consist of bedrock. No support problems are anticipated for flexible or rigid pipes founded on the bedrock. Service pipes require an adequate base to ensure proper pipe connection and positive flow is maintained post construction. As such, pipe bedding should be placed to be of uniform thickness and compactness. The pipe bedding and cover material should conform to OPSD 802.010 and 802.013 specifications for flexible pipes and to OPSD 802.031 to 802.033 with Class ‘B’ bedding for rigid pipes.

For pipes installed within bedrock trenches, the following is recommended:

- Install 300 mm of 19 mm clear stone gravel (OPSS 1004) or Granular ‘A’ (OPSS 1010) below the pipe extending up the sides to the spring line;
- If clear stone is used as bedding material, then a non-woven geotextile (Terrafix 360R or equivalent) is to be placed over the clear stone and pipe extending up vertically along the side walls of the bedrock and pipe a minimum distance of 500 mm;
- The pipe cover material should consist of either a Granular ‘B’ Type I (OPSS 1010) with a maximum particle diameter size of 26.5 mm or bedding sand and should extend to a minimum of 300 mm above the top of the pipe; and

- If rock shatter is present a non-woven geotextile (Terrafix 360R or equivalent) may be required to prevent the migration of fines from the bedding material into the rock shatter. Where blasting is required for site services, over blast of at least 600 mm of rock shatter should be performed. Over blast material may stay in the trench.

All granular fill material is to be placed in maximum 200 mm thick loose lifts compacted to a minimum of 98% SPMDD.

If constant groundwater infiltration becomes an issue, then an approximate 150 mm granular pad consisting of 19 mm clear stone gravel (OPSS 1004) wrapped in a non-woven geotextile (Terrafix 270R or equivalent) should be considered. The clear stone should contain a minimum of 50% crushed particles. Water collected within the stone should be controlled through sumps and filtered pumps.

5.5.2 Trench Backfill

Where the adjacent material consists of bedrock, the trench can be backfilled with well graded blast rock fill, with a gradation similar to OPSS 1010 Granular 'B' Type I. The soil should be placed to the underside of the granular subbase of the pavement structure, and be compacted in maximum 300 mm thick lifts to 98% SPMDD within 4% of the optimum moisture content. This is recommended to provide soil compatibility and help minimize potential abrupt differential frost heave between surrounding natural materials similar in composition.

All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Quality control will be the utmost importance when selecting the material. The selection of the material should be done as early in the contract as possible to allow sufficient time for gradation and proctor testing on representative samples to ensure it meets the projects specifications.

It is anticipated that imported material will be required to backfill the trenches due to minimal amount of natural soil observed at the Site. Imported material should consist of a Granular 'A', Granular 'B' Type I, or Select Subgrade Material (OPSS 1010). Heavy construction equipment and truck traffic should not cross any pipe until at least 1 m of compacted soil is placed above the top of the pipe.

Post compaction settlement of finer grained soil can be expected, even when placed to compaction specifications. As such, fill materials should be installed as far in advance as possible before finishing the roadway in order to mitigate post compaction settlements.

5.5.3 Frost Protection

The frost penetration depth in Ottawa, Ontario is estimated to extend to approximately 2.1 mbgs in open roadways cleared of snow. As such, it is recommended to place water services at a minimum depth of 300 mm below this elevation with the top of the pipe located at 2.4 mbgs or lower as dictated by municipal service requirements. If a minimum of 2.4 m of soil cover cannot be provided, then the pipe should be insulated with a rigid polystyrene insulation (DOW Styrofoam HI40, or equivalent) or a pre-insulated pipe be utilized.

The insulation design configuration may either consist of placing horizontal insulation to a specified design distance beyond the outside edge of the pipe or an inverted "U" surrounding the top and sides of the pipe. Any method chosen requires suitable design and installation in accordance with the manufacture's recommendations. To accommodate the placement of horizontal insulation a wider excavation trench may be required.

5.6 Foundation Design

5.6.1 Discussion

Bedrock was encountered within the boreholes at depths ranging from approximately 0.8 to 3.2 mbgs. As such, based on the anticipated depth to the underside of footing of 8.0 mbgs, Pinchin recommends to construct the building on conventional shallow strip and spread footings founded on the limestone bedrock.

5.6.2 Shallow Foundations Bearing on Bedrock

For conventional shallow strip and spread footings established directly on the weathered bedrock surface encountered approximately 4.0 mbgs, a factored bearing resistance of 750 kPa may be used at Ultimate Limit States (ULS) design. For conventional shallow strip and spread footings established on unweathered competent bedrock, a factored bearing resistance of 2,000 kPa at ULS may be used. Prior to installing foundation formwork, the bedrock is to be reviewed by a geotechnical engineer. SLS does not apply to foundations bearing directly on bedrock, since the loads required for unacceptable settlements to occur would be much larger than the factored ULS and would be limited to the elastic compression of the bedrock and concrete.

The above bearing resistances assume the bedrock is cleaned of all overburden material and any loose rock pieces. In addition, it is assumed that the bedrock is free of soil filled seams. Therefore, the bedrock should be cleaned with air or water pressure exposing clean sound bedrock, and 1.5 m long probe holes should be advanced at selected locations to check for bedrock defects and soil filled seams. In the event soil filled seams are encountered, bedrock may need to be removed to the soil seam in order to achieve the recommended bearing resistances.

If construction proceeds during freezing weather conditions water should not be allowed to pool and freeze in bedrock depressions. All concrete should be installed and maintained above freezing temperatures as required by the concrete supplier.

The bedrock is to be relatively level with slopes not exceeding 10 degrees from the horizontal. Pinchin notes that it may be beneficial to install an approximate 150 mm thick layer of 19 mm clear stone gravel overlying the bedrock surface, to provide the forming contractor with a level working surface. Where the bedrock slope exceeds 10 degrees from the horizontal and does not exceed 25 degrees from the horizontal, shear dowels can be incorporated into the design to resist sliding. Where rock slopes are steeper, the bedrock is to be levelled and stepped as required. The change in vertical height will be a function of the rock quality at the proposed foundation location and will need to be determined at the time of construction.

As an alternative to levelling the bedrock, where the bedrock surface is irregular and jagged, it may be more practical to provide a level benching over these areas by pouring lean mix concrete (minimum 10 MPa) prior to constructing the foundations. This decision is made on Site, since each situation will depend on the Site specific bedrock conditions.

5.6.3 Foundation Transition Zones

Where strip footings are founded at different elevations, the bedrock is to have a maximum slope of 2 H to 1 V, with the concrete footing having a maximum rise of 600 mm and a minimum run of 600 mm between each step, as detailed in the latest edition of the Ontario Building Code (OBC). The lower footing should be installed first to mitigate the risk of undermining the upper footing.

Individual spread footings are to be spaced a minimum distance of one and a half times the largest footing width apart from each other to avoid stress bulb interaction between footings. This assumes the footings are at the same elevation.

5.6.4 Estimated Settlement

All individual spread footings should be founded on bedrock, reviewed and approved by a licensed geotechnical engineer.

Foundations installed in accordance with the recommendations outlined in the preceding sections are not expected to exceed total settlements of 25 mm and differential settlements of 19 mm.

All foundations are to be designed and constructed to the minimum widths as detailed in the latest edition of the OBC.

5.6.5 Building Drainage

To assist in maintaining the building dry from surface water seepage, it is recommended that exterior grades around the buildings be sloped away at a 2% gradient or more, for a distance of at least 2.0 m. Roof drains should discharge a minimum of 1.5 m away from the structure to a drainage swale or appropriate storm drainage system.

It is recommended that exterior perimeter foundation drains be installed where subsurface walls are exposed to the interior (basement walls).

The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. The clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be damp proofed.

5.6.6 Shallow Foundation Frost Protection & Foundation Backfill

In the Ottawa, Ontario area, exterior perimeter foundations for heated buildings require a minimum of 1.8 m of soil cover above the underside of the footing to provide soil cover for frost protection.

It is noted that for foundations established on well-draining bedrock (i.e. no ponding adjacent to the foundation), frost protection is not required. This decision is typically made on Site, since each situation will depend on Site specific bedrock conditions.

Where the foundations for heated buildings do not have the minimum 1.8 m of soil cover frost protection, they should be protected from frost with a combination of soil cover and rigid polystyrene insulation, such as Dow Styrofoam or equivalent product. If required, Pinchin can provide appropriate foundation frost protection recommendations as part of the design review.

To minimize potential frost movements from soil frost adhesion, the perimeter foundation backfill should consist of a free draining granular material, such as a Granular 'B' Type I (OPSS 1010) or an approved sand fill, extending a minimum lateral distance of 600 mm beyond the foundation. The backfill material used against the foundation must be placed so that the allowable lateral capacity is achieved. All granular material is to be placed in maximum 300 mm thick lifts compacted to a minimum of 100% SPMDD in hard landscaping areas and 95% SPMDD in soft landscaping areas. It is recommended that inspection and testing be carried out during construction to confirm backfill quality, thickness and to ensure compaction requirements are achieved.

5.6.7 Site Classification for Seismic Site Response and Soil Behaviour

The following information has been provided to assist the building designer from a geotechnical perspective only. These geotechnical seismic design parameters should be reviewed in detail by the structural engineer and be incorporated into the design as required.

The seismic site classification has been based on the 2012 OBC. The parameters for determination of Site Classification for Seismic Site Response are set out in Table 4.1.8.4.A of the OBC. The site classification is based on the average shear wave velocity in the top 30 m of the site stratigraphy.

Geophysics GPR completed one shear wave velocity sounding at the Site (see Appendix VI). Based on the results of this shear wave velocity sounding, this Site has been classified as Class B; however, a re-calculated Site Class A has been provided for foundations founded directly on competent bedrock. Pinchin notes that as the final foundation design has not been completed, it is recommended that should a Site Class A be used for design purposes, it is clearly stated that the foundations must be founded on competent, unweathered bedrock.

5.7 Underground Parking Garage Design

At this time the final grades for the underside of the underground parking garage footings is unknown. As such, depending on the proposed final grades, there is a potential for the building to have to be designed to either resist hydrostatic uplift or to be provided with underfloor and foundation wall drainage systems connected to a suitable frost-free outlet.

The magnitude of the hydrostatic uplift may be calculated using the following formula:

$$P = \gamma \times d$$

Where:

P = hydrostatic uplift pressure acting on the base of the structure (kPa)

γ = unit weight of water (9.8 kN/m^3)

d = depth of base of structure below the design high water level (m)

Due to the close proximity of the Ottawa River, it is recommended that the 100-year flood level be assumed as the high water level.

The resistance of gross uplift of the structure can be increased by simply increasing the mass of the structure, incorporating oversize footings into the structure or by installing soil/rock anchors.

Alternatively, exterior perimeter foundation drains should be installed where subsurface walls are exposed to the interior. The foundation drains should consist of a minimum 150 mm diameter fabric wrapped perforated drainage tile surrounded by 19 mm diameter clear stone (OPSS 1004) with a minimum cover of 150 mm on top and sides and 50 mm below the drainage tile. Since the natural soil contains a significant amount of silt sized particles, the clear stone gravel should be wrapped in a non-woven geotextile (Terrafix 270R or equivalent). The water collected from the weeping tile should be directed away from the building to appropriate drainage areas; either through gravity flow or interior sump pump systems. All subsurface walls should be waterproofed.

If the proposed basement floor level is constructed close to the stabilized groundwater level, an underfloor drainage system should be installed beneath the slab, in addition to the installation of perimeter weeping tiles at the footing level. The floor slab sub drains should be constructed in a similar fashion to the foundation drains and be connected to a suitable frost free outlet or sump.

If the building is constructed below the groundwater table and utilities sub drains and pumps are used to remove the groundwater from around the building footprint, there is the potential that a Permit to Take Water from the Ministry of the Environment, Conservation and Parks (MECP) will be required for the long term dewatering of the Site.

The walls must also be designed to resist lateral earth pressure. Depending on the design of the building the earth pressure computations must take into account the groundwater level at the Site. For calculating the lateral earth pressure, the coefficient of at-rest earth pressure (K_0) may be assumed at 0.5 for non-cohesive sandy soil. The bulk unit weight of the retained backfill may be taken as 20 kN/m^3 for well compacted soil. An appropriate factor of safety should be applied.

5.7.1 Lower Level Parking Garage Concrete Slab-on-Grade

Prior to the installation of the engineered fill material, all organics and deleterious materials should be removed to the underlying bedrock surface. The underlying bedrock encountered within the boreholes is considered adequate for the support of a concrete slab-on-grade provided it is inspected and approved by an experienced geotechnical engineering consultant.

Based on the in-situ conditions, it is recommended to establish a concrete floor slab-on-grade on a minimum 200 mm thick layer of Granular ’A’ (OPSS 1010). The purpose of the Granular ’A’ is mainly to provide a level surfaced for the concrete formwork. Alternatively, consideration may also be given to using a 200 mm thick layer of uniformly compacted 19 mm clear stone. Any required up fill should consist of a Granular ‘B’ Type I or Type II (OPSS 1010).

The installation of a vapour barrier may be required under the floor slab. If required, the vapour barrier should conform to the flooring manufacturer's and designer's requirements. Consideration may be given to carrying out moisture emission and/or relative humidity testing of the slab to determine the concrete condition prior to flooring installation. To minimize the potential for excess moisture in the floor slab, a concrete mixture with a low water-to-cement ratio (i.e. 0.5 to 0.55) should be used.

The following table provides the unfactored modulus of subgrade reaction values:

Material Type	Modulus of Subgrade Reaction (kN/m^3)
Granular A (OPSS 1010)	85,000
Granular 'B' Type I (OPSS 1010)	75,000
Granular 'B' Type II (OPSS 1010)	85,000

5.8 Asphaltic Concrete Pavement Structure Design

5.8.1 Discussion

Parking areas and access driveways will be constructed adjacent to the proposed buildings. Pinchin presumes that all overburden material will be removed during the construction of the buildings. As such, it is believed that any surficial asphalt pavement structure will be on foundation wall backfill. In areas where the existing fill is not removed due to construction activities, the fill could remain below the pavement structure subject to proof rolling, inspection by a geotechnical engineering and any future settlements accepted by the owner.

At this time Pinchin is unaware of the proposed final grades for the parking lot and access roadways. As such, the following pavement structure is recommended based on the pavement structure overlying granular backfill or the existing fill material.

5.8.2 Pavement Structure

The following table presents the minimum specifications for a flexible asphaltic concrete pavement structure:

Pavement Layer	Compaction Requirements	Light Duty Traffic and Parking Areas	Heavy Duty Traffic Areas and Access Laneways
Surface Course Asphaltic Concrete HL-4 (OPSS 1150)	92% MRD as per OPSS 310	50 mm	50 mm
Binder Course Asphaltic Concrete HL-8 (OPSS 1150)	92% MRD as per OPSS 310	N/A	70 mm
Base Course: Granular "A" (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM-D698)	150 mm	150 mm
Subbase Course: Granular 'B' Type I (OPSS 1010)	100% Standard Proctor Maximum Dry Density (ASTM D698)	300 mm	400 mm

Notes:

i) Any required up fill material below the asphalt concrete pavement structure is to consist of a Granular B Type I (OPSS 1010) installed in maximum 300 mm thick lifts and compacted to 100% SPMDD.

ii) The recommended pavement structure may have to be adjusted according to the City of Ottawa standards.

iii) Performance grade PG 58-34 asphaltic concrete should be specified for Marshall mixes.

5.8.3 Pavement Structure Subgrade Preparation and Granular Up Fill

The proper placement of base and subbase fill materials becomes very important in addressing the proper load distribution to provide a durable pavement structure.

The subgrade should be inspected and approved by a qualified geotechnical engineering consultant prior to placement of the Granular 'B' up fill and/or subbase course.

Samples of both the Granular 'A' and Granular 'B' Type I aggregates should be tested for conformance to OPSS 1010 prior to utilization on Site and during construction. All stockpiled material should be protected from deleterious materials, additional moisture and be kept from freezing.

Post compaction settlement of fine grained soil can be expected, even when placed to compaction specifications. As such, fill material should be installed as far in advance as possible before finishing the parking lot and access roadways for best grade integrity.

Where the subgrade material types differ below the underside of the pavement structure, the transition between the materials should be sloped as per frost heave taper OPSD 205.60.

5.8.4 Drainage

Control of surface water is a critical factor in achieving good pavement structure life. The pavement thickness designs are based on a drained pavement subgrade via sub-drains or ditches. It is recommended that sub drains be installed in the low areas of the on grade parking and be connected to the catch basins.

The surface of the roadways should be free of depressions and be sloped at a minimum grade of 1% in order to drain to appropriate drainage areas. Subgrade soil should slope a minimum of 3% toward stormwater collection points. Positive slopes are very important for the proper performance of the drainage system. The granular base and subbase materials should extend horizontally to any potential ditches or swales.

In addition, routine maintenance of the drainage systems will assist with the longevity of the pavement structure. Ditches, culverts, sewers and catch basins should be regularly cleared of debris and vegetation.

7.0 SITE SUPERVISION & QUALITY CONTROL

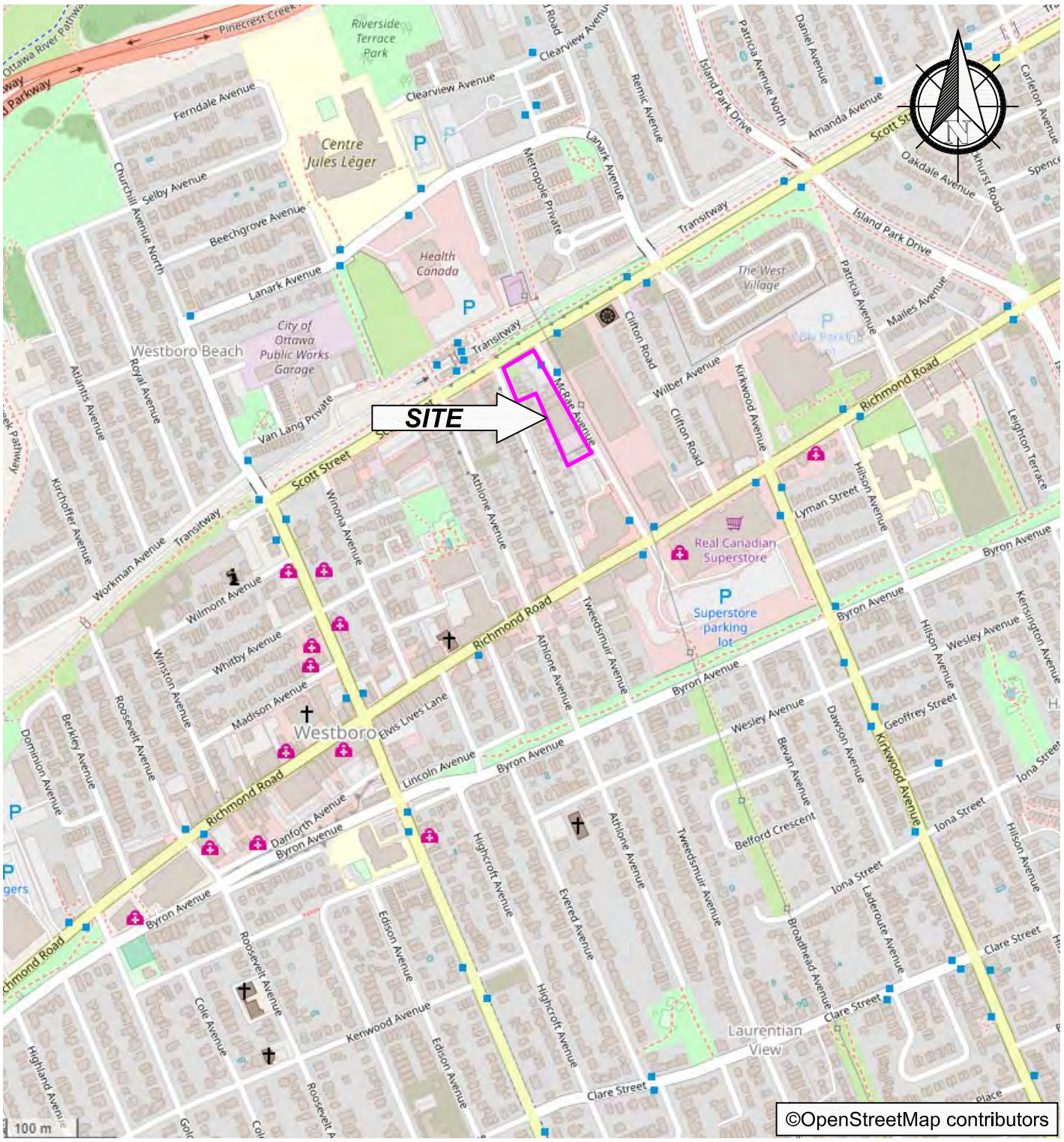
It is recommended that all geotechnical aspects of the project be reviewed and confirmed under the appropriate geotechnical supervision, to routinely check such items. This includes but is not limited to inspection and confirmation of the granular fill and bedrock prior to pouring any foundations or footings, backfilling, or engineered fill installation to ensure that the actual conditions are not markedly different than what was observed at the borehole locations and geotechnical components are constructed as per Pinchin's recommendations. Compaction quality control of engineered fill material (full-time monitoring) is recommended as standard practice, as well as regular sampling and testing of aggregates and concrete, to ensure that physical characteristics of materials for compliance during installation and satisfies all specifications presented within this report.

8.0 DISCLAIMER

This report was prepared pursuant to and in accordance with the master services agreement (the "MSA") dated July 16, 2007 (amended August 8, 2018) between The Pinchin Group of Companies ("Consultant") and the other parties listed thereto, and the project specific agreement dated October 2, 2018, between Consultant and 1213763 Ontario Inc. The report was prepared by Consultant for the use of Owner and Manager (as those terms are defined under the MSA). In addition to the use of and reliance on this report by Owner and Manager, any person who has received a reliance letter for this report may use and rely on this report as if it was prepared for such persons. Any use of or reliance on this report by any other person (i.e., a person other than any Owner, Manager or otherwise permitted person) is the sole and exclusive responsibility of such other person. Consultant accepts no responsibility for damages, if any, suffered by such other person as a result of the use of or reliance on this report.

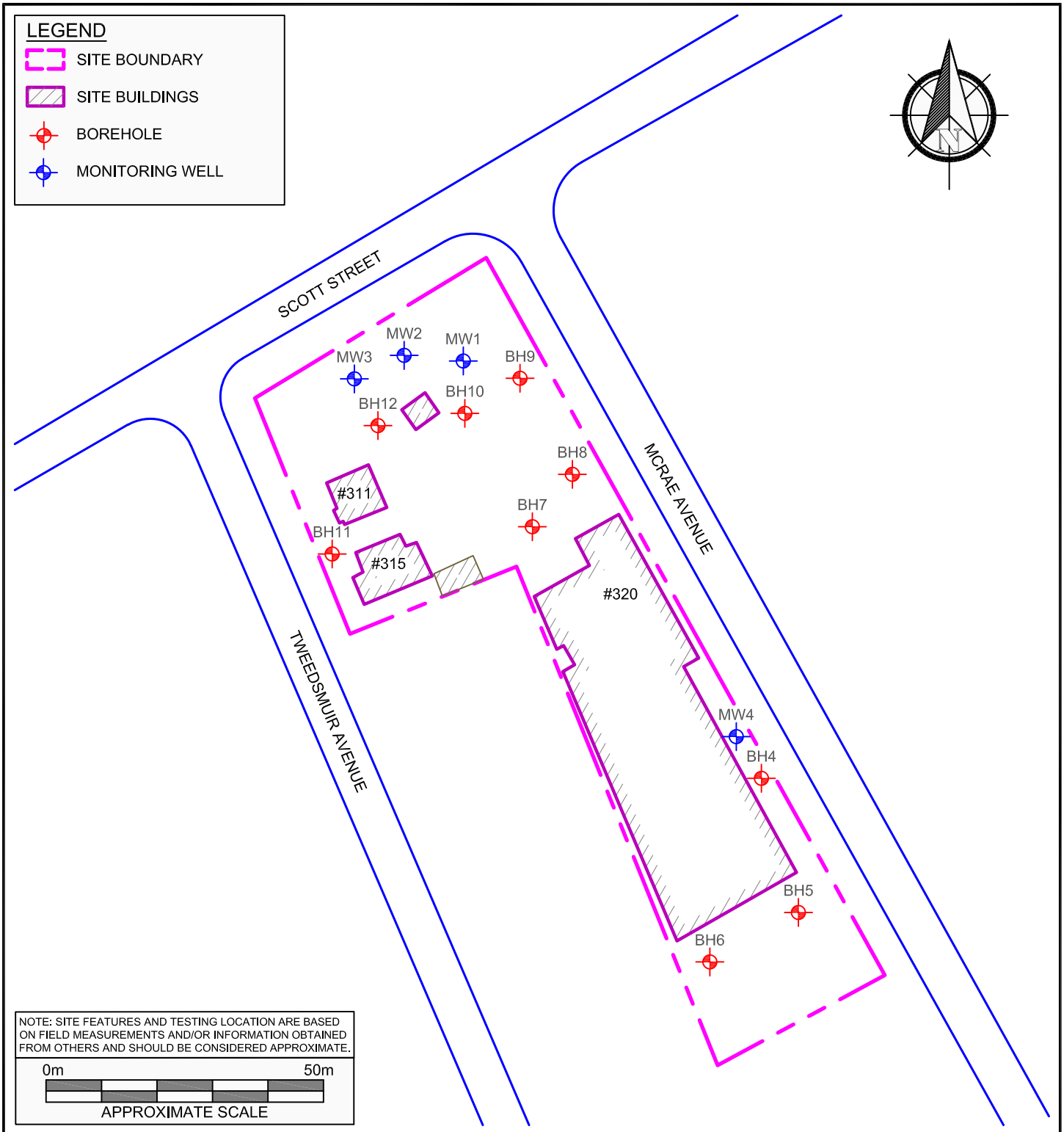
This report is based on the best information available to Consultant at the time of preparing this report after Consultant has used best industry practices, in the circumstances, to obtain information. To the extent that Consultant was required to rely on information from other persons, Consultant has verified such information to the extent reasonably possible in the circumstances. The material provided in this report reflects best industry judgment in light of the information available at the time of preparation of this report.

FIGURES



PINCHIN

PROJECT NAME	GEOTECHNICAL INVESTIGATION	
CLIENT NAME	1213763 ONTARIO INC.	
PROJECT LOCATION	320 MCRAE AVENUE, 1976 SCOTT STREET, 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO	
FIGURE NAME	KEY MAP	FIGURE NO. 1
APPROXIMATE SCALE: AS SHOWN	PROJECT NO.: 230236.004	DATE: APRIL 2020



PINCHIN

PROJECT NAME	GEOTECHNCIAL INVESTIGATION		
CLIENT NAME	1213763 ONTARIO INC.		
PROJECT LOCATION	320 MCRAE AVENUE, 1976 SCOTT STREET, 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO		
FIGURE NAME	BOREHOLE LOCATION PLAN		FIGURE NO. 2
APPROXIMATE SCALE	PROJECT NO.	DATE	
AS SHOWN	230236.004	APRIL 2020	

APPENDIX I

Abbreviations, Terminology and Principle Symbols used in Report and Borehole Logs

ABBREVIATIONS, TERMINOLOGY & PRINCIPAL SYMBOLS USED

Sampling Method

AS	Auger Sample	**w**	Washed Sample
SS	Split Spoon Sample	**HQ**	Rock Core (63.5 mm diam.)
ST	Thin Walled Shelby Tube	**NQ**	Rock Core (47.5 mm diam.)
BS	Block Sample	**BQ**	Rock Core (36.5 mm diam.)

In-Situ Soil Testing

Standard Penetration Test (SPT), "N" value is the number of blows required to drive a 51 mm outside diameter spilt barrel sampler into the soil a distance of 300 mm with a 63.5 kg weight free falling a distance of 760 mm after an initial penetration of 150 mm has been achieved. The SPT, "N" value is a qualitative term used to interpret the compactness condition of cohesionless soils and is used only as a very approximation to estimate the consistency and undrained shear strength of cohesive soils.

Dynamic Cone Penetration Test (DCPT) is the number of blows required to drive a cone with a 60 degree apex attached to "A" size drill rods continuously into the soil for each 300 mm penetration with a 63.5 kg weight free falling a distance of 760 mm.

Cone Penetration Test (CPT) is an electronic cone point with a 10 cm2 base area with a 60 degree apex pushed through the soil at a penetration rate of 2 cm/s.

Field Vane Test (FVT) consists of a vane blade, a set of rods and torque measuring apparatus used to determine the undrained shear strength of cohesive soils.

Soil Descriptions

The soil descriptions and classifications are based on an expanded Unified Soil Classification System (USCS). The USCS classifies soils on the basis of engineering properties. The system divides soils into three major categories; coarse grained, fine grained and highly organic soils. The soil is then subdivided based on either gradation or plasticity characteristics. The classification excludes particles larger than 75 mm. To aid in quantifying material amounts by weight within the respective grain size fractions the following terms have been included to expand the USCS:

Soil Classification		Terminology	Proportion
Clay	< 0.002 mm		
Silt	0.002 to 0.06 mm	“trace”, trace sand, etc.	1 to 10%
Sand	0.075 to 4.75 mm	“some”, some sand, etc.	10 to 20%
Gravel	4.75 to 75 mm	Adjective, sandy, gravelly, etc.	20 to 35%
Cobbles	75 to 200 mm	And, and gravel, and silt, etc.	>35%
Boulders	>200 mm	Noun, Sand, Gravel, Silt, etc.	>35% and main fraction

Notes:

- Soil properties, such as strength, gradation, plasticity, structure, etcetera, dictate the soils engineering behaviour over grain size fractions; and
- With the exception of soil samples tested for grain size distribution or plasticity, all soil samples have been classified based on visual and tactile observations. The accuracy of visual and tactile observation is not sufficient to differentiate between changes in soil classification or precise grain size and is therefore an approximate description.

The following table outlines the qualitative terms used to describe the compactness condition of cohesionless soil:

Cohesionless Soil	
Compactness Condition	**SPT N-Index (blows per 300 mm)**
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	> 50

The following table outlines the qualitative terms used to describe the consistency of cohesive soils related to undrained shear strength and SPT, N-Index:

Cohesive Soil		
Consistency	**Undrained Shear Strength (kPa)**	**SPT N-Index (blows per 300 mm)**
Very Soft	<12	<2
Soft	12 to 25	2 to 4
Firm	25 to 50	4 to 8
Stiff	50 to 100	8 to 15
Very Stiff	100 to 200	15 to 30
Hard	>200	>30

Note: Utilizing the SPT, N-Index value to correlate the consistency and undrained shear strength of cohesive soils is only very approximate and needs to be used with caution.

Soil & Rock Physical Properties

General

W Natural water content or moisture content within soil sample
γ Unit weight
γ' Effective unit weight
γ_d Dry unit weight
γ_{sat} Saturated unit weight
ρ Density
ρ_s Density of solid particles
ρ_w Density of Water
ρ_d Dry density
ρ_{sat} Saturated density e Void ratio
n Porosity
S_r Degree of saturation
E_{50} Strain at 50% maximum stress (cohesive soil)

Consistency

W_L	Liquid limit
W_P	Plastic Limit
I_P	Plasticity Index
W_S	Shrinkage Limit
I_L	Liquidity Index
I_C	Consistency Index
e_{max}	Void ratio in loosest state
e_{min}	Void ratio in densest state
I_D	Density Index (formerly relative density)

Shear Strength

C_u, S_u	Undrained shear strength parameter (total stress)
C'_d	Drained shear strength parameter (effective stress)
r	Remolded shear strength
τ_p	Peak residual shear strength
τ_r	Residual shear strength
ø’	Angle of interface friction, coefficient of friction = tan ø’

Consolidation (One Dimensional)

C_C	Compression index (normally consolidated range)
C_r	Recompression index (over consolidated range)
C_S	Swelling index
m_V	Coefficient of volume change
c_V	Coefficient of consolidation
T_V	Time factor (vertical direction)
U	Degree of consolidation
σ'_O	Overburden pressure
σ'_p	Preconsolidation pressure (most probable)
OCR	Overconsolidation ratio

Permeability

The following table outlines the terms used to describe the degree of permeability of soil and common soil types associated with the permeability rates:

Permeability (k cm/s)	Degree of Permeability	Common Associated Soil Type
$> 10^{-1}$	Very High	Clean gravel
10^{-1} to 10^{-3}	High	Clean sand, Clean sand and gravel
10^{-3} to 10^{-5}	Medium	Fine sand to silty sand
10^{-5} to 10^{-7}	Low	Silt and clayey silt (low plasticity)
$>10^{-7}$	Practically Impermeable	Silty clay (medium to high plasticity)

Rock Coring

Rock Quality Designation (RQD) is an indirect measure of the number of fractures within a rock mass, Deere et al. (1967). It is the sum of sound pieces of rock core equal to or greater than 100 mm recovered from the core run, divided by the total length of the core run, expressed as a percentage. If the core section is broken due to mechanical or handling, the pieces are fitted together and if 100 mm or greater included in the total sum.

RQD is calculated as follows:

$$\text{RQD (\%)} = \frac{\Sigma \text{ Length of core pieces} > 100 \text{ mm} \times 100}{\text{Total length of core run}}$$

The following is the Classification of Rock with Respect to RQD Value:

RQD Classification	RQD Value (%)
Very poor quality	**<25**
Poor quality	25 to 50
Fair quality	50 to 75
Good quality	75 to 90
Excellent quality	90 to 100

APPENDIX II

Pinchin's Borehole Logs

Log of Borehole: MW-1

Project #: 230236.002 **Logged By:** MK

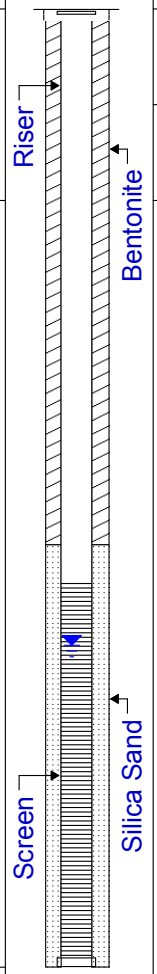
Project: Phase II Environmental Site Assessment

Client: 1213763 Ontario Inc.

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
		Asphalt			30	SS1	0/0	
		Sand and Gravel - Fill Brown, damp.			30	SS2	0/1	Metals, PHCs, PAHs, VOCs, pH
5 / 1.52		***Limestone***	1.52					
25 / 7.62		End of Borehole	7.62					



Water level measured at 5.07 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.29

Top of Casing Elevation: 100.19

Sheet: 1 of 1

Log of Borehole: MW-2

Project #: 230236.002 **Logged By:** MK

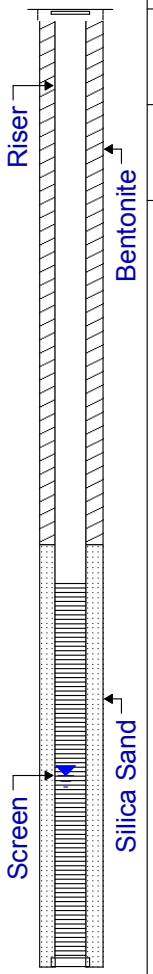
Project: Phase II Environmental Site Assessment

Client: 1213763 Ontario Inc.

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0–5 / 0–1		***Sand and Gravel*** Grey/brown, damp.			30	SS1	0/1	
					30	SS2	0/1	Metals, PHCs, PAHs, VOCs
			1.52					
5–25 / 1–7		***Limestone***						
			7.62					
25 / 8		End of Borehole						



Water level measured at 6.13 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.17

Top of Casing Elevation: 100.28

Sheet: 1 of 1

Log of Borehole: MW-3

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 Ontario Inc.

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0–1 / 0		***Sand and Gravel*** Grey/brown, damp.	0.28	Riser	30	SS1	0/1	PHCs, VOCs, PAHs
3 / 1		Limestone fragments @ 0.75 mbgs		Bentonite				
4–24 / 1–7		***Limestone***		Silica Sand; Screen				
25 / 7.62		End of Borehole	7.62	Well was submerged and frozen on November 13, 2018.				

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: MW-4

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 Ontario Inc.

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					PHCs, VOCs, PAHs, Metals
1		***Sand and Gravel*** With brick fragments, damp.	0.76		50	SS1	0/2	
3 / 1		***Fill*** Sand, brick and glass.	1.07		20	SS2	0/1	
4		***Limestone***		Riser; Bentonite				
2								
3								
4								
5								
6 / 2								
7								
8								
9								
10 / 3								
11								
12								
13 / 4								
14								
15								
16 / 5								
17								
18								
19								
20 / 6				Screen; Silica Sand				
21								
22								
23 / 7								
24								
25			7.62					
26 / 8		End of Borehole		Water level measured at 4.37 mbgs on November 13, 2018.				
27								
28								
29								

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.81

Top of Casing Elevation: 100.70

Sheet: 1 of 1

Log of Borehole: BH4

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20, 40, 60)	Shear Strength kPa (100, 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 50 mm Fill - gravelly silty sand, trace brick, trace glass, trace bedrock fragments, damp, brown, very loose to compact			SS	SS1	50	28						
1					SS	SS2	5	1						
			-1.07											
		Limestone rock, slightly weathered. Grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. Very poor to poor quality			NQ	Run 1	60							RQD=7%
2														
3					NQ	Run 2	70							RQD=40%
4			-4.11											
		End of Borehole												
5														
6														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH5

Project #: 230236.004 ***Logged By:*** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 ***Project Manager:*** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20, 40, 60)	Shear Strength kPa (100, 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, very loose to compact			SS	SS1	50	9						
1					SS	SS2	10	44						
			-1.52											
		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group ***Grade Elevation:*** N/A

Drilling Method: Direct Push/Split Spoon ***Top of Casing Elevation:*** N/A

Well Casing Size: N/A ***Sheet:*** 1 of 1

Log of Borehole: BH6

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20, 40, 60)	Shear Strength kPa (100, 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, compact to dense			SS	SS1	50	17						
1					SS	SS2	80	41						
			-1.52											
		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH7

Project #: 230236.004 ***Logged By:*** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 ***Project Manager:*** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, compact			SS	SS1	50	10						
1					SS	SS2	80	18						
			-1.52											
		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group ***Grade Elevation:*** N/A

Drilling Method: Direct Push/Split Spoon ***Top of Casing Elevation:*** N/A

Well Casing Size: N/A ***Sheet:*** 1 of 1

Log of Borehole: BH8

Project #: 230236.004 ***Logged By:*** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 ***Project Manager:*** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value □ 20 40 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 100 mm Fill - gravelly silty sand, trace brick, trace bedrock fragments, damp, brown, compact			SS	SS1	50	13	□					
			-0.91											
1		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group ***Grade Elevation:*** N/A

Drilling Method: Direct Push/Split Spoon ***Top of Casing Elevation:*** N/A

Well Casing Size: N/A ***Sheet:*** 1 of 1

Log of Borehole: BH9

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20, 40, 60)	Shear Strength kPa (100, 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 100 mm Fill - gravelly sand, trace to some silt, damp, brown, loose to dense			SS	SS1	50	13						
1					SS	SS2	50	28						
2					SS	SS3	60	10						
					SS	SS4	60	9						
3			-3.20		SS	SS5	40	50						
		End of Borehole Due to SPT refusal on bedrock												
4														
5														
6														

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 1

Log of Borehole: BH10

Project #: 230236.004 ***Logged By:*** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 ***Project Manager:*** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (20 40 60)	Shear Strength kPa (100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 100 mm Fill - gravelly silty sand, trace bedrock fragments, damp, brown, loose to dense			SS	SS1	45	9						
1					SS	SS2	80	33						
			-1.68		SS	SS3	40	50						
2		End of Borehole Due to SPT refusal on bedrock												
3														
4														
5														
6														

Contractor: Strata Drilling Group

Grade Elevation: N/A

Drilling Method: Direct Push/Split Spoon

Top of Casing Elevation: N/A

Well Casing Size: N/A

Sheet: 1 of 1

Log of Borehole: BH11

Project #: 230236.004 ***Logged By:*** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 2, 2018 ***Project Manager:*** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value □ 20 40 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Asphalt ~ 50 mm Fill - gravelly silty sand, trace bedrock fragments, damp, brown, dense			SS	SS1	75	35	□					
			-0.76											
1		End of Borehole Due to SPT refusal on bedrock												
2														
3														
4														
5														
6														

Contractor: Strata Drilling Group ***Grade Elevation:*** N/A

Drilling Method: Direct Push/Split Spoon ***Top of Casing Elevation:*** N/A

Well Casing Size: N/A ***Sheet:*** 1 of 1

Log of Borehole: BH12

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 9 and 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (□ 20 40 60 □)	Shear Strength (△ kPa △ 100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Organics ~ 200 mm Fill - gravelly silty sand, trace brick, trace glass, trace bedrock fragments, damp, brown, compact			SS	SS1	60	17						
1			-1.22		SS	SS2	25	12						
		Limestone rock, slightly weathered in the upper layers and fresh in the deeper layers. Grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. Poor Quality			NQ	Run 1	72							RQD=33%
2														
3					NQ	Run 2	88							RQD=51%
4			-4.27											
		Good to excellent quality			NQ	Run 3	92							RQD=82%
5														
6					NQ	Run 4	100							RQD=88%
7														
8					NQ	Run 5	95							RQD=93%
9														
10					NQ	Run 6	98							RQD=93%

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH12

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 9 and 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value □ 20 40 60 □	Shear Strength △ kPa △ 100 200	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
11					NQ	Run 7	100							RQD=85%
12					NQ	Run 8	98							RQD=83%
13					NQ	Run 9	97							RQD=80%
14														
15					NQ	Run10	100							RQD=77%
16			-16.46											
		Very poor quality			NQ	Run11	100							RQD=17%
17			-17.37											
		Fair quality			NQ	Run12	100							RQD=63%
18			-18.90											
		Excellent quality			NQ	Run13	100							RQD=93%
19														
20			-20.42											
		End of Borehole												

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 2 of 2

APPENDIX III

Analytical Laboratory Testing Reports for Soil Samples

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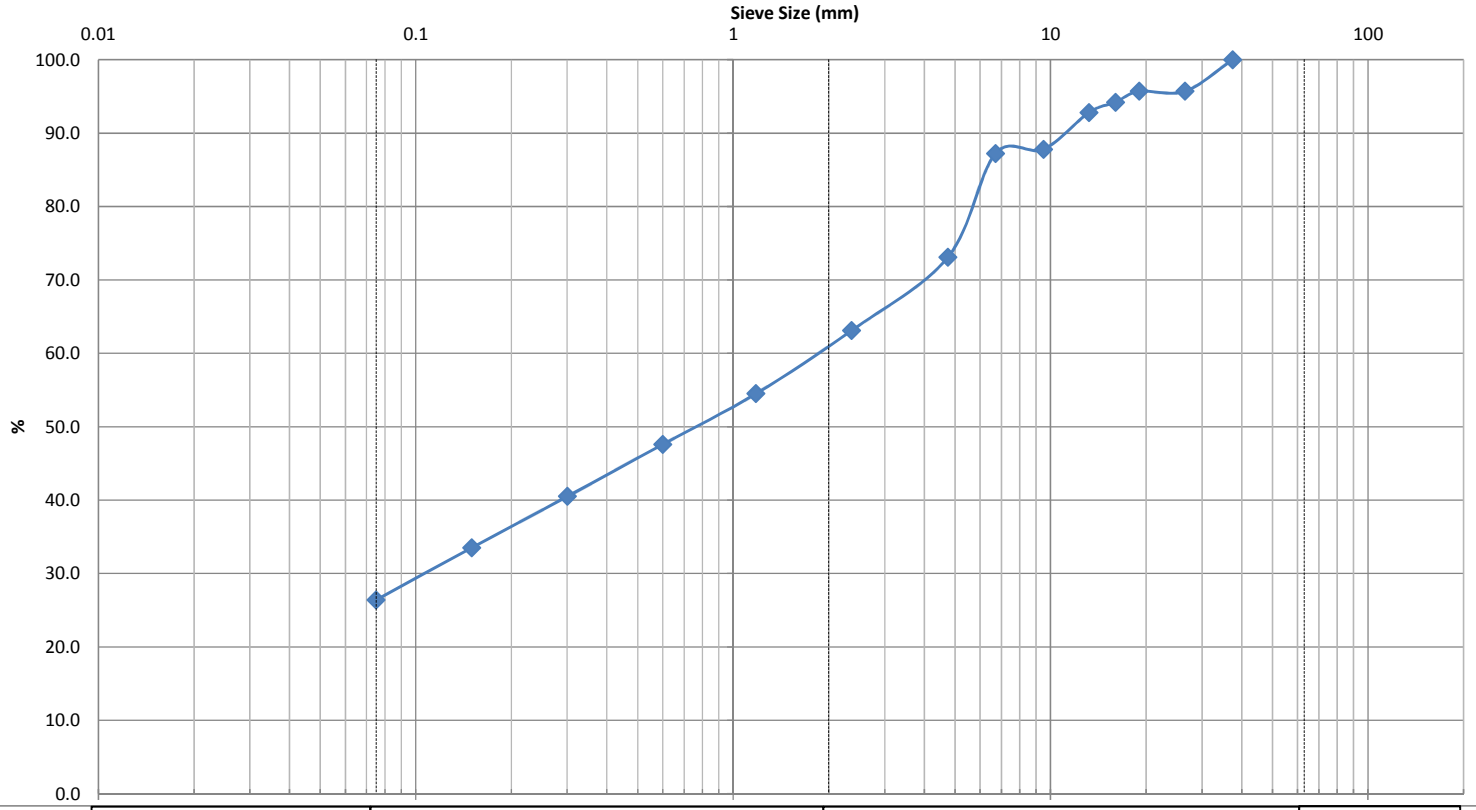
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	06324
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE RECEIVED:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	2-Nov-18	SOURCE LOCATION:	BH6	DATE REPORTED:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
										0.33	92.5
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
	38.5	1.85	0.11	0.02	26.9	46.7		26.4			

Comments	

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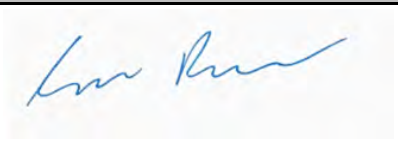
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	06324
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE REC'D:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	02-Nov-18	SOURCE LOCATION:	BH6	DATE REP'D:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B

WEIGHT BEFORE WASH			A+B	934.7
WEIGHT AFTER WASH	A	B	A+B	699.6

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5	0.0	0.0	**100.0**			
26.5	**40.6**	4.3	**95.7**			
19	**40.6**	4.3	**95.7**			
16	**54.2**	5.8	**94.2**			
13.2	**67.1**	7.2	**92.8**			
9.5	**114.0**	12.2	**87.8**			
6.7	**119.7**	12.8	**87.2**			
4.75	**251.4**	26.9	**73.1**			
2.36	**345.1**	36.9	**63.1**			
1.18	**425.3**	45.5	**54.5**			
0.6	**489.9**	52.4	**47.6**			
0.3	**555.7**	59.5	**40.5**			
0.15	**621.4**	66.5	**33.5**			
0.075	**687.5**	73.6	**26.4**			
PAN	**699.6**					
SIEVE CHECK FINE		0.00	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	RESULT	LAB NO.	RESULT

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.
		

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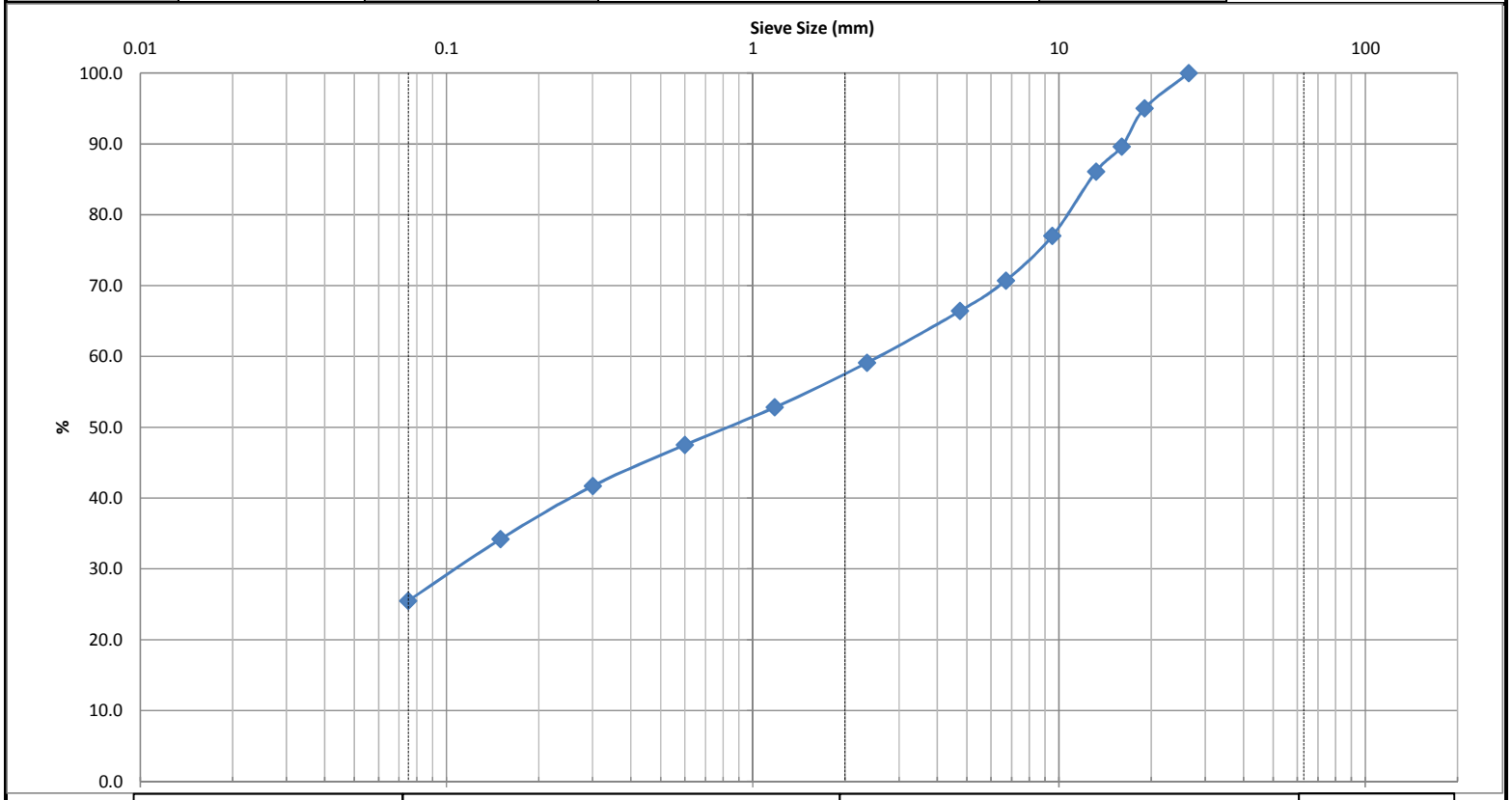
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	06325
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE RECEIVED:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	2-Nov-18	SOURCE LOCATION:	BH10	DATE REPORTED:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification					MC(%)	LL	PL	PI	Cc	Cu
										0.19	104.0
	D100	D60	D30	D10	Gravel (%)	Sand (%)		Silt (%)		Clay (%)	
	26.5	2.6	0.11	0.025	33.6	40.9		25.5			

Comments	

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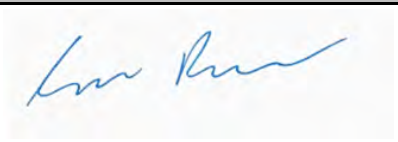
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	06325
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE REC'D:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	02-Nov-18	SOURCE LOCATION:	BH10	DATE REP'D:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	2.5 - 4.5'	TESTED BY:	D.K/D.B

WEIGHT BEFORE WASH			A+B	983.0
WEIGHT AFTER WASH	A	B	A+B	764.8

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5	**0.0**	0.0	**100.0**			
19	**49.3**	5.0	**95.0**			
16	**102.0**	10.4	**89.6**			
13.2	**137.1**	13.9	**86.1**			
9.5	**225.6**	23.0	**77.0**			
6.7	**287.9**	29.3	**70.7**			
4.75	**329.8**	33.6	**66.4**			
2.36	**402.2**	40.9	**59.1**			
1.18	**464.1**	47.2	**52.8**			
0.6	**516.1**	52.5	**47.5**			
0.3	**572.8**	58.3	**41.7**			
0.15	**646.6**	65.8	**34.2**			
0.075	**732.7**	74.5	**25.5**			
PAN	**764.8**					
SIEVE CHECK FINE		0.00	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	RESULT	LAB NO.	RESULT

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.
		

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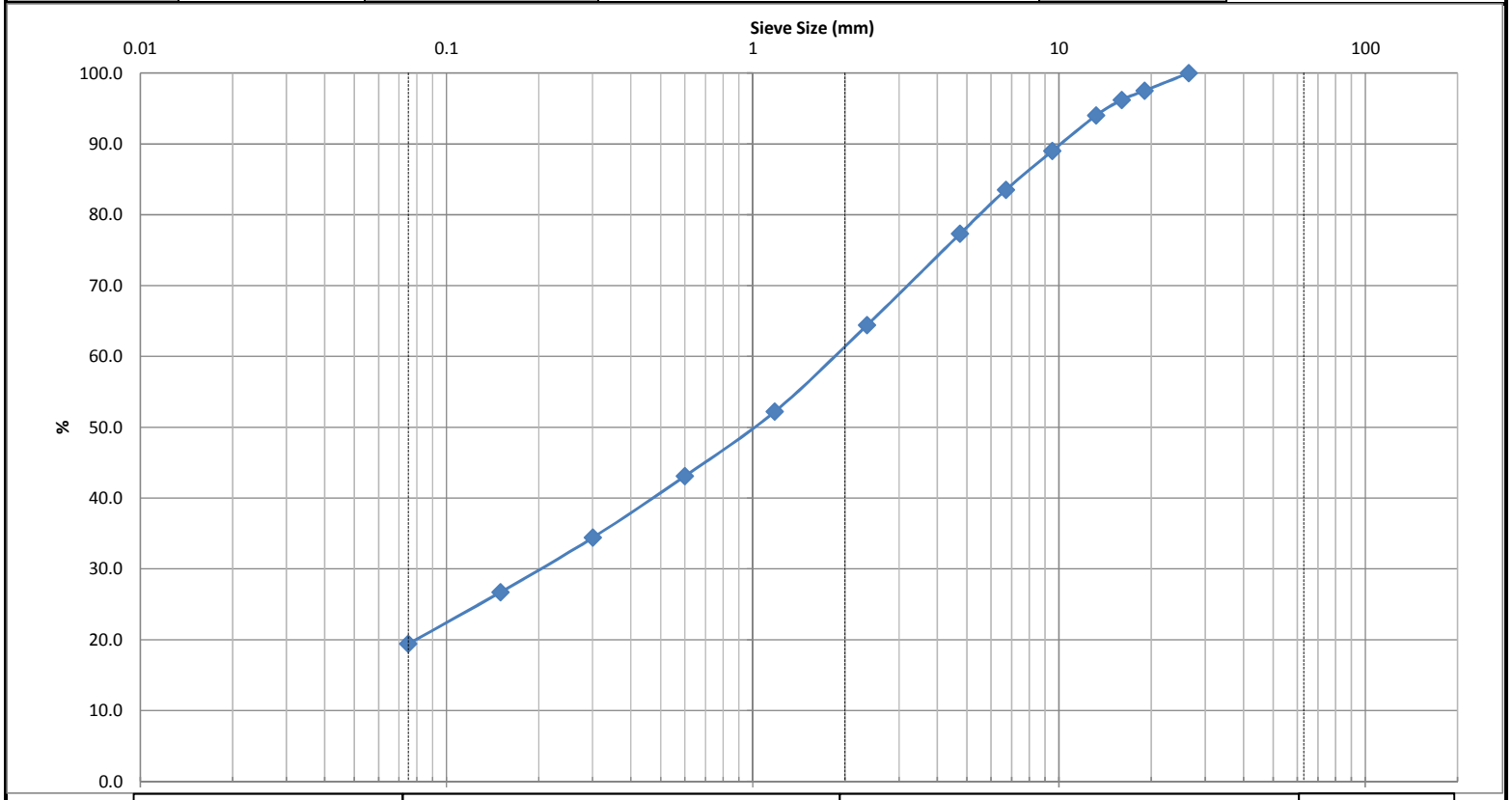
SIEVE ANALYSIS
ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO:	06326
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE RECEIVED:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	2-Nov-18	SOURCE LOCATION:	BH11	DATE REPORTED:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	0 - 2'	TESTED BY:	D.K/D.B



Silt and Clay	Sand			Gravel		Cobble
	Fine	Medium	Coarse	Fine	Coarse	

Identification	Soil Classification							MC(%)	LL	PL	PI	Cc	Cu
												1.05	95.0
	D100	D60	D30	D10	Gravel (%)		Sand (%)			Silt (%)		Clay (%)	
	26.5	1.9	0.2	0.02	22.7		57.9			19.4			

Comments	

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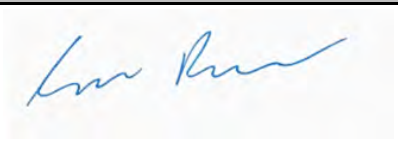
SIEVE ANALYSIS ASTM C136

CLIENT:	Pinchin Environmental	DESCRIPTION:	Silty Sand	FILE NO.:	PM4184
CONTRACT NO.:	-	SPECIFICATION:	-	LAB NO.:	06326
PROJECT:	Laboratory Testing Job # 230236.004	INTENDED USE:	-	DATE REC'D:	19-Nov-18
		PIT OR QUARRY:	-	DATE TESTED:	20-Nov-18
DATE SAMPLED:	02-Nov-18	SOURCE LOCATION:	BH11	DATE REP'D:	22-Nov-18
SAMPLED BY:	Client	SAMPLE LOCATION:	0 - 2'	TESTED BY:	D.K/D.B

WEIGHT BEFORE WASH			A+B	884.6
WEIGHT AFTER WASH	A	B	A+B	728.6

SIEVE SIZE (mm)	WEIGHT RETAINED	PERCENT RETAINED	PERCENT PASSING	LOWER SPEC	UPPER SPEC	REMARK
150						
106						
75						
63						
53						
37.5						
26.5	**0.0**	0.0	**100.0**			
19	**22.3**	2.5	**97.5**			
16	**33.3**	3.8	**96.2**			
13.2	**53.4**	6.0	**94.0**			
9.5	**97.7**	11.0	**89.0**			
6.7	**145.7**	16.5	**83.5**			
4.75	**200.8**	22.7	**77.3**			
2.36	**314.9**	35.6	**64.4**			
1.18	**423.0**	47.8	**52.2**			
0.6	**503.7**	56.9	**43.1**			
0.3	**580.5**	65.6	**34.4**			
0.15	**648.2**	73.3	**26.7**			
0.075	**713.4**	80.6	**19.4**			
PAN	**728.0**					
SIEVE CHECK FINE		0.08	0.3% max.			**REFERENCE MATERIAL**

OTHER TESTS	RESULT	LAB NO.	RESULT

REVIEWED BY:	Curtis Beadow	Joe Forsyth, P. Eng.
		

APPENDIX IV

Report Limitations and Guidelines for Use

REPORT LIMITATIONS & GUIDELINES FOR USE

This information has been provided to help manage risks with respect to the use of this report.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES, PERSONS AND PROJECTS

This report was prepared for the exclusive use of the Client and their authorized agents, subject to the conditions and limitations contained within the duly authorized work plan. Any use which a third party makes of this report, or any reliance on or decisions to be made based on it, are the responsibility of the third parties. If additional parties require reliance on this report, written authorization from Pinchin will be required. Pinchin disclaims responsibility of consequential financial effects on transactions or property values, or requirements for follow-up actions and costs. No other warranties are implied or expressed. Furthermore, this report should not be construed as legal advice.

SUBSURFACE CONDITIONS CAN CHANGE

This geotechnical report is based on the existing conditions at the time the study was performed, and Pinchin's opinion of soil conditions are strictly based on soil samples collected at specific test hole locations. The findings and conclusions of Pinchin's reports may be affected by the passage of time, by manmade events such as construction on or adjacent to the Site, or by natural events such as floods, earthquakes, slope instability or groundwater fluctuations.

LIMITATIONS TO PROFESSIONAL OPINIONS

Interpretations of subsurface conditions are based on field observations from test holes that were spaced to capture a 'representative' snap shot of subsurface conditions. Site exploration identifies subsurface conditions only at points of sampling. Pinchin reviews field and laboratory data and then applies professional judgment to formulate an opinion of subsurface conditions throughout the Site. Actual subsurface conditions may differ, between sampling locations, from those indicated in this report.

LIMITATIONS OF RECOMMENDATIONS

Subsurface soil conditions should be verified by a qualified geotechnical engineer during construction. Pinchin should be notified if any discrepancies to this report or unusual conditions are found during construction.

Sufficient monitoring, testing and consultation should be provided by Pinchin during construction and/or excavation activities, to confirm that the conditions encountered are consistent with those indicated by the test hole investigation, and to provide recommendations for design changes should the conditions revealed during the work differ from those anticipated. In addition, monitoring, testing and consultation by Pinchin should be completed to evaluate whether or not earthwork activities are completed in

accordance with our recommendations. Retaining Pinchin for construction observation for this project is the most effective method of managing the risks associated with unanticipated conditions. However, please be advised that any construction/excavation observations by Pinchin is over and above the mandate of this geotechnical evaluation and therefore, additional fees would apply.

MISINTERPRETATION OF GEOTECHNICAL ENGINEERING REPORT

Misinterpretation of this report by other design team members can result in costly problems. You could lower that risk by having Pinchin confer with appropriate members of the design team after submitting the report. Also retain Pinchin to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering or geologic report. Reduce that risk by having Pinchin participate in pre-bid and preconstruction conferences, and by providing construction observation. Please be advised that retaining Pinchin to participation in any 'other' activities associated with this project is over and above the mandate of this geotechnical investigation and therefore, additional fees would apply.

CONTRACTORS RESPONSIBILITY FOR SITE SAFETY

This geotechnical report is not intended to direct the contractor's procedures, methods, schedule or management of the work Site. The contractor is solely responsible for job Site safety and for managing construction operations to minimize risks to on-Site personnel and to adjacent properties. It is ultimately the contractor's responsibility that the Ontario Occupational Health and Safety Act is adhered to, and Site conditions satisfy all 'other' acts, regulations and/or legislation that may be mandated by federal, provincial and/or municipal authorities.

SUBSURFACE SOIL AND/OR GROUNDWATER CONTAMINATION

This report is geotechnical in nature and was not performed in accordance with any environmental guidelines. As such, any environmental comments are very preliminary in nature and based solely on field observations. Accordingly, the scope of services do not include any interpretations, recommendations, findings, or conclusions regarding the, assessment, prevention or abatement of contaminants, and no conclusions or inferences should be drawn regarding contamination, as they may relate to this project. The term "contamination" includes, but is not limited to, molds, fungi, spores, bacteria, viruses, PCBs, petroleum hydrocarbons, inorganics, pesticides/insecticides, volatile organic compounds, polycyclic aromatic hydrocarbons and/or any of their by-products.

Pinchin will not be responsible for any consequential or indirect damages. Pinchin will only be held liable for damages resulting from the negligence of Pinchin. Pinchin will not be liable for any losses or damage if the Client has failed, within a period of two years following the date upon which the claim is discovered within the meaning of the Limitations Act, 2002 (Ontario), to commence legal proceedings against Pinchin to recover such losses or damage.

APPENDIX V

Rock Core Photographs

Photo 1 – Borehole BH4, Rock Core (Runs 1 and 2)

Photo 2 – Borehole BH12, Rock Core (Runs 1 to 3)

Photo 3 – Borehole BH12, Rock Core (Runs 4 to 6)

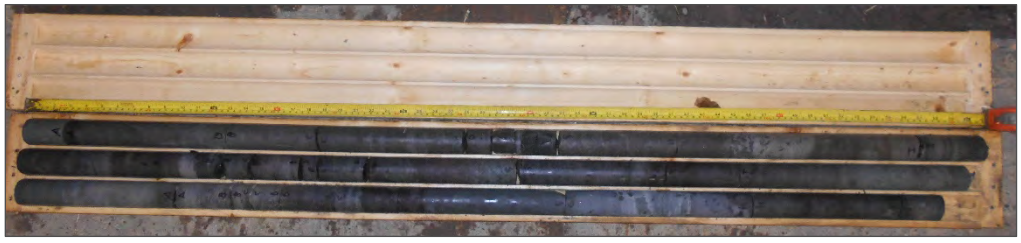


Photo 4 – Borehole BH12, Rock Core (Runs 7 to 9)

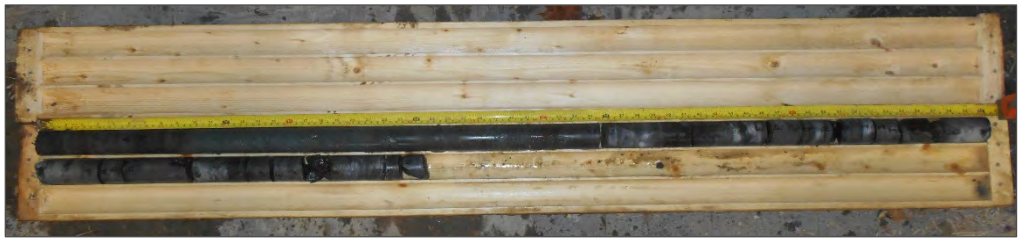


Photo 5 – Borehole BH12, Rock Core (Runs 10 and 11)

Photo 6 – Borehole BH12, Rock Core (Runs 12 and 13)

APPENDIX VI

Geophysics GPR International Inc. Shear-Wave Velocity Sounding

GEOPHYSICS GPR INTERNATIONAL INC.

6741 Columbus Road
Unit 14
Mississauga, Ontario
Canada L5T 2G9

Tel.: (905) 696-0656
Fax: (905) 696-0570
gprtor@gprtor.com
www.geophysicsgpr.com

February 14, 2020 GPR file: T201972

Wesley Tabaczuk, P.Eng.
Project Manager, Geotechnical Services
Pinchin Ltd.
1001 – 555 Legget Drive, Tower A
Kanata, Ontario
K2K 2X3

RE: Shear-wave velocity sounding at 320 McRae Avenue, Ottawa, Ontario

Dear Mr. Tabaczuk:

Geophysics GPR International Inc. has been requested by Pinchin Ltd. to carry out a shear-wave velocity sounding at the above site in Ottawa. Figure 1 shows the location of the test profile.

The survey was performed on February 3rd, 2020.

The investigation included the multi-channel analysis of surface waves (MASW), the micro-tremor array measurements (MAM) and the refraction methods to generate a shear-wave velocity model (Figure 4).

The following paragraphs describe the survey design, the principles of the test method, the methodology for interpreting the data, and provide a culmination of the results in table format.

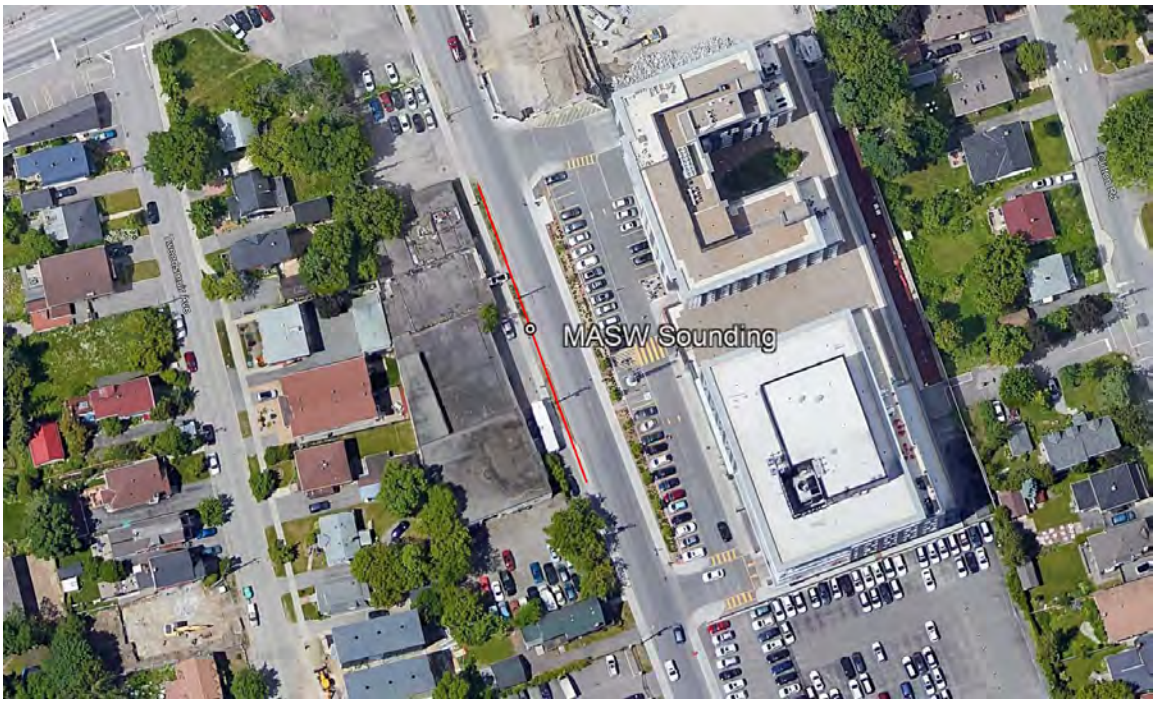


Figure 1: Approximate location of the shear-wave velocity sounding

MASW and MAM Surveys

Basic Theory

The Multi-channel Analysis of Surface Waves (MASW) and the Micro-tremor Array Measurements (MAM) are seismic methods used to evaluate the shear-wave velocities of subsurface materials through the analysis of the dispersion properties of Rayleigh surface waves ("ground roll"). The dispersion properties are measured as a change in phase velocity with frequency. Surface wave energy will decay exponentially with depth. Lower frequency surface waves will travel deeper and thus be more influenced by deeper velocity layering than the shallow higher frequency waves. Inversion of the Rayleigh wave dispersion curve yields a shear-wave (V_s) velocity depth profile (sounding). Figure 2 outlines the basic operating procedure for the MASW method. Figure 3 is an example image of a typical MASW record and resulting 1D V_s model. A more detailed description of the method can be found in the paper *Multi-channel Analysis of Surface Waves,* Park, C.B., Miller, R.D. and Xia, J. Geophysics, Vol. 64, No. 3 (May-June 1999); P. 800–808.

Survey Design

The geometry of an MASW survey is similar to that of a seismic refraction investigation (i.e. 24 geophones in a linear array). The fundamental principle involves intentionally generating an acoustic wave at the surface and digitally recording the surface waves from the moment of source impact with a linear series of geophones on the surface. This is referred to as an "active source" method. An elastic-wave hammer was used as the primary energy source with traces being recorded at 6 locations: approximately 6 m off both ends, 25 to 30 m off both

ends, and in the middle of the spread. Data were collected with geophones spacing of 3m and 1m for a total of 10 shot records per sounding.

Unlike the refraction method, which produces a data point beneath each geophone, the shear-wave depth profile is the average of the bulk area within the middle third of the geophone spread.

The theoretical maximum depth of penetration (34.5m) is half of the maximum seismic array length (69 m), in practice the maximum depth of penetration is often influenced by the geology.

The MAM/passive survey used the same geophone array set up as for the MASW survey. Unlike the MASW survey, the MAM method is considered a "passive source" method in that there is no time break and the motions recorded are from ambient energy generated by cultural noise such as traffic, wind, wave motion, etc. Data collection for the passive method involves recording approximately 10 minutes of background "noise." The records generated by the MAM method contain lower frequency data, thus increasing the data resolution at greater depths of investigation. Typically the MAM results aid in clarifying the MASW results for depths greater than 20 m; however, the direction of noise propagation relative to the spread orientation can influence the results.

Interpretation Method and Accuracy of Results

The main processing sequence involved plotting, picking, and 1-D inversion of the MASW/MAM shot records using the SeisimagerSW™ software package. In theory, all MASW shot records should produce a similar shear-wave velocity profile. In practice, however, differences can arise due to energy dissipation and localized surface variations. The results of the inversion process are inherently non-unique and the final model must be judged to be geologically realistic. The inversion modelling also assumes that all layering is flat/horizontal and laterally uniform.

The results of the MASW/MAM tests are presented in chart format as Figure 4. The chart presents the 1-D shear wave velocity values from the inversion models of the passive and active seismic records.

The V_s30 values for the sounding are presented in Table 1. The V_s30 values are based on the harmonic mean of the shear wave velocities over the upper 30 m. The V_s30 value is calculated by dividing the total depth of interest (e.g. 30 m) by the sum of the time spent in each velocity layer up to that depth. This harmonic mean value reflects the equivalent single layer response.

The estimated error in the average V_s30 value determined through MASW tests is typically +/-10 to 15% for overburden sites. The shear-wave velocities modelled through the MASW method within bedrock have a higher estimated error.

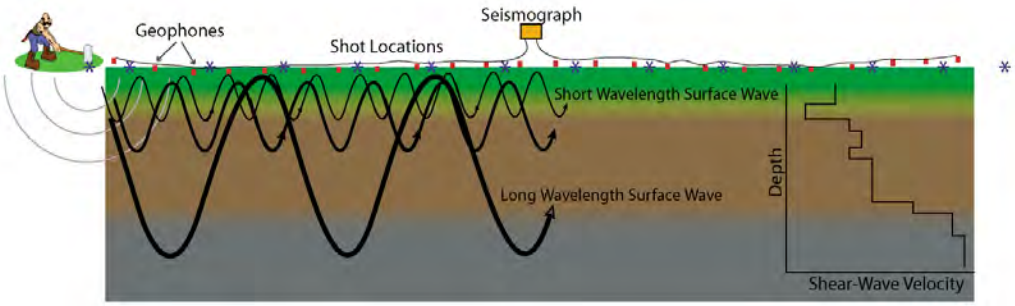


Figure 2: MASW Operating Principle

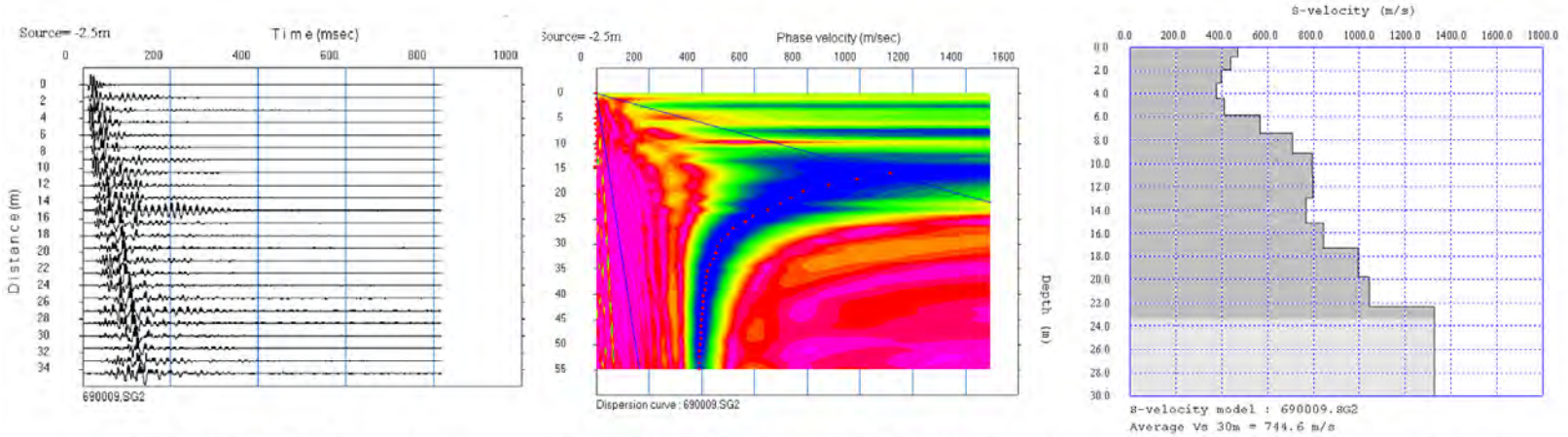


Figure 3: Example of a typical MASW shot record, phase velocity/frequency curve and resulting 1D shear-wave velocity model.

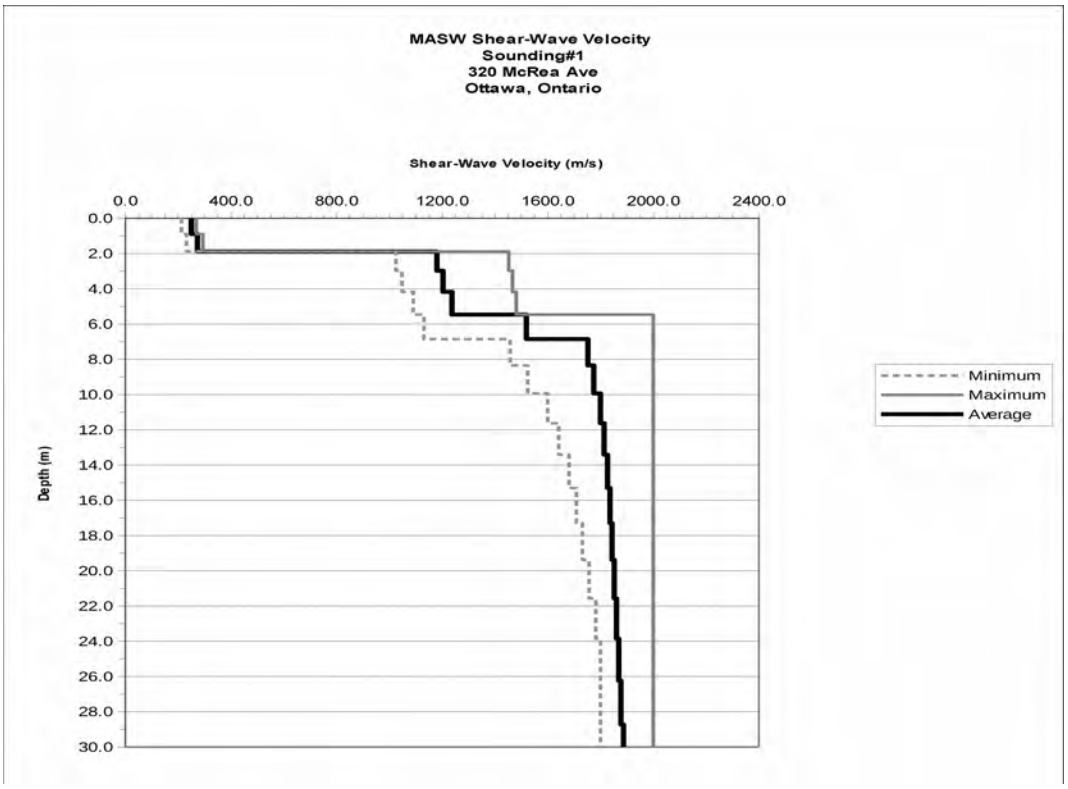


Figure 4: MASW Shear-wave Velocity Sounding

CONCLUSIONS

The approximate location of the shear-wave sounding is indicated in Figure 1.

The MASW shear-wave models are presented in Figure 4. The results are summarized in Table 1. The background seismic noise levels at this site were moderate. The quality of the seismic records and the resulting dispersion were good.

Simple critical distance calculations from refracted P-waves show that bedrock is shallow, in the order of 2m. Refracted P-wave velocities of approximately 4800m/s were measured for the competent bedrock.

The provided boreholes confirmed the general depth of the bedrock in the area.

Table 1: Calculted V_s30 values (m/s) from the MASW data (0 to 30m)

Sounding	Minimum	Average	Maximum	Site Class
1	1121	**1266**	1400	**B***

* NBC 2015 Commentary "J" requirements

The calculated average V_s30 values from the 1D MASW soundings collected was 1266m/s +/-15% to 20%.

The V_s30 values calculated for the minimum and the maximum envelopes ranged from 1121 to 1400m/s.

Based on the average V_s30 values (as determined through the MASW method) and table 4.1.8.4.A of the National Building Code of Canada, 2015 Edition, the investigated area is site class "C" ($360 < V_S30 \leq 760$ m/s).

At the request of the client, the V_s30 values have also been re-calculated taking in to consideration of the overburden. The building will be built directly on competent bedrock. The application of these recalculated Vs30* value is discussed below and the validity of these assumptions is at the discretion of the design engineer. The recalculated Vs30* values are presented in Table 2.

Table 2: Re-calculated V_s30 values (m/s) from the MASW data (2 to 32m)

Sounding	Minimum	Average	Maximum	Site Class
1	1627	**1784**	1966	**A***

* NBC 2015 Commentary "J" requirements

Based on the average V_s30* values (as determined through the MASW method), taking into consideration the proposed excavation depth as provided by the client, and table 4.1.8.4.A of the National Building Code of Canada, 2010 Edition, the investigated area is site class "A" ($V_S30 > 1500$ m/s). This assumes that the building will be founded directly on the competent bedrock and that the rock is of consistent or better quality at depth.

The use of site class "A" is conditional on the requirements of Commentary "J" sentence 100, specifically, *"Site Classes A and B, are not to be used if there is more than 3 m of soil between the rock surface and the bottom of the spread footing or mat foundation, even if the computed average shear wave velocity is greater than 760m/s*".

It must be noted that the site classification provided in this report is based solely on the V_s30 value as derived from the MASW method and that it can be superseded by other geotechnical information. This geotechnical information includes, but is not limited to, the presence of sensitive and/or liquefiable soils, more than 3m of soft clays, high moisture content, etc. The reader is referred to section 4.1.8.4 of the National Building Code of Canada, 2015 Edition for more information on the requirements for site classification.

This report has been written by Lhoucin Taghya, P.Geo.

Lhoucin Taghya, P.Geo.
Geophysicist

FINAL

Water Taking and Discharge Plans

320 McRae Avenue, 1976 Scott Street, and 311 & 315 Tweedsmuir Avenue, Ottawa, Ontario

Prepared for:

1213763 Ontario Inc.
33 Yonge Street, Suite 1000
Toronto, ON M5E 1G4

Attn: Mr. Andrew Hanna

August 7, 2020

Pinchin File: 230236.008

Issued to:	**1213763 Ontario Inc.**
Contact:	**Mr. Andrew Hanna**
Issued on:	**August 7, 2020**
Pinchin File:	**230236.008**
Issuing Office:	**Kanata, ON**

Author: Wesley Tabaczuk, P.Eng.
Project Manager
613.592.3387 ext. 1829
wtabaczuk@pinchin.com

Reviewer: Tim McBride, P. Geo.
Project Hydrogeologist
705.521.0560 ext. 3416
tmcbride@pinchin.com

Reviewer: Robert Daciw, B.Sc., P.Geo.
Director, Real Estate Due Diligence,
Environmental Due Diligence & Remediation
905.363.1460
rdaciw@pinchin.com

TABLE OF CONTENTS

APPENDICES

LIST OF FIGURES

1.0 INTRODUCTION

Pinchin Ltd. (Pinchin) was retained by 1213763 Ontario Inc. (Client) to complete a Water Taking and Discharge Plan Report for the proposed commercial/residential development to be located at 320 McRae Avenue, 1976 Scott Street, and 311 & 315 Tweedsmuir Avenue, Ottawa, Ontario (Site). The Site location is indicated on Figure 1.

The Client provided proposed future development plans for the Site, which include a 25-storey mixed-use commercial and residential building with a two level underground parking garage (UPG) extending across the entire property. Based on the proposed UPG levels, Pinchin anticipates that excavations will extend upwards of 8 metres below existing ground surface (mbgs) for the two levels of UPG.

In order facilitate the required subgrade construction activities, the contractor will be required to manage surface water and infiltrated groundwater within the excavation, until the subgrade components of the construction activities are completed.

The purpose of these Water Taking and Discharge Plans are to:

- Support the registration of the water taking activity with the Ministry of the Environment Conservation and Parks (MECP) Environmental Activities & Sectors Registry (EASR);
- Confirm that the takings do not result in unacceptable impacts on the natural environment or on existing water users;
- Confirm water discharge requirements:
- Establish a monitoring program for the volume of water taken daily as required by Ontario Regulation 387/04: Water Taking and Transfer;
- Affirm any significant deviation between actual and predicted impact; and
- Trigger contingency measures, if unacceptable impacts do occur.

To fulfill these objectives, sampling frequency must be adequate, suitable sample locations must be identified, proper parameters must be monitored, and appropriate mitigation measured must be defined.

A contingency plan is required to provide recommended mitigation measures that should be implemented to protect the natural environment and existing water users, if potential impacts and/or interference associated with the permitted water-taking is suspected.

1.1 Guidelines, Standards and Acts

The following guidelines, standards and Acts have been used to prepare these Water Taking and Discharge Plans:

- The Aggregate Resources of Ontario: Provincial Standards, Version 1.0 (Ministry of Natural Resources, 1997);
- Technical Guidance Document for Hydrogeological Studies in Support of Category 3 Applications (Ministry of the Environment and Climate Change, 2016a);
- Water Management: Policies, Guidelines, Provincial Water Quality Objectives (Ministry of Environment and Energy, 1994); and
- Soil, Ground Water and Sediment Standards for Use Under Part XV.1 of the Environmental Protection Act (Ministry of the Environment, 2011).

1.2 Personnel

A Qualified Person should oversee the proposed water taking program. For the purposes of these plans, a Qualified Person, is a person holding a minimum of a bachelor degree specializing in hydrogeology, water resource management, or engineering with experience and expertise in groundwater studies (Ministry of the Environment and Climate Change, 2016).

The sampling should be done by a trained environmental technician.

1.3 Description of Water Taking

The Client intends to develop the Site with two mixed-use commercial/residential buildings complete with a two level UPG extending beneath the entire property. The combined footprint area of the UPG occupies approximately 100% of the Site. In order facilitate the required subgrade construction activities, the contractor will be required to manage surface water and infiltrated groundwater within the excavation, until the subgrade components of the construction activities are completed. The average current Site elevation is approximately 64 metres above sea level (masl). The contractor intends to complete a mass excavation to an elevation of approximately 55.5 masl to accommodate the two-level UPG.

The southern portion of UPG excavation is anticipated to be approximately 110 metres (m) long with an average width of 30 m and a depth of up to 8 mbgs, while the northern portion of the UPG excavation is anticipated to be approximately 50 m long and 45 m wide with a depth of up to 8 mbgs.

The water taking will involve the excavation of sumps on an as-needed basis during soil and bedrock excavation to collect the infiltrated water, pumping water from the excavation(s) with trash pumps, with the requirement for on-Site treatment prior to discharge to the City of Ottawa Sanitary Sewer.

The water taking will have two phases, initial dewatering during the excavation, and maintenance dewatering to keep the excavation dewatered during construction works. It is anticipated that a maximum of two 2-inch trash pumps will be required for dewatering purposes. Taking into consideration the potential friction and head losses during pumping, the maximum anticipated discharge rate for each pump is estimated to be approximately 19,300 litres per hour. Assuming a maximum pumping duration of 10 hours per day (i.e. a typical construction work-day), the maximum dewatering volume meets the requirements of the EASR (400,000 litres per day). Additional details will be provided below as to how these estimates were calculated.

1.4 Physical Setting

1.4.1 Geology and Hydrogeology

Data obtained from the Ontario Geological Survey Maps, as published by the Ontario Ministry of Natural Resources, indicates that the Site is located on sandy silt to silty sand textured till on Paleozoic terrain. The underlying bedrock at this Site is of the Shadow Lake Formation consisting of limestone, dolostone, shale, arkose, and sandstone (Ontario Geological Survey Map 1972, published 1978).

Bedrock was encountered during borehole drilling at the Site between approximately 0.28 (Pinchin MW-3) and 3.38 mbgs (Paterson BH3-14). The water level elevations recorded across the Site, in November of 2018 (at MW1 and MW5), ranged from 58 to 58.31 masl (i.e. ~ 5.07 to 5.95 mbgs) and indicate that the static water levels are within the underlying bedrock.

Manual measurements of stabilized groundwater levels in two of the monitoring wells on Site were collected periodically throughout a duration of approximately six months. Groundwater was encountered at depths ranging between approximately 3.6 and 6.1 mbgs. The following table summarizes the water level measurements over the six month period:

Date of Reading	MW1	MW5
Nov 13, 2018	5.95 mbgs	5.07 mbgs
Jan 22, 2020	4.18 mbgs	3.70 mbgs
March 17, 2020	4.07 mbgs	3.62 mbgs
May 15, 2020	6.09 mbgs	5.18 mbgs

The water level elevations observed across the Site, in January of 2020, ranged from 59.2 to 60.25 masl (i.e. ~ 4.18 to 3.70 mbgs). The water level elevations observed across the Site, in March of 2020, ranged from 59.31 to 60.33 masl (i.e. ~ 4.07 to 3.62 mbgs). The water level elevations observed across the Site, in May of 2020, ranged from 57.86 to 58.2 masl (i.e. ~ 5.18 to 6.09 mbgs). These results, spanning over several years and multiple seasons, indicate that there are minimal seasonal fluctuations in the static water table within the bedrock aquifer, with the exception of the high water associated with spring freshet events.

No surface water (inflow or outflow drainage) and no groundwater (leaching, springs, seeps, etc.) were identified that would suggest a hydrological or hydrogeological connection to the surrounding area that could be impacted by the dewatering activities.

1.4.2 Surface Water Features

The Site is located in the Rideau Valley Watershed. No surface water features were identified on-Site. The closest surface water source is the Ottawa River located approximately 800 m west of the Site.

1.5 Hydrogeological Assessment Program

1.5.1 Borehole Drilling and Monitoring Well Installation

In order to acquire the Site-specific information needed to characterize the hydrogeological setting and assess the potential for environmental impacts, Pinchin relied on previously completed intrusive drilling and soil sampling program, including monitoring well installations.

Pinchin retained Strata Drilling Group (Strata) to complete the borehole drilling program at the Site on November 1, 2, 9, and 12, 2018, following the clearance of underground services in the vicinity of the work area by public utility locators and a private utility locator retained by Pinchin. Strata is licensed by the MECP in accordance with Ontario Regulation 903 (as amended) to undertake borehole drilling/well installation activities.

Pinchin completed the field component of the geotechnical investigation at the Site by advancing a total of nine sampled boreholes (Boreholes BH4 to BH12) throughout the Site. The boreholes were advanced to sampled depths ranging from approximately 0.8 to 3.2 mbgs, where refusal was encountered on bedrock. In addition, a 3.0 m and a 19.8 m long bedrock core with NQ sized diamond bit core barrel were advanced at the base of boreholes BH4 and BH12, respectively, to confirm the presence of bedrock and to evaluate the Rock Quality Designation (RQD). The approximate spatial locations of the boreholes advanced at the Site are indicated on Figure 2.

The boreholes were advanced with the use of a Geoprobe 7822 DT direct push drill rig which was equipped with standard soil sampling equipment. Soil samples were collected at 0.76 m intervals using a 51 mm outside diameter (OD) split spoon barrel in conjunction with Standard Penetration Tests (SPT) “N” values (ASTM D1586). The SPT “N” values were used to assess the compactness condition of the non-cohesive soil.

Groundwater observations and measurements were obtained from the open boreholes during and upon completion of drilling. The groundwater observations and measurements recorded are included on the appended borehole logs.

The bedrock cores were advanced in accordance with ASTM D2113. The bedrock types and RQD’s were evaluated immediately upon core retrieval.

The field investigation was monitored by experienced Pinchin personnel. Pinchin logged the drilling operations and identified the soil samples and rock cores as they were retrieved. The recovered soil samples were sealed into plastic bags and carefully transported to an independent and accredited materials testing laboratory for detailed analysis and testing. All soil samples were classified according to visual and index properties by the project engineer.

In addition to the intrusive investigation described above, Pinchin as part of the environmental component of the project, advanced five additional boreholes to a maximum depth of 7.6 mbgs using a Geomachine GM 100 direct push drill rig equipped with air-rotary hammer and Geoprobe 7822DT direct push drill with split spoons. Boreholes MW-1 through MW-5 were advanced into the shallow bedrock stratigraphy encountered at the Site. Soil samples were collected at continuous intervals using 38 mm inner diameter (ID) direct push soil samplers with dedicated single-use sample liners and 51 mm outer diameter split-spoon samplers. No bedrock sampling was completed during the environmental portion of the program due to the destructive nature of the drilling methodology.

Subsurface soil conditions were logged on-Site by Pinchin personnel at the time of drilling. The locations of the boreholes are indicated on Figure 2 and a description of the subsurface stratigraphy encountered during the drilling program is documented in the borehole logs included in Appendix II.

Groundwater monitoring wells were installed in boreholes MW-1, MW-2, MW-3, MW-4 and MW-5 to enable groundwater monitoring and sampling. The monitoring wells were constructed with 51 mm ID flush-threaded Schedule 40 polyvinyl chloride (PVC) risers, followed by a length of 51 mm ID No. 10 slot PVC screen that intersected the water table.

Each well screen was sealed at the bottom using a threaded cap and each riser was sealed at the top with a lockable J-plug cap. Silica sand was placed around and above the screened interval to form a filter pack around the well screen. A layer of bentonite was placed above the silica sand and was extended to just below the ground surface. A protective aboveground monument casing was installed at the ground surface over each riser pipe and outer casing and cemented in place.

The location of the monitoring wells are depicted on Figure 2. The monitoring well construction details are provided on the borehole logs included in Appendix II and in Table 1.

Table 1: Monitoring Well Construction Details:

Monitoring Well Location	TOC Elevation (masl)	Surveyed Ground Elevation (masl)	Calculated Difference Between Ground and TOC (m)	Length of Screen (m)
MW-1	100.19	100.29	-0.10	3.05
MW-2	100.17	100.28	-0.11	3.05
MW-3	NM	NM	NM	3.05
MW-4	100.70	100.81	-0.11	3.05
MW-5	100.46	100.56	-0.10	3.05

Notes: TOC Indicates Top of Casing
masl Indicates elevation in metres above sea level

1.5.2 Groundwater Hydraulic Conductivity

Hydraulic conductivity, which is a measure of water's ability to move through the aquifer medium, is one of the important factors in monitoring the fate and transport of contaminants in the subsurface and yields of the aquifer. This characteristic controls the rate and the distribution of the water within the overburden, as well as the bedrock. Hydraulic conductivity (permeability) of a geological formation is determined by the rate of the recovery of the water level after a known volume of water is added or removed.

Hydraulic conductivity testing was completed at monitoring wells MW-1 and MW-5 on March 17, 2020. The depth to groundwater was measured at each of the monitoring wells prior to the completion of the hydraulic conductivity testing activities in order to determine static water levels.

A negative hydraulic head displacement (rising head test) was created by removing the standing water in the monitoring well using dedicated inertial pumps comprised of Waterra polyethylene tubing and foot valves to draw the groundwater to the surface. The hydraulic recovery within the monitoring wells was recorded over the course of the rising head tests.

The rising head test procedure employs the hydrostatic time-lag method for groundwater recovery following the removal of a volume of water from a monitoring well and makes use of the theory of Hvorslev (1951), as described in Freeze and Cherry (1979). Hvorslev's method is expressed by the following equation:

where:

$$K = \frac{r2\ln\left(\frac{L}{R}\right)}{2LT_0}$$

K = hydraulic conductivity of the tested material (m/sec)

r = inner radius of the well riser pipe (m)

R = outer radius of the well riser pipe (m)

L = length of screen and sand pack (m)

To = time lag (s), where (H-h)/(H-Ho) = 0.37

h = water level at each time of measurement (m)

Ho = initial water level (m, start of test)

H = stabilized water level prior to introducing slug (m)

The time lag, T_0, is defined as the time required for the water level to recover to 63% of the stabilized level, if the initial flow rate into the well is maintained. This time lag is determined graphically as the time for which (H-h) divided by (H-Ho) is equal to 0.37.

A summary of the hydraulic conductivity estimates is provided below, and graphed results of the rising head test data completed for the monitoring wells are provided in Appendix III. The hydraulic conductivities estimated from the rising head tests are as follows:

Well ID	Well Depth (m)	Screen Length (m)	Screened Unit	Hydraulic Conductivity (K-Value) (cm/s)
MW1	7.62	3.05	Bedrock	8.57×10^{-6}
MW5	7.62	3.05	Bedrock	4.00×10^{-5}

A sensitivity analysis of the recovery data for each well was also completed based on curve matching and resulted in the following ranges:

Well ID	K-Value (cm/s) Best Fit Curve	K-Value (cm/s) Low Curve	K-Value (cm/s) High Curve	Average K-Value (cm/s)
MW1	8.57×10^{-6}	7.76×10^{-6}	9.477×10^{-6}	8.6×10^{-6}
MW5	4.00×10^{-5}	3.57×10^{-5}	4.13×10^{-5}	3.9×10^{-5}

The hydraulic conductivity (K-value) results from the rising head tests ranged from 8.57×10^{-6} cm/s to 4.00×10^{-5} cm/s, with an overall geometric mean of 1.85×10^{-5} cm/s.

The final hydraulic conductivity values for each of the tested monitoring wells were utilized to calculate geometric mean hydraulic conductivity value for the bedrock aquifer at the Site. Calculated hydraulic conductivity values ranged from 4.66×10^{-6} m/min to 2.5×10^{-5} m/min.

The calculated results are consistent with the recovery observations made during the field program and are consistent with the range of hydraulic conductivities for the Site-specific limestone bedrock that range from 10-3 to 10-7 m/min (Freeze & Cherry, 1979). Therefore, Pinchin has determined that the calculated hydraulic conductivities are acceptable for use as an approximation of the hydraulic conductivity for the Site.

A summary of the calculations and assumptions utilized to calculate hydraulic conductivity are provided in Appendix III.

1.5.3 Groundwater Chemistry Testing

Given the setting of the Site and the surrounding environment (i.e., The Site is situated in an area that predominantly consists of vacant, residential, institutional, commercial and light industrial land uses) there is no opportunity to discharge the water taken as part of the construction dewatering exercise directly to the environment. Based on the findings of the Phase II Environmental Site Assessment (ESA), previously completed by Pinchin and dated November 29, 2018, there is the potential for groundwater exceedances within the bedrock aquifer. This previous Phase II ESA reported concentrations in groundwater samples submitted for analysis of petroleum hydrocarbons (PHCs) (F1-F4), volatile organic compounds (VOCs), polycyclic aromatic hydrocarbons (PAHs) and metals that satisfied the applicable Ontario Ministry of the Environment, Conservation and Parks *Table 7 Standards* (for residential/parkland/institutional land use and coarse-textured soil), with the exception of groundwater sample MW-1, which exceeded the *Table 7 Standards* for PHCs (F1 and F2), benzene, xylene, and naphthalene.

Similarly, the results of the Phase Two ESA (completed to the O. Reg. 153/04 - Record of Site Condition Standards) as identified in the "*Phase Two Environmental Site Assessment – Summary*" completed by Pinchin and dated July 16, 2020, indicated that the reported concentrations in the groundwater samples submitted for analysis of PHCs (F1-F4), VOCs, PAHs and metals satisfied the *Table 7 Standards* with the following exceptions:

- Groundwater sample collected at MW-1, which had concentrations of PHCs (F1 and F2), benzene, ethylbenzene, xylenes, naphthalene, and 1-methylnaphthalene that exceeded the *Table 7 Standards*; and
- Groundwater sample collected at EXMW-1, which had a concentration of mercury that exceeded the *Table 7 Standards*.

In order to assess the dewatering discharge/disposal options available during construction, one water quality sample was collected from MW-1 for an enhanced suite of parameters. The groundwater sample was submitted to Paracel Laboratories Ltd. in Ottawa, Ontario for analysis to facilitate comparison of the current groundwater quality with the City of Ottawa Sewer Use By-law Number 2003-514. Paracel Laboratories is an independent laboratory accredited by the Standards Council of Canada. Formal chain of custody records of the sample submission was maintained between Pinchin and the staff at Paracel Laboratories. The Laboratory Certificate of Analysis is included as Appendix IV.

The results of the laboratory analysis was evaluated by comparison with the sanitary and combined sewer discharge limits, as well as the storm sewer discharge limits presented in the City of Ottawa Sewer Use By-law Number 2003-514, Schedule A, Tables 1 & 2, respectively (Sewer Use By-law). A summary of the laboratory analyses along with the Sewer Use By-law are presented in Table 1 of Appendix IV of this report. As indicated in Table 1, the water quality sampling results indicated all analyzed parameter concentrations satisfied the storm and sanitary discharge limits for the Sewer Use By-law, with the following exceptions:

- The groundwater sample collected from monitoring well MW-1 exceeded the Sewer Use By-law for the Sanitary and Combined Sewer Discharge for Sulphide (3.56 mg/L versus the Sewer Use By-law of 2 mg/L), Benzene (0.0691 mg/L versus the Sewer Use By-law of 0.01 mg/L), Ethylbenzene (0.307 mg/L versus the Sewer Use By-law of 0.057 mg/L) and 1,3,5 Trimethylbenzene (0.0081 mg/L versus the Sewer Use By-law of 0.003 mg/L);

- The groundwater sample collected from monitoring well MW-1 exceeded the Sewer Use By-law for the Storm Sewer Discharge for Phenolics (0.012 mg/L versus the Sewer Use By-law of 0.008 mg/L), Total Suspended Solids (147 mg/L versus the Sewer Use By-law of 15 mg/L), Manganese (0.36 mg/L versus the Sewer Use By-law of 0.05 mg/L), Benzene (0.0691 mg/L versus the Sewer Use By-law of 0.002 mg/L), Ethylbenzene (0.307 mg/L versus the Sewer Use By-law of 0.002 mg/L), Toluene (0.0065 mg/L versus the Sewer Use By-law of 0.002 mg/L) and Xylenes (0.0092 mg/L versus the Sewer Use By-law of 0.0044 mg/L); and
- The groundwater sample collected from the monitoring well MW-1 exceeded the both the Storm, as well as the Sanitary and Combined Sewer Discharge limits for Benzene and Ethylbenzene.

With respect to the Sewer Use By-law discharge parameters, as noted above and summarized in Table 1, the current groundwater conditions confirm that pumped groundwater should be expected to require pre-treatment prior to discharge to any City of Ottawa sewer system. Given the nature of the contaminants detected within the groundwater it is anticipated that the contractor will have to provide a mobile treatment unit (with an Environmental Compliance Approval) to ensure that the water will meet the City of Ottawa Sewer Use By-law Number 2003-514, Schedule A, Table 1 for the Sanitary and Combined Sewer Discharge Limits as a minimum.

2.0 WATER TAKING PLAN

2.1 Expected Area of Influence

The proposed dewatering is not anticipated to have any impacts on existing groundwater users as there are no drinking water wells identified within 500 m of the Site, as the Site and surrounding properties are serviced via a municipal water supply.

The anticipated radius of influence from continuous dewatering of an excavation within an aquifer system (or equivalent porous medium) can be categorized by the equation:

$$R = b \times (\text{sqrt } (k/(2*N)))$$

Where

R= Radius of influence (m)

b = depth of the excavation (m)

k = hydraulic conductivity of the formation (m/s)

N= recharge rate of the formation (m/s)

Assuming the depth of the excavation will be upwards of 6 m below the normal groundwater levels, and applying a conservative rate of recharge of approximately 1.0 m/year (6 x 10^{-8} m/s), the anticipated theoretical radius of influence beyond the face of the excavation can be expected to extend to the order of 8.5 m.

2.2 Soil Settlement

On-Site soils consist of a limited unsaturated thickness of sand and gravel fill deposits underlain by a continuous limestone bedrock. Given the anticipated radius of influence and the assumption that the surrounding developments and infrastructure are founded on or in bedrock in is unlikely that the proposed dewatering would result in the depressurization of surrounding aquifers that would result in the consolidation of clays or other materials which could lead to soil settlement.

Care should be exercised during dewatering to ensure that areas beyond the excavation are not being significantly dewatered.

2.3 Anticipated Seepage Rates

Based on the results of the recovery testing program, the average hydraulic conductivity of the instrumented bedrock aquifer material is on the order of 1.85 x 10^{-5} cm/s (1.85 x 10^{-7}m/s). To assist in the quantification of seepage water to be anticipated, using Darcy's Law, and applying it to a vertical excavation (i.e. slope is 1.0), the anticipated seepage during initial excavation will be approximately 6.89 x 10^{-4} L/s/m^2 of vertically exposed excavation (i.e., approximately 275,000 l/day). During extended pumping, as the radius of influence is extended outward, the seepage rate will decline as the slope increases as it approaches its limits. During prolonged pumping, it should be anticipated that the seepage rates will drop to the order of 1.62 x 10^{-4} L/s/m^2 of vertically exposed excavation (i.e., approximately 65,000 l/day).

It should be noted that this dewatering estimation has been prepared for discussion purposes only. It is the responsibility of the contractor to propose a suitable dewatering system based on the groundwater elevation at the time of construction. The method used should not adversely impact any nearby structures. It is the responsibility of the contractor to make this application if required. Depending on the groundwater at the time of the excavation works, a more involved dewatering system may be required.

2.4 Water Quantity Monitoring

Ontario Regulation (O. Reg.) 63/16: Registrations Under Part II.2 of the Act - Dewatering requires all registrants to measure and record the volume of water taken daily using a flow meter or another calculation method acceptable to MECP. This data must be reported to MECP annually by March 31st.

3.0 DISCHARGE PLAN

3.1 Discharge Location and Methods

The water will be pumped from the excavation using submersible pumps (during normal operation and in the event of a 100 year storm event), and will be conveyed to an on-Site storage vessel to facilitate precipitation of suspended solids prior to treatment via an appropriately designed (from a quantity and quality perspective) mobile treatment unit to accommodate discharge to the City of Ottawa Sanitary Sewer in accordance with the limits dictated in the Sewer Use By-law.

The final discharge location will be established through consultation with City of Ottawa.

3.2 Water Quality Monitoring

Typically, Environmental Compliance Approvals (ECAs) that accompany mobile treatment unit require water quality monitoring prior to and after treatment, to ensure the compliance with the standards applicable to the receiver. As a result, water quality will need to follow the conditions of the ECA for the mobile treatment unit, as well as the requirements of the City of Ottawa Sewer Use By-law.

Water samples will need to be collected and sent to a CALA accredited laboratory for analysis to ensure that the water discharge does not result in unacceptable impacts on the ultimate receiver. The City of Ottawa Sewer Use By-law indicates that sampling and analysis required by the by-law shall be carried out in accordance with the procedures, modified or unmodified, as described in Standard Methods, the Ministry of Energy and the Environment and Energy publication entitled "Protocol for the Sampling and Analysis of Industrial/Municipal Wastewater" dated August, 1994, the United States Environmental Protection Agency methods or analytical methods adopted by the City. The following sections outline the recommended sampling protocols and the parameters that samples should be analysed for.

3.2.1 Parameters for Laboratory Analysis

Samples need to be collected in the field, as outlined in the aforementioned publication, using appropriate equipment and sampling bottles. The appropriate sample bottles for the parameters for analysis will be provided by the analytical laboratory. In general, samples need to be accurately, clearly and concisely labelled, then packed into a cooler with ice with a chain of custody form in a waterproof bag and shipped to the laboratory as soon as possible.

It is recommended that the analytical results are compared to the Sewer Use By-law to determine if the water generated from the construction dewatering remains of acceptable quality prior to discharge.

Water quality analyses generally involve laboratory analysis of the following parameters:

3.2.1.1 Metals

Aluminum-dissolved	Chromium	Dissolved Mercury	Titanium
Antimony	Chromium (VI)	Molybdenum	Vanadium
Arsenic	Cobalt	Nickel	Zinc
Bismuth	Copper	Selenium	
Boron	Lead	Silver	
Cadmium	Manganese	Tin	

3.2.1.2 Other Parameters

Chemical Biological Oxygen Demand	Total Kjeldahl Nitrogen	Oil & Grease (Animal/Vegetable, Mineral/Synthetic, Total)
Cyanide (Total)	Total Phosphorus	Semi Volatiles
pH	Sulphide	Pesticides (Hexachlorobenzene)
Phenolics	Fluoride	Polychlorinated Biphenyls (PCB)
Total Suspended Solids	Sulphate	Volatiles

3.2.2 Field Sampling

The recommended field parameters to be measured during each sampling event are:

Dissolved oxygen (DO)	Dissolved oxygen percent	pH
Temperature	Turbidity	Electrical conductivity

It is recommended that field parameters are collected using a handheld multi-meter and turbidity is measured using a turbidity meter. Prior to using this equipment, it will need to be calibrated as outlined in the information supplied with the equipment. Calibration and usage guidelines need to be reviewed with a Qualified Person to ensure proper usage.

It is recommended that all equipment be handled with care to prevent damage to sensors, probes, housing and storage cases. It should be stored in a secure, dry, temperature controlled (i.e. above 5^{o}C) that is inaccessible to anyone other than the Qualified Person and their designate(s).

3.2.3 Sampling Locations and Frequency

The recommended sampling locations are the influent and effluent of the mobile treatment unit. Sampling must be conducted in accordance with the frequency specified in the City of Ottawa Sewer Use By-law (as dictated by the Compliance Officer) and the conditions of the ECA for the mobile treatment unit.

3.2.3.1 Quality Assurance and Quality Control

It is important to ensure quality of field sampling and laboratory analysis (QA/QC), as such it is recommended that random duplicate samples be collected. Duplicate samples are two samples collected at the same time and in the same place.

3.3 Monitoring for Impacts to the Environment

Dependant on the configuration of the discharge to the City of Ottawa Sewer, this dewatering program has the potential to cause erosion and sedimentation at the discharge point. To prevent these problems, a visual inspection along the path of the dewatering and treatment equipment and at the discharge location should be conducted at least daily during the initial dewatering. During maintenance dewatering, visual inspections should be conducted on a minimum of a weekly basis, while also following the requirements of the Sewer Use By-law (as dictated by the Compliance Officer) and the ECA.

The following mitigation measures should be implemented to avoid erosion (Sustainable Resource Development, 2010):

- Maintain the existing vegetation at the discharge point to prevent exposing soil;
- Avoid concentrating the flow of water to prevent the creation of rills and gullies;
- Install a flow/energy dissipation measure at the discharge point; and
- Develop and maintain a sediment and erosion control program for the duration of the construction period, including the dewatering phase.

4.0 CONTINGENCY PLAN

A contingency plan recommends mitigation measures that should be implemented to protect the natural environment and existing water users from impacts and interference associated with the permitted water-taking.

4.1 Triggers

This contingency plan should be implemented if:

- A complaint is received;
- The proponent or MECP determines interference is occurring;

- The water quality of the influent or effluent of the mobile treatment unit is outside of specifications; or
- The natural environment is being degraded (e.g. erosion and sedimentation are occurring; flooding associated with water discharge is occurring, etc.).

4.2 Mitigation Measures

4.2.1 Interference with Water Users and Impacts to the Natural Environment

The following steps will be followed to prevent interference and impacts:

1. Conduct monitoring as outlined in the Plan;
2. If this contingency plan is triggered, dewatering shall cease immediately;
3. The complaint, or problem will be investigated and rectified. Rectifying the issue may involve changes to the methods or equipment, the timing of the dewatering, increased monitoring, groundwater monitoring, etc.;
4. Dewatering will recommence; and
5. The revised monitoring program will be implemented.

5.0 TERMS AND LIMITATIONS

This report was prepared pursuant to and in accordance with the master services agreement (the “MSA”) dated July 16, 2007 (amended August 8, 2018) between The Pinchin Group of Companies (“Consultant”) and the other parties listed thereto, and the project specific agreement dated January 17, 2020, between Consultant and 1213763 Ontario Inc. The report was prepared by Consultant for the use of Owner and Manager (as those terms are defined under the MSA). In addition to the use of and reliance on this report by Owner and Manager, any person who has received a reliance letter for this report may use and rely on this report as if it was prepared for such persons. Any use of or reliance on this report by any other person (i.e., a person other than any Owner, Manager or otherwise permitted person) is the sole and exclusive responsibility of such other person. Consultant accepts no responsibility for damages, if any, suffered by such other person as a result of the use of or reliance on this report.

This report is based on the best information available to Consultant at the time of preparing this report after Consultant has used best industry practices, in the circumstances, to obtain information. To the extent that Consultant was required to rely on information from other persons, Consultant has verified such information to the extent reasonably possible in the circumstances. The material provided in this report reflects best industry judgment in light of the information available at the time of preparation of this report.

6.0 REFERENCES

1. City of Ottawa. (2003). Sewer Use (By-law No. 2003-514). Retrieved from https://ottawa.ca/en/living-ottawa/laws-licences-and-permits/laws/law-z/sewer-use-law-no-2003-514#sewer-use-law-no-2003-514
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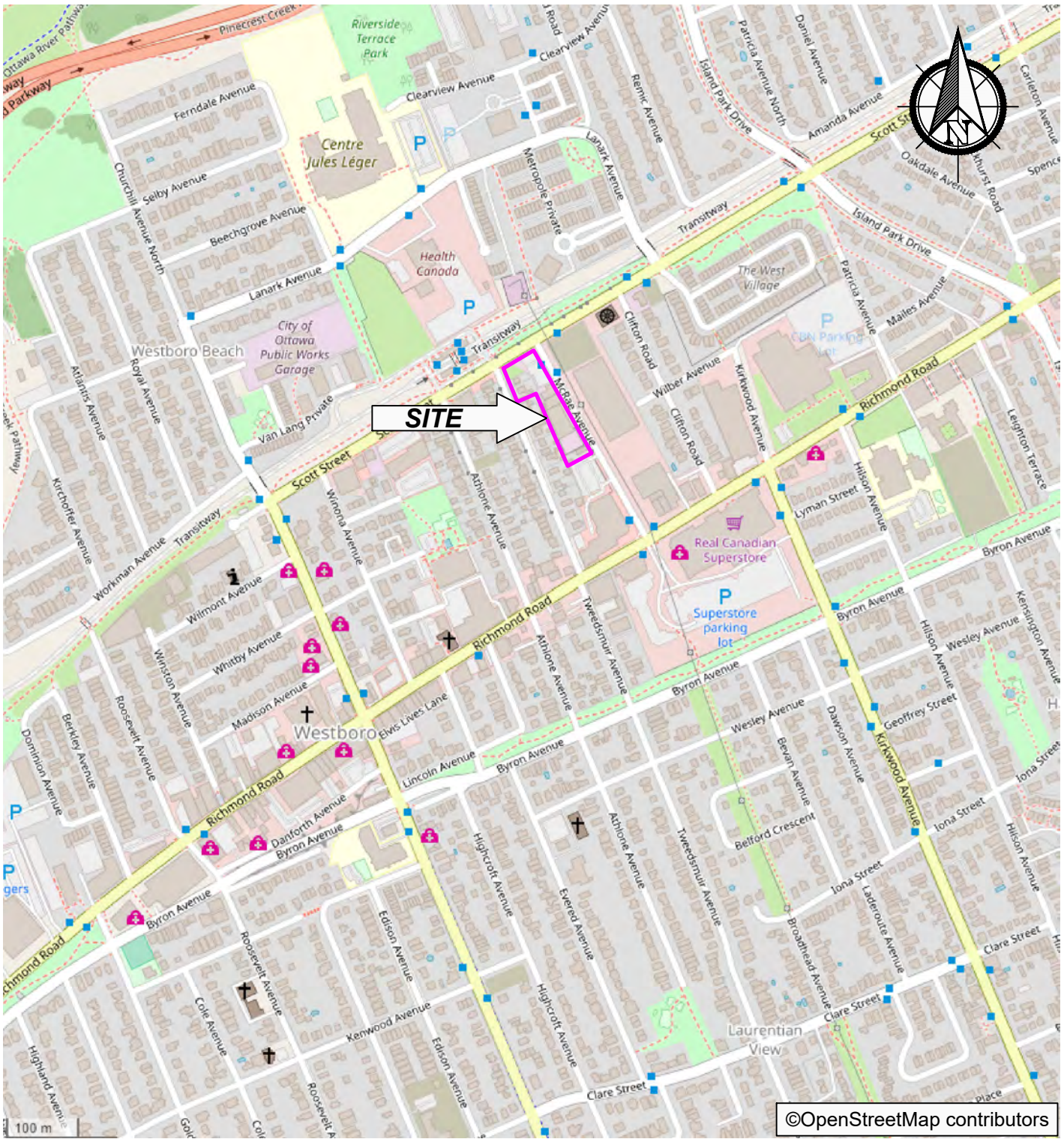
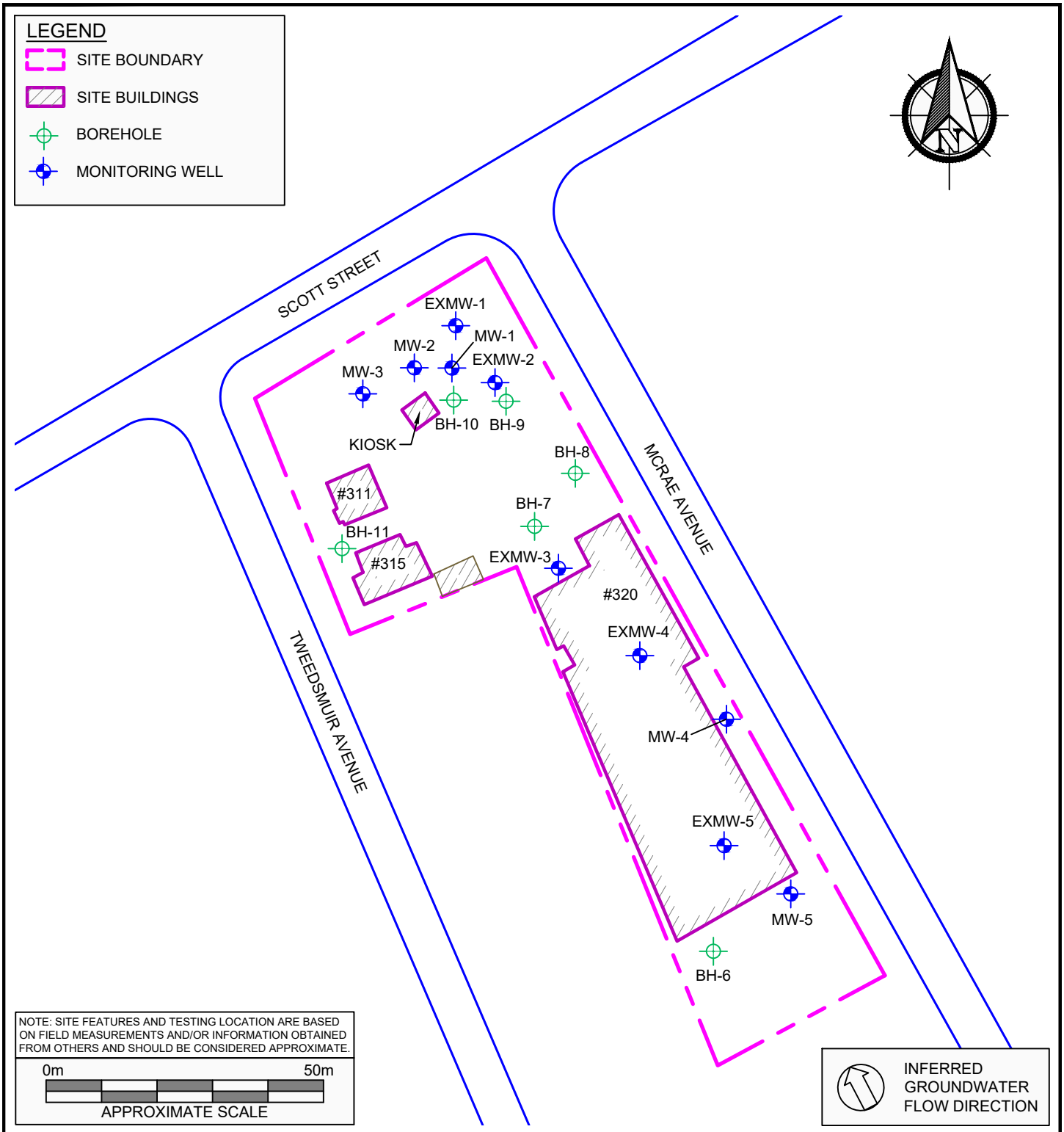
APPENDIX I
Figures



PINCHIN

PROJECT NAME	WATER TAKING AND DISCHARGE PLANS	
CLIENT NAME	1213763 ONTARIO INC.	
PROJECT LOCATION	320 MCRAE AVENUE, 1976 SCOTT STREET, 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO	
FIGURE NAME	KEY MAP	FIGURE NO. 1
APPROXIMATE SCALE	AS SHOWN	
PROJECT NO.	230236.008	
DATE	JULY 2020	



PINCHIN

PROJECT NAME	WATER TAKING AND DISCHARGE PLANS		
CLIENT NAME	1213763 ONTARIO INC.		
PROJECT LOCATION	320 MCRAE AVENUE, 1976 SCOTT STREET, 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO		
FIGURE NAME	BOREHOLE AND MONITORING WELL LOCATION PLAN		FIGURE NO. 2
APPROXIMATE SCALE	PROJECT NO.	DATE	
AS SHOWN	230236.008	JULY 2020	

APPENDIX II
Borehole Logs

Log of Borehole: BH12

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 9 and 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE									
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (□ 20 40 60 □)	Shear Strength (△ kPa △ 100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
0		Ground Surface	0.00											
		Organics ~ 200 mm Fill - gravelly silty sand, trace brick, trace glass, trace bedrock fragments, damp, brown, compact			SS	SS1	60	17						
1			-1.22		SS	SS2	25	12						
		Limestone rock, slightly weathered in the upper layers and fresh in the deeper layers. Grey with black and white banding, fine to medium grained, and contained few natural fractures with little to no oxidation. Poor Quality			NQ	Run 1	72							RQD=33%
2														
3					NQ	Run 2	88							RQD=51%
4			-4.27											
		Good to excellent quality			NQ	Run 3	92							RQD=82%
5														
6					NQ	Run 4	100							RQD=88%
7														
8					NQ	Run 5	95							RQD=93%
9														
10					NQ	Run 6	98							RQD=93%

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 1 of 2

Log of Borehole: BH12

Project #: 230236.004 **Logged By:** WT

Project: Geotechnical Investigation

Client: 1213763 Ontario Inc.

Location: McRae Ave., Scott St., and Tweedsmuir Ave., Ottawa, Ontario

Drill Date: November 9 and 12, 2018 **Project Manager:** WT

SUBSURFACE PROFILE					SAMPLE										
Depth (m)	Symbol	Description	Elevation (m)	Monitoring Well Details	Sample Type	Sampler #	Recovery (%)	SPT N-Value	Standard Penetration N-Value (□ 20 40 60 □)	Shear Strength (△ kPa △ 100 200)	Water Content (%)	Sample ID	Soil Vapour Concentration (ppm)	Laboratory Analysis
11					NQ	Run 7	100							RQD=85%
12					NQ	Run 8	98							RQD=83%
13					NQ	Run 9	97							RQD=80%
14														
15					NQ	Run10	100							RQD=77%
16			-16.46											
		Very poor quality			NQ	Run11	100							RQD=17%
17			-17.37											
		Fair quality			NQ	Run12	100							RQD=63%
18			-18.90											
		Excellent quality			NQ	Run13	100							RQD=93%
19														
20			-20.42											
		End of Borehole												

Contractor: Strata Drilling Group **Grade Elevation:** N/A

Drilling Method: Direct Push/Split Spoon **Top of Casing Elevation:** N/A

Well Casing Size: N/A **Sheet:** 2 of 2

Log of Borehole: MW-1

Project #: 230236.002 **Logged By:** MK

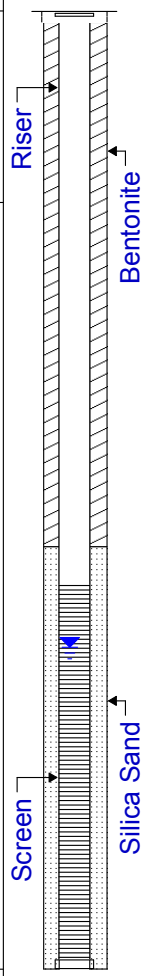
Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0 / 0		***Asphalt***						
1 / 0		***Sand and Gravel - Fill*** Brown, damp.			30	SS1	0/0	
3 / 1					30	SS2	0/1	Metals, PHCs, PAHs, VOCs, pH
4 / 1			1.52					
5 / 2		***Limestone***						
25 / 7			7.62					
26 / 8		End of Borehole						



Water level measured at 5.07 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.29

Top of Casing Elevation: 100.19

Sheet: 1 of 1

Log of Borehole: MW-2

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0–4 / 0–1		***Sand and Gravel*** Grey/brown, damp.			30	SS1	0/1	
4–5 / 1			1.52		30	SS2	0/1	Metals, PHCs, PAHs, VOCs
5–25 / 2–7		***Limestone***						
25 / 8			7.62					
		End of Borehole						

Riser, Bentonite, Screen, Silica Sand

Water level measured at 6.13 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.17

Top of Casing Elevation: 100.28

Sheet: 1 of 1

Log of Borehole: MW-3

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 1, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0–3 / 0–1		***Sand and Gravel*** Grey/brown, damp. Limestone fragments @ 0.75 mbgs	0.28	Riser; Bentonite	30	SS1	0/1	PHCs, VOCs, PAHs
3–25 / 1–7		***Limestone***		Screen; Silica Sand				
25 / 7		End of Borehole	7.62	Well was submerged and frozen on November 13, 2018.				

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: MW-4

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					PHCs, VOCs, PAHs, Metals
1		***Sand and Gravel*** With brick fragments, damp.	0.76		50	SS1	0/2	
3 / 1		***Fill*** Sand, brick and glass.	1.07		20	SS2	0/1	
4		***Limestone***		Riser; Bentonite				
2								
3								
4								
5								
6								
7				Screen; Silica Sand				
8			7.62					
		End of Borehole		Water level measured at 4.37 mbgs on November 13, 2018.				

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.81

Top of Casing Elevation: 100.70

Sheet: 1 of 1

Log of Borehole: MW-5

Project #: 230236.002 **Logged By:** MK

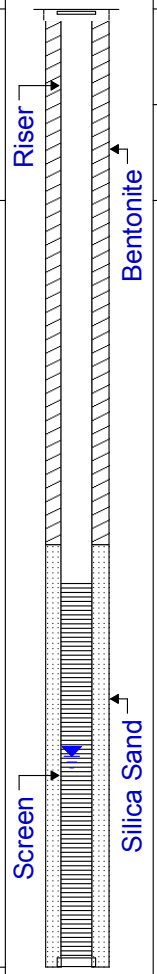
Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0 / 0		***Asphalt***						
1 / 0		***Sand and Gravel*** Grey/brown.			50	SS1	0/2	PHCs,
3 / 1					50	SS2	0/3	VOCs, PAHs, Metals, Grain Size
4 / 1			1.52					
5 / 2		***Limestone***						
25 / 7			7.62					
26 / 8		End of Borehole						



Water level measured at 5.95 mbgs on November 13, 2018.

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon / Air Rotary

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: 100.46

Top of Casing Elevation: 100.56

Sheet: 1 of 1

Log of Borehole: BH-8

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
		Asphalt	0.10	No Monitoring Well Installed	50	SS1	0/1	PHCs, VOCs, PAHs, Metals
1		***Sand and Gravel*** Brown, with some silt and organics, damp.						
2			0.84					
3 / 1		End of Borehole Due to refusal on Bedrock.						
4								
5								

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NA

Top of Casing Elevation: NA

Sheet: 1 of 1

Log of Borehole: BH-6

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00	No Monitoring Well Installed				
		Asphalt	0.15		50	SS1	0/1	
1		***Sand and Gravel*** Grey/brown, damp.						
2								
3 / 1					80	SS2	0/3	PHCs, VOCs, PAHs, Metals, pH
4								
5			1.52					
		End of Borehole Due to refusal on Bedrock.						
6								
7 / 2								
8								
9								
10 / 3								

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: BH-7

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00	No Monitoring Well Installed				
		Asphalt	0.10		50	SS1	0/3	PHCs, VOCs, PAHs, Metals, pH
1		***Sand and Gravel*** With wood fragments, damp, black staining, no odours.	0.76					
2								
3 / 1		***Sand and Gravel*** Brown/grey, with silt, damp.			50	SS2	0/1	
4								
5			1.52					
		End of Borehole Due to refusal on Bedrock.						
6								
7 / 2								
8								
9								
10 / 3								

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: BH-9

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
0–10 / 0–3		***Sand and Gravel*** Brown, damp.		No Monitoring Well Installed	50	SS1	0/2	
					50	SS2	0/1	
					50	SS3	0/2	PHCs, VOCs, PAHs, Metals, pH, Grain Size
					50	SS4	0/2	
10 / 3			3.29					
11 / 3.4		End of Borehole						
		Due to refusal on Bedrock.						

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: BH-10

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
		Asphalt	0.10	No Monitoring Well Installed	45	SS1	0/2	PHCs, VOCs, PAHs, Metals, pH
2			0.76					
3 / 1		***Sand and Gravel*** Brown/black, with limestone fragments thoughout, damp.			85	SS2	0/1	
5			1.68					
6 / 2		End of Borehole Due to refusal on Bedrock.						
7								
8								
9								
10 / 3								

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

Log of Borehole: BH-11

Project #: 230236.002 **Logged By:** MK

Project: Phase II Environmental Site Assessment

Client: 1213763 ONTARIO INC

Location: 320 McRae Avenue, 1976 Scott Street, 311 and 316 Tweedsmuir Avenue, Ottawa, Ontario

Drill Date: November 2, 2018

SUBSURFACE PROFILE					SAMPLE			
Depth (ft / m)	Symbol	Description	Measured Depth (m)	Monitoring Well Details	Recovery (%)	Sample ID	Soil Vapour Concentration (ppm)(HEX/IBL)	Laboratory Analysis
0 / 0		Ground Surface	0.00					
		Asphalt	0.05	No Monitoring Well Installed	75	SS1	0/2	PHCs, VOCs, PAHs, Metals
1		***Sand and Gravel*** Brown, with large stones, damp.						
2			0.76					
		End of Borehole						
3		Due to refusal on Bedrock.						
1								
4								
5								

Contractor: Strata Drilling Group

Drilling Method: Direct Push / Split Spoon

Well Casing Size: 5.08 cm

Note:
Soil vapour concentrations measured using a RKI Eagle 2 equipped with a photoionization detector (PID) and a combustible gas indicator (CGI).

Grade Elevation: NM

Top of Casing Elevation: NM

Sheet: 1 of 1

APPENDIX III
Hydraulic Conductivity Data

MW-1

HYDROGEOLOGY

PIEZOMETER RECOVERY TESTS - HVORSLEV METHOD

Instructions: 1) Take initial water level before pumping well or removing/inserting any pumping equipment.
2) Using water level meter, measure depth to bottom of piezometer.
2) Have water level meter ready.
3) Pump piezometer down to lowest possible level using manual or mechanical methods.
4) Remove all pumping equipment from piezometer and store in sterile container, if possible.
5) Immediately insert water level meter to read 1st reading as Time 0 seconds (i.e.T_0=0s).
6) Take water level at regular time intervals increasing with time and fill in table below.
7) Enter data in table below as well as in marked (non-highlighted) cells.
8) On chart, apply best fit line to plotted data and find "t" value corresponding with

Variables:

Datum = Elevation at or below bottom of piezometer can be assumed as 0m

Z_w = Measured Depth of Well = 7.62 m

Z_∞ = Measured Depth to Stable Water Level = 4.07 m

Z_t = Measured depth of water at time t (enter in table)

Z_0 = Measured Depth to Water Level at t=0 min = 5.7 m

H = Stable water elevation above Datum = $Z_w - Z_\infty$ = 3.55 m

h_t = Height of water above datum at time t (enter in table)

h_0 = Height of water above datum at start of test (T_0)= $Z_w - Z_0$ = 1.92 m

R = Radius of Borehole (m) = 0.1 m

L = Length of filter sand pack (m) = 3.4 m

r = Radius of well casing (m) = 0.025 m

Time (sec)	Time (hrs)	Depth Z_t (m)	h_t (m)	$(H-h_t)/(H-h_0)$ (H-ht)/(H-h0)
0	0	5.7	1.92	1
60	0.016666667	5.66	1.96	0.97546012
120	0.033333333	5.6	2.02	0.93865031
180	0.05	5.57	2.05	0.9202454
240	0.066666667	5.54	2.08	0.90184049
300	0.083333333	5.52	2.1	0.88957055
360	0.1	5.5	2.12	0.87730061
420	0.116666667	5.48	2.14	0.86503067
480	0.133333333	5.46	2.16	0.85276074
540	0.15	5.44	2.18	0.8404908
600	0.166666667	5.42	2.2	0.82822086
900	0.25	5.34	2.28	0.7791411
1200	0.333333333	5.26	2.36	0.73006135
1500	0.416666667	5.18	2.44	0.6809816
1800	0.5	5.1	2.52	0.63190184
2400	0.666666667	4.98	2.64	0.55828221
2700	0.75	4.91	2.71	0.51533742
3000	0.833333333	4.85	2.77	0.47852761
3300	0.916666667	4.8	2.82	0.44785276
3600	1	4.75	2.87	0.41717791
4200	1.166666667	4.67	2.95	0.36809816
4800	1.333333333	4.58	3.04	0.31288344
5400	1.5	4.5	3.12	0.26380368
6000	1.666666667	4.42	3.2	0.21472393
6600	1.833333333	4.36	3.26	0.17791411
7200	2	4.3	3.32	0.14110429
7800	2.166666667	4.24	3.38	0.10429448
8400	2.333333333	4.18	3.44	0.06748466
9000	2.5	4.12	3.5	0.03067485

From chart, T_o = 1.05

$K = \frac{r^2 \ln(L/R)}{2LT_0}$ = 0.0003087 m²/(m·h)

K = 8.574E-06 cm/s

Sensitivity Analysis

From chart, T_o = 1.16 Low

$K = \frac{r^2 \ln(L/R)}{2LT_0}$ = 0.0002794 m²/(m·h)

K = 7.761E-06 cm/s

From chart, T_o = 0.95 High

$K = \frac{r^2 \ln(L/R)}{2LT_0}$ = 0.0003412 m²/(m·h)

K = 9.477E-06 cm/s

0	0.37
1000	0.37

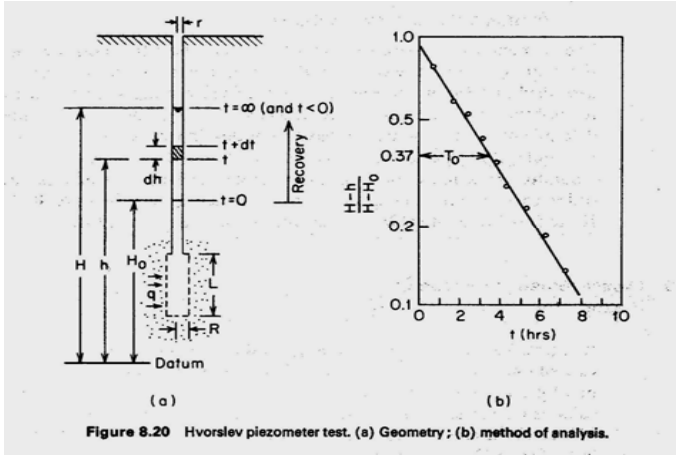


Figure 8.20 Hvorslev piezometer test. (a) Geometry; (b) method of analysis.

MW-5

HYDROGEOLOGY

PIEZOMETER RECOVERY TESTS - HVORSLEV METHOD

Instructions: 1) Take initial water level before pumping well or removing/inserting any pumping equipment.
2) Using water level meter, measure depth to bottom of piezometer.
2) Have water level meter ready.
3) Pump piezometer down to lowest possible level using manual or mechanical methods.
4) Remove all pumping equipment from piezometer and store in sterile container, if possible.
5) Immediately insert water level meter to read 1st reading as Time 0 seconds (i.e.T_0=0s).
6) Take water level at regular time intervals increasing with time and fill in table below.
7) Enter data in table below as well as in marked (non-highlighted) cells.
8) On chart, apply best fit line to plotted data and find "t" value corresponding with

Variables:

Datum = Elevation at or below bottom of piezometer can be assumed as 0m

Z_w = Measured Depth of Well = 7.62 m

Z_∞ = Measured Depth to Stable Water Level = 3.62 m

Z_t = Measured depth of water at time t (enter in table)

Z_0 = Measured Depth to Water Level at t=0 min = 5.15 m

H = Stable water elevation above Datum = $Z_w - Z_\infty$ = 4 m

h_t = Height of water above datum at time t (enter in table)

h_0 = Height of water above datum at start of test (T_0)= $Z_w - Z_0$ = 2.47 m

R = Radius of Borehole (m) = 0.1 m

L = Length of filter sand pack (m) = 3.4 m

r = Radius of well casing (m) = 0.025 m

Time (sec)	Time (hrs)	Depth Z_t (m)	h_t (m)	$(H-h_t)/(H-h_0)$ $(H-ht)/(H-h0)$
0	0	5.15	2.47	1
60	0.016666667	5.02	2.6	0.91503268
120	0.033333333	4.95	2.67	0.86928105
180	0.05	4.86	2.76	0.81045752
240	0.066666667	4.79	2.83	0.76470588
300	0.083333333	4.73	2.89	0.7254902
360	0.1	4.64	2.98	0.66666667
420	0.116666667	4.59	3.03	0.63398693
480	0.133333333	4.52	3.1	0.58823529
540	0.15	4.45	3.17	0.54248366
600	0.166666667	4.39	3.23	0.50326797
660	0.183333333	4.33	3.29	0.46405229
720	0.2	4.27	3.35	0.4248366
780	0.216666667	4.22	3.4	0.39215686
840	0.233333333	4.17	3.45	0.35947712
900	0.25	4.12	3.5	0.32679739
960	0.266666667	4.09	3.53	0.30718954
1020	0.283333333	4.05	3.57	0.28104575
1080	0.3	4.02	3.6	0.26143791
1140	0.316666667	3.99	3.63	0.24183007
1200	0.333333333	3.96	3.66	0.22222222
1260	0.35	3.93	3.69	0.20261438
1320	0.366666667	3.91	3.71	0.18954248
1380	0.383333333	3.89	3.73	0.17647059
1440	0.4	3.87	3.75	0.16339869
1500	0.416666667	3.85	3.77	0.1503268
1560	0.433333333	3.84	3.78	0.14379085
1620	0.45	3.82	3.8	0.13071895
1680	0.466666667	3.81	3.81	0.12418301
1740	0.483333333	3.8	3.82	0.11764706
1800	0.5	3.79	3.83	0.11111111
2100	0.583333333	3.75	3.87	0.08496732
2400	0.666666667	3.73	3.89	0.07189542
2700	0.75	3.72	3.9	0.06535948
3000	0.833333333	3.71	3.91	0.05882353
3300	0.916666667	3.705	3.915	0.05555556
3600	1	3.7	3.92	0.05228758
3900	1.083333333	3.7	3.92	0.05228758
4200	1.166666667	3.7	3.92	0.05228758
4500	1.25	3.7	3.92	0.05228758
4800	1.333333333	3.7	3.92	0.05228758

0	0.37		
1000	0.37		

From chart, T_o = 0.225

$K = \frac{r^2 \ln(L/R)}{2LT_0}$ = 0.0014405 m²/(m·h)

K = 4.001E-05 cm/s

Sensitivity Analysis

From chart, T_o = 0.252 Low

$K = \frac{r^2 \ln(L/R)}{2LT_0}$ = 0.0012862 m²/(m·h)

K = 3.573E-05 cm/s

From chart, T_o = 0.218 High

$K = \frac{r^2 \ln(L/R)}{2LT_0}$ = 0.0014868 m²/(m·h)

K = 4.13E-05 cm/s

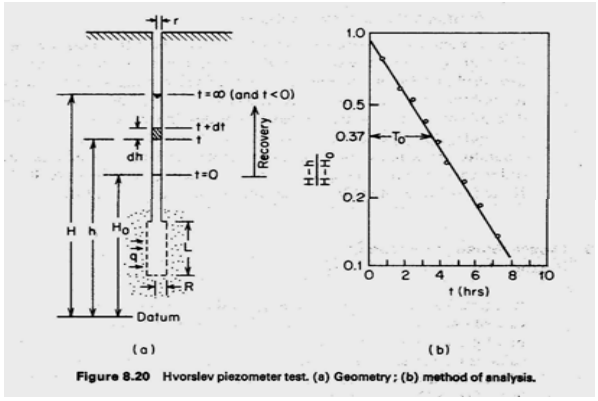
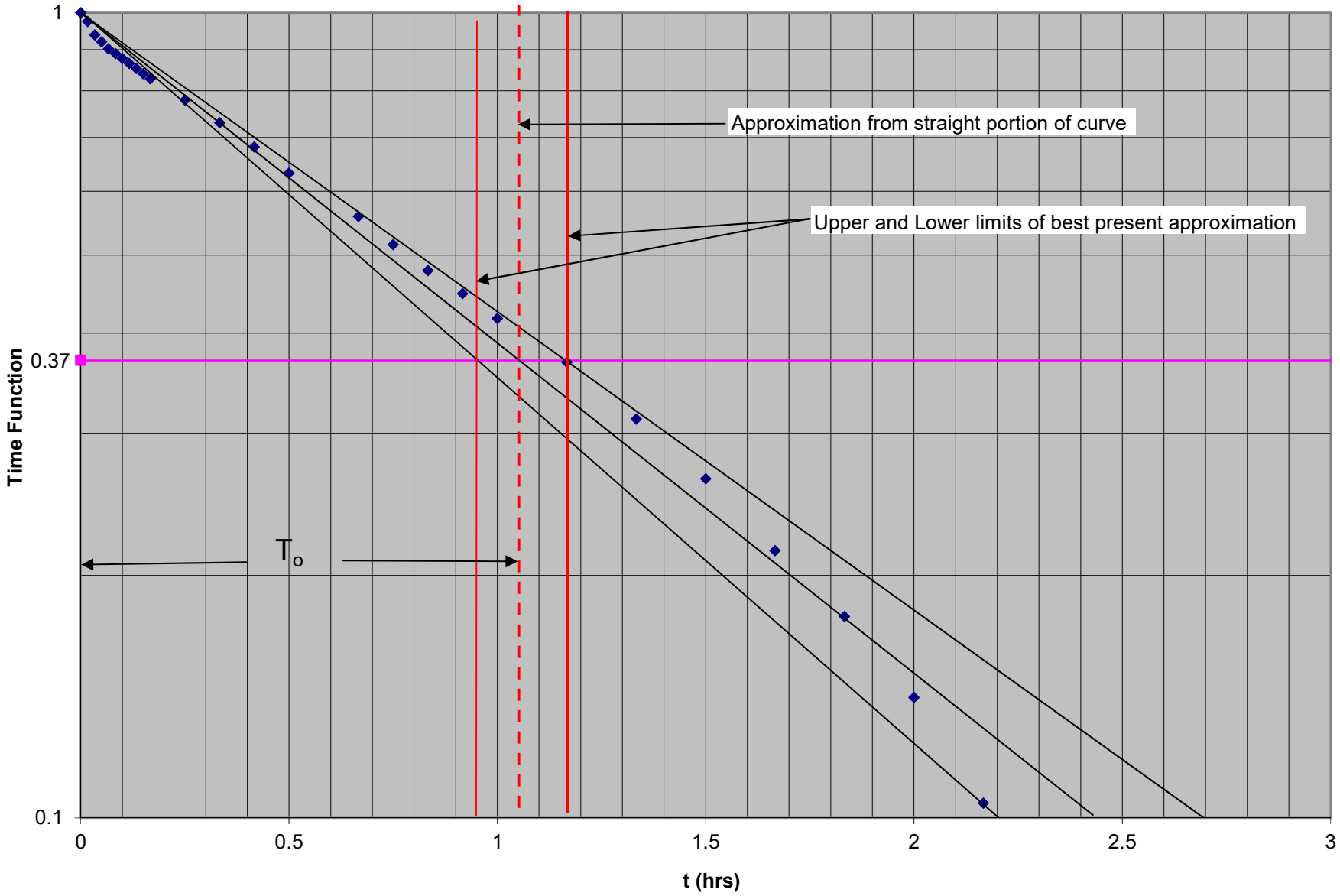


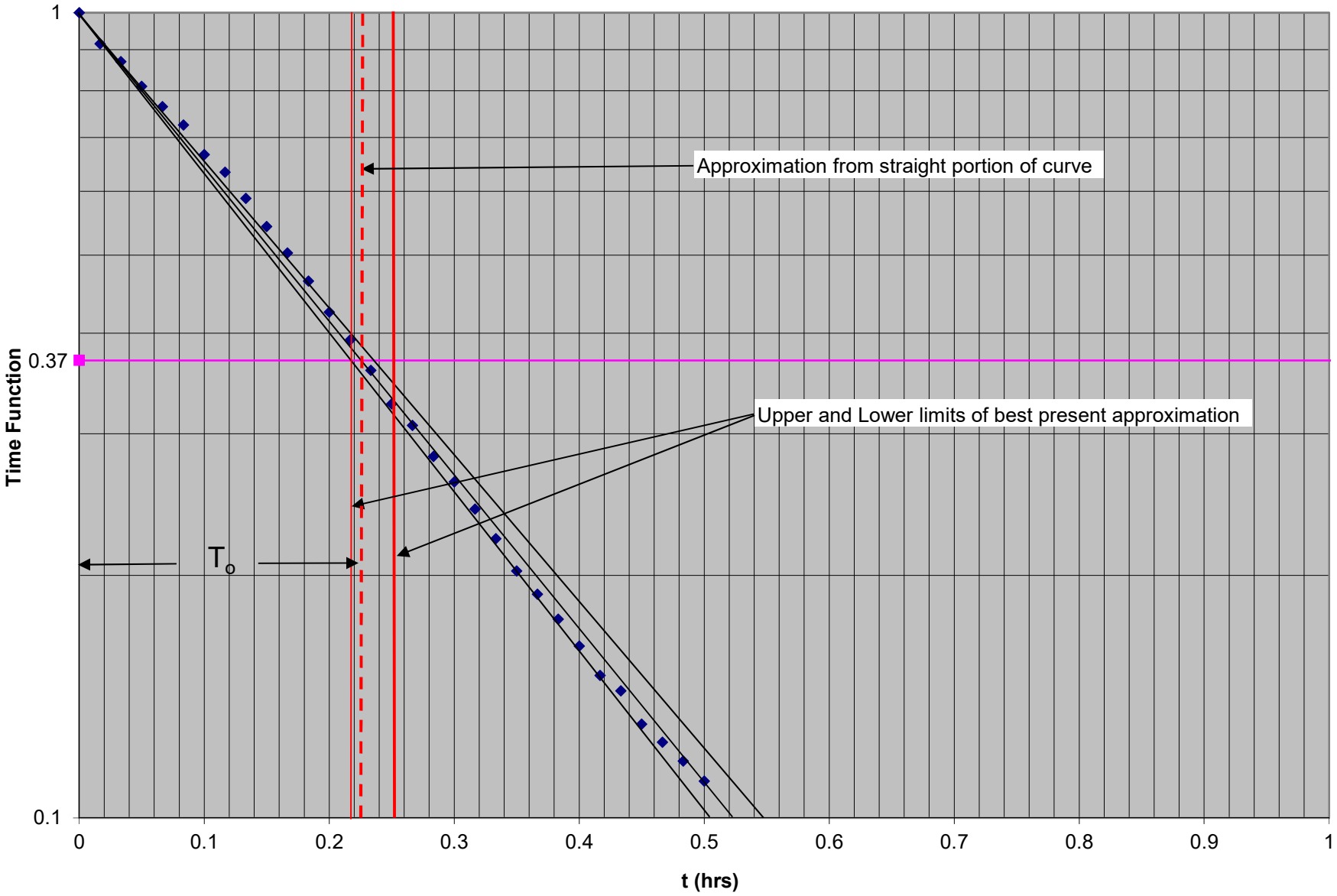
Figure 8.20 Hvorslev piezometer test. (a) Geometry; (b) method of analysis.

ChartMW1



\\pinchin.com\ott\Job\230000s\0230236.000 GWL,320McRaeAve,EDR,PRJMT\0230236.008 GWL,320McRaeAve,GEO,CONS\Field Notes\Hvorslev Test Well

ChartMW5


1
0.37
0.1
Time Function
Approximation from straight portion of curve
Upper and Lower limits of best present approximation
T_o
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
t (hrs)

APPENDIX IV
Groundwater Data

TABLE 1.
Sanitary and Storm Effluent Analysis
320 McRae Avenue, Ottawa, Ontario
Pinchin Project # 230236.008

Parameter	Units	Sample (Location / Date)	Sewer Use By-Law Ottawa Sanitary and Combined Sewer Discharge	Sewer Use By-Law Ottawa Sewer Discharge
		MW1 Sample 1 27-Feb-20		
General Inorganics				
CBOD	mg/L	3	300 mg/L	25 mg/L
Cyanide, total	mg/L	ND (0.01)	2 mg/L	0.02 mg/L
pH	pH Units	7.2	N/V	NV
Phenolics	mg/L	**0.012**	1 mg/L	0.008 mg/L
Phosphorus, total	mg/L	0.11	10 mg/L	0.4 mg/L
Total Suspended Solids	mg/L	**147**	350 mg/L	15 mg/L
Sulphide	mg/L	**3.56**	2 mg/L	NV
Total Kjeldahl Nitrogen	mg/L	1.1	100 mg/L	NV
Anions				
Fluoride	mg/L	0.6	10 mg/L	NV
Sulphate	mg/L	196	1500 mg/L	NV
Metals				
Aluminum	mg/L	0.53	50 mg/L	NV
Antimony	mg/L	0.003	5 mg/L	NV
Arsenic	mg/L	ND (0.01)	1 mg/L	0.02 mg/L
Bismuth	mg/L	ND (0.005)	5 mg/L	NV
Boron	mg/L	0.2	25 mg/L	NV
Cadmium	mg/L	ND (0.001)	0.02 mg/L	0.008 mg/L
Chromium	mg/L	ND (0.05)	5 mg/L	0.08 mg/L
Cobalt	mg/L	0.001	5 mg/L	NV
Copper	mg/L	ND (0.005)	3 mg/L	0.04 mg/L
Lead	mg/L	ND (0.001)	5 mg/L	0.12 mg/L
Manganese	mg/L	**0.36**	5 mg/L	0.05 mg/L
Mercury	mg/L	ND (0.0001)	0.001 mg/L	0.0004 mg/L
Molybdenum	mg/L	ND (0.005)	5 mg/L	NV
Nickel	mg/L	ND (0.005)	3 mg/L	0.08 mg/L
Selenium	mg/L	ND (0.005)	5 mg/L	0.02 mg/L
Silver	mg/L	ND (0.001)	5 mg/L	0.12 mg/L
Tin	mg/L	ND (0.01)	5 mg/L	NV
Titanium	mg/L	ND (0.01)	5 mg/L	NV
Vanadium	mg/L	0.003	5 mg/L	NV
Zinc	mg/L	ND (0.02)	3 mg/L	0.04 mg/L
Volatiles				
Benzene	mg/L	**0.0691**	0.01 mg/L	0.002 mg/L
Bromodichloromethane	mg/L	ND (0.0005)	0.35 mg/L	NV
Bromoform	mg/L	ND (0.0005)	0.63 mg/L	NV
Bromomethane	mg/L	ND (0.0005)	0.11 mg/L	NV
Carbon Tetrachloride	mg/L	ND (0.0002)	0.057 mg/L	NV
Chlorobenzene	mg/L	ND (0.0005)	0.057 mg/L	NV
Chloroethane	mg/L	ND (0.0010)	0.27 mg/L	NV
Chloroform	mg/L	ND (0.0005)	0.08 mg/L	0.002 mg/L
Chloromethane	mg/L	ND (0.0030)	0.19 mg/L	NV
Dibromochloromethane	mg/L	ND (0.0005)	0.057 mg/L	NV
Ethylene dibromide (dibromoethane, 1,2-	mg/L	ND (0.0002)	0.028 mg/L	NV
1,2-Dichlorobenzene	mg/L	ND (0.0005)	0.088 mg/L	0.0056 mg/L
1,3-Dichlorobenzene	mg/L	ND (0.0005)	0.036 mg/L	NV
1,4-Dichlorobenzene	mg/L	ND (0.0005)	0.017 mg/L	0.0068 mg/L
1,1-Dichloroethane	mg/L	ND (0.0005)	0.2 mg/L	NV
1,2-Dichloroethane	mg/L	ND (0.0005)	0.21 mg/L	NV
1,1-Dichloroethylene	mg/L	ND (0.0005)	0.04 mg/L	NV
cis-1,2-Dichloroethylene	mg/L	ND (0.0005)	0.2 mg/L	0.0056 mg/L
trans-1,2-Dichloroethylene	mg/L	ND (0.0005)	0.2 mg/L	NV
1,2-Dichloropropane	mg/L	ND (0.0005)	0.85 mg/L	NV
cis-1,3-Dichloropropylene	mg/L	ND (0.0005)	0.07 mg/L	NV
trans-1,3-Dichloropropylene	mg/L	ND (0.0005)	0.07 mg/L	0.0056 mg/L
Ethylbenzene	mg/L	**0.307**	0.057 mg/L	0.002 mg/L
Methylene Chloride	mg/L	ND (0.0050)	0.211 mg/L	0.0052 mg/L
Styrene	mg/L	ND (0.0005)	0.04 mg/L	NV
1,1,2,2-Tetrachloroethane	mg/L	ND (0.0005)	0.04 mg/L	0.017 mg/L
Tetrachloroethylene	mg/L	ND (0.0005)	0.05 mg/L	0.0044 mg/L
Toluene	mg/L	**0.0065**	0.08 mg/L	0.002 mg/L
1,1,1-Trichloroethane	mg/L	ND (0.0005)	0.054 mg/L	NV
1,1,2-Trichloroethane	mg/L	ND (0.0005)	0.8 mg/L	NV
Trichloroethylene	mg/L	ND (0.0005)	0.054 mg/L	0.0076 mg/L
Trichlorofluoromethane	mg/L	ND (0.0010)	0.02 mg/L	NV
1,3,5-Trimethylbenzene	mg/L	**0.0081**	0.003 mg/L	NV
Vinyl Chloride	mg/L	ND (0.0005)	0.4 mg/L	NV
Xylenes, total	mg/L	**0.0092**	0.32 mg/L	0.0044 mg/L
Hydrocarbons				
Oil & Grease, animal/vegetable	mg/L	ND (0.500)	150 mg/L	NV
Oil & Grease, mineral/synthetic	mg/L	0.7	15 mg/L	NV
Oil & Grease, total	mg/L	0.7	N/V	NV
Semi-Volatiles				
Benzylbutylphthalate	mg/L	ND (0.001)	0.017 mg/L	NV
BIS(2-Chloroethoxy)methane	mg/L	ND (0.001)	0.036 mg/L	NV
Bis(2-ethylhexyl)phthalate	mg/L	ND (0.001)	0.28 mg/L	NV
Diethylphthalate	mg/L	ND (0.001)	0.2 mg/L	NV
Di-n-butylphthalate	mg/L	ND (0.001)	0.057 mg/L	NV
Di-n-octylphthalate	mg/L	ND (0.001)	0.03 mg/L	NV
Indole	mg/L	ND (0.001)	0.05 mg/L	NV
2,4-Dichlorophenol	mg/L	ND (0.001)	0.044 mg/L	NV
Pesticides, OC				
Hexachlorobenzene	mg/L	ND (0.00001)	NV	NV
Polychlorinated Biphenyls				
PCBs	ug/L	ND (0.05)	300 mg/L	0.0004 mg/L (0.4 ug/L)

Notes:

All units in mg/L and ug/L

NV = No Value

N/A = Not Analyzed

< = Reported Concentration Below Method Detection Limit (Non-Detect)

500 Parameter Concentration Exceeds "By-Law Number 2003-514, A By-Law to Provide for the Regulation of Waste Water Services and Waste Discharges to Municipal Sewers for The City of Ottawa" dated January, 2003, Schedule A, Table 1. Limits for Sanitary and Combined Sewers Discharge.

500 Parameter Concentration Exceeds "By-Law Number 2003-514, A By-Law to Provide for the Regulation of Waste Water Services and Waste Discharges to Municipal Sewers for The City of Ottawa" dated January 2003, Schedule A, Table 2. Limits for Storm Sewer Discharge

500 Parameter exceed both Limits for Sanitary and Combined Sewer Discharge/Limits for Storm Sewer Discharge as defined above.

MEMORANDUM

DATE: December 4, 2020

MEMO TO: John Wu – City of Ottawa

COPIES TO: Ashley Burke – GWLRA, Andrew Hanna – GWLRA, Farzi Jalali - GWLRA

FROM: Rob MacKenzie - Pinchin

RE: Remedial Plan for Addressing Groundwater Impacts at 320 McRae Avenue, 1976 Scott Street, 311 and 315 Tweedsmuir Avenue

PINCHIN FILE: 230236.006

This memorandum provides supplementary information for the Phase Two Environment Site Assessment (ESA) completed by Pinchin Ltd. (Pinchin) at the 320 McRae Avenue, 1976 Scott Street, 311 and 315 Tweedsmuir Avenue property (Site). The Phase Two ESA findings are provided in the draft report prepared by Pinchin titled *"Phase Two Environmental Site Assessment, 320 McRae Avenue, 1976 Scott Street, 311 and 315 Tweedsmuir Avenue, Ottawa, Ontario"* and dated November 30, 2020. The Phase Two ESA was completed to support the filing of a Record of Site Condition (RSC) for the proposed redevelopment of the Site.

1.0 BACKGROUND

The Phase Two ESA identified groundwater at monitoring well MW-1 with concentrations of benzene, ethylbenzene, petroleum hydrocarbons in the F1 to F3 fractions and naphthalene exceeding the applicable Ontario Ministry of the Environment, Conservation and Parks (MECP) Table 7 Site Condition Standards (*Table 7 Standards*) and/or the City of Ottawa's criteria for discharge to the sanitary sewer (*Sanitary Discharge Criteria*). These parameters are collectively referred to as the contaminants of concern (COCs). The location of MW-1 and the estimated areal extent of groundwater impacts in the vicinity of this well are shown on the attached Figure 1. Monitoring well MW-1 was installed to the intersect the shallow water table at the Site and screened between 4.5 and 7.6 metres below ground surface (mbgs). A deep monitoring well (MW201), screened between 12.1 and 15.2 mbgs, was also installed to vertically delineate the groundwater impacts at MW-1. Groundwater samples collected from MW201 met the *Table 7 Standards*, confirming that the impacts were confined to the shallow groundwater zone at MW-1.

2.0 REMEDIAL PLAN

Given that the identified groundwater impacts are limited to the shallow groundwater zone in a relatively small area measuring approximately 20 metres by 20 metres in the northern portion of the Site, Pinchin plans to undertake groundwater remedial activities comprised of the following:

- Excavate the impacted groundwater area to a depth of approximately 8 mbgs (i.e., the planned depth for the underground parking garage);
- Using a vacuum truck and/or an on-Site tanker, extract groundwater from the excavation for a period of approximately two weeks. The frequency of pumping will depend on the rate of recharge but based on previous hydrogeological testing at the Site, it is expected that daily pumping of the excavation will be completed. The contaminated groundwater will be disposed of off-Site at a licenced liquid waste disposal facility;
- Collect groundwater samples from the excavation to monitor remedial progress; and
- Compare the results of the groundwater samples to the *Table 7 Standards* and the *Sanitary Discharge Criteria*.

The goal of the groundwater remediation program will be to remediate the groundwater to meet the *Table 7 Standards*. If this is achieved, then the groundwater will also be suitable for discharge to the sanitary sewer given that the *Sanitary Discharge Criteria* are less stringent than the *Table 7 Standards* for the COCs. Depending on the effectiveness of groundwater remediation, one of the following scenarios will occur.

Scenario 1

Should the groundwater remediation program successfully reduce COC levels to meet the *Table 7 Standards*, then the groundwater can be discharged to the sanitary sewer without treatment. Remediating the groundwater to meet the *Table 7 Standards* will also mean that a risk assessment will not be required to support the filing of the RSC for the Site, which can be filed following the excavation of previously identified impacted soil that will be removed as part of Site redevelopment. It is important to note that the MECP requires post-remediation groundwater monitoring consisting of two quarterly groundwater sampling events with results meeting the *Table 7 Standards* before the RSC can be filed. This will require the installation of new monitoring wells in the remediated area as the MECP will not allow excavation grab samples to be used to verify that remediation has been achieved.

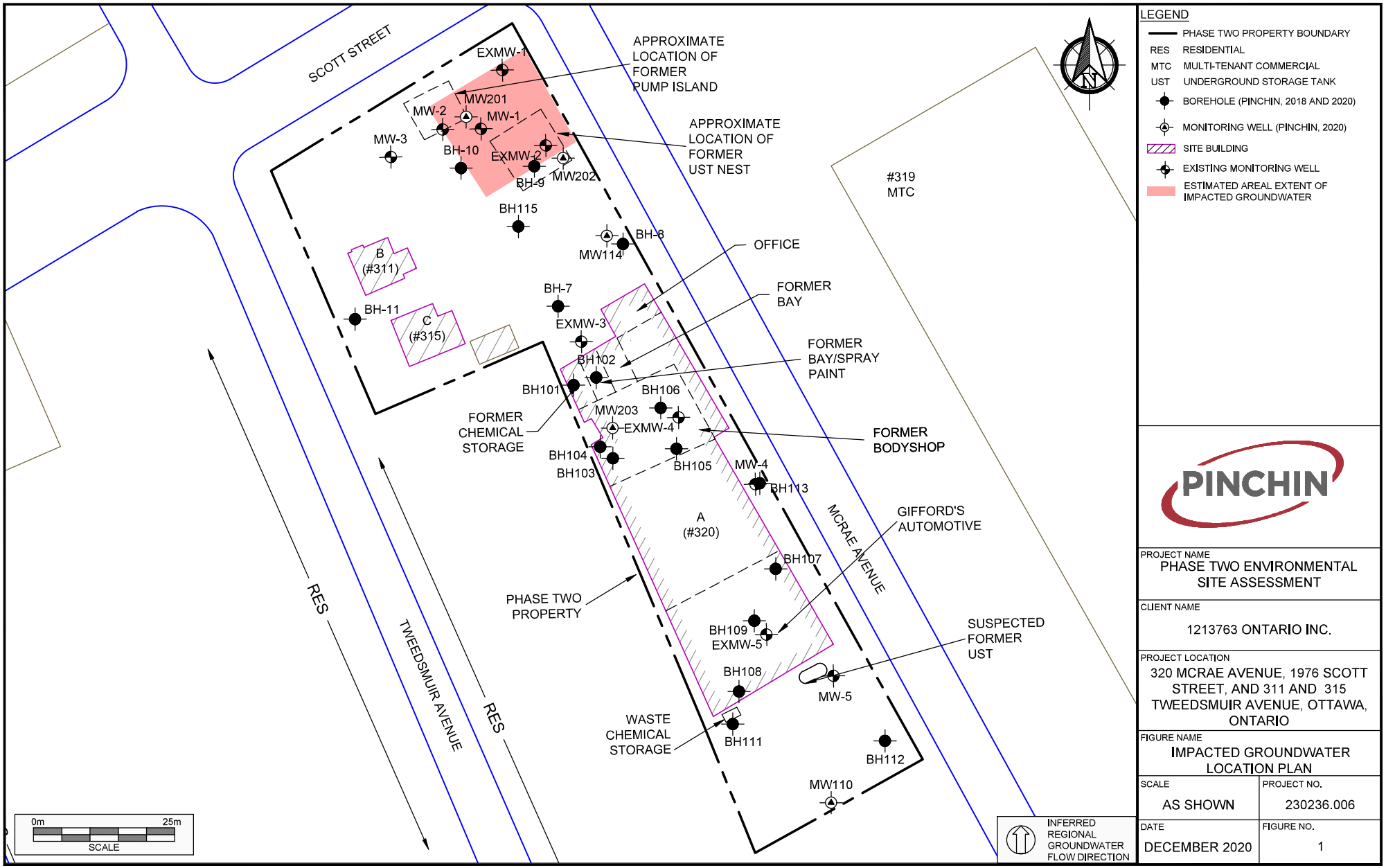
Scenario 2

Should the groundwater remediation program not be successful in lowering the COC levels to below the *Table 7 Standards* but the *Sanitary Discharge Criteria* are met, then the groundwater can be discharged to the sanitary sewer without treatment. A risk assessment (RA) will be required to address the residual groundwater impacts not meeting the *Table 7 Standards* through the development of Property Specific Standards (PSS) before the RSC can be filed. The RA will include risk management measures intended to be protective of building occupants and the environment.

Scenario 3

In the event that the groundwater remediation program is not successful in reducing COC levels to meet the *Sanitary Discharge Criteria*, then treatment of the impacted groundwater before discharge to the sanitary sewer will be required during post-construction operation of the building. However, given that the amount of impacted groundwater at the Site is finite and much of it will be removed through the excavation for the underground parking garage and the remedial activities described above, it is expected that post-construction groundwater treatment will not be a permanent requirement and that the treatment system can be removed from the Site once influent monitoring to the system shows that the groundwater originating from the foundation drainage system meets the *Sanitary Discharge Criteria*. It is anticipated that a mobile treatment unit (MTU) will be used for the groundwater treatment that will receive influent from the foundation drainage and discharge the treated groundwater to the sanitary sewer. The design of the MTU will depend in part on the residual groundwater concentrations and the volume of groundwater derived from the drainage system which will be reassessed after the building is constructed, and will meet all required codes. Given that the COCs are petroleum hydrocarbon-related parameters, the impacted groundwater will be treated by passing it through activated carbon cylinders and no air emissions will be generated. As per Scenario 2, an RA will be needed to develop PSS before filing the RSC.

I trust that the above is sufficient to explain Pinchin's plan to address the groundwater impacts at the Site. Should you have any questions or require additional information, feel free to contact me.



LEGEND

- PHASE TWO PROPERTY BOUNDARY
- RES RESIDENTIAL
- MTC MULTI-TENANT COMMERCIAL
- UST UNDERGROUND STORAGE TANK
- BOREHOLE (PINCHIN, 2018 AND 2020)
- MONITORING WELL (PINCHIN, 2020)
- SITE BUILDING
- EXISTING MONITORING WELL
- ESTIMATED AREAL EXTENT OF IMPACTED GROUNDWATER

PINCHIN

PROJECT NAME
PHASE TWO ENVIRONMENTAL SITE ASSESSMENT

CLIENT NAME
1213763 ONTARIO INC.

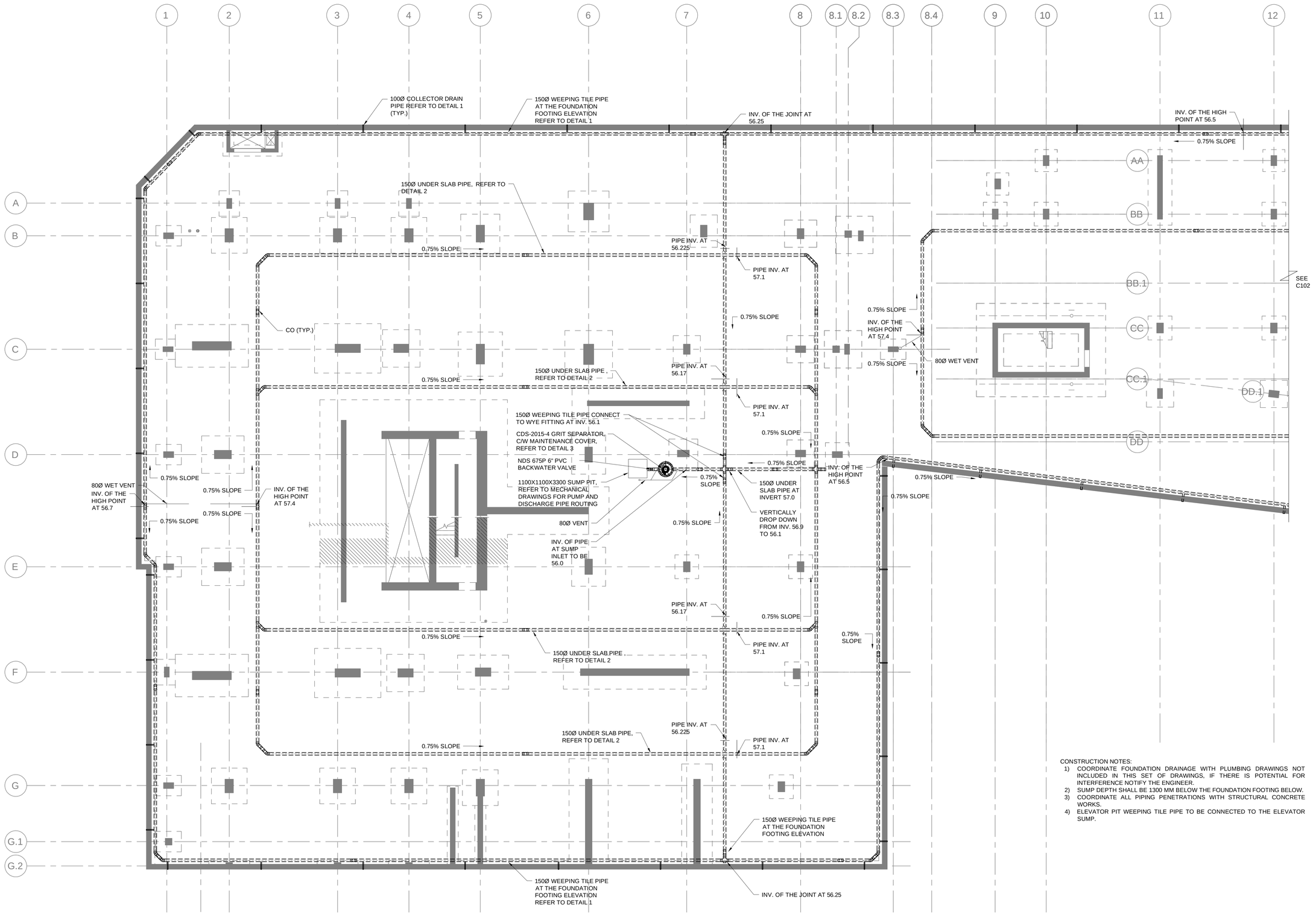
PROJECT LOCATION
320 MCRAE AVENUE, 1976 SCOTT STREET, AND 311 AND 315 TWEEDSMUIR AVENUE, OTTAWA, ONTARIO

FIGURE NAME
IMPACTED GROUNDWATER LOCATION PLAN

SCALE	PROJECT NO.
AS SHOWN	230236.006
DATE	FIGURE NO.
DECEMBER 2020	1

APPENDIX 5

Foundation Drainage Design Drawings



1 C101 BASEMENT 2 FOUNDATION DRAIN PLAN VIEW

1 : 100 1m 0 2m 4m @M100

CONSTRUCTION NOTES:

1) COORDINATE FOUNDATION DRAINAGE WITH PLUMBING DRAWINGS NOT INCLUDED IN THIS SET OF DRAWINGS, IF THERE IS POTENTIAL FOR INTERFERENCE NOTIFY THE ENGINEER.
2) SUMP DEPTH SHALL BE 1300 MM BELOW THE FOUNDATION FOOTING BELOW.
3) COORDINATE ALL PIPING PENETRATIONS WITH STRUCTURAL CONCRETE WORKS.
4) ELEVATOR PIT WEEPING TILE PIPE TO BE CONNECTED TO THE ELEVATOR SUMP.

[redacted]

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STRUCTURE Structural
Adjeleian Allen Rubeli Ltd. (AAR)
75 Albert Street, Suite 1005
Ottawa, ON K1P 5E7
T 613 232 5786

ARCHITECTURE DE PAYSAGE Landscape Architect
James B. Lennox & Associates Inc.
3532 Carling Ave.
Ottawa, ON K2H 5A8
T 613 722 5168

CIVIL Civil
R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1844

HYDRAULIQUE/ÉLECTRIQUE Hydraulic/Electrical
Goodkey, Weedmark & Associates Limited (GWAL)
1688 Woodward Drive
Ottawa, ON K2C 3R8
T 613 727 5111

URBANISTE Urban Planner
Fotenn Planning + Design
396 Cooper Street - Suite 300
Ottawa, ON, K2P 2H7
T 613 730 5709

ARCHITECTES Architect
NEUF architect(e)s SENCRL
630, boul. René-Lévesque O. 32e étages, Montréal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

SCEAU / Seal

LICENSED PROFESSIONAL ENGINEER
B. J. MAJANO
100127839
RVA# 205187
Mar 09 2021
PROVINCE OF ONTARIO

R.V. Anderson Associates Limited
engineering • environment • infrastructure

PROJECT#: 194453

SHEET PRINT DATE: 10/12/2020 3:44:49 PM

CLIENT Client

GWL REALTY ADVISORS

OUVRAGE Project

320 McRAE AVENUE

EMPLACEMENT Location
1976 SCOTT ST & 320 McRAE AVE
OTTAWA, ON

NO PROJET No.

NO	REVISION	DATE
	ISSUED FOR BUILDING PERMIT	

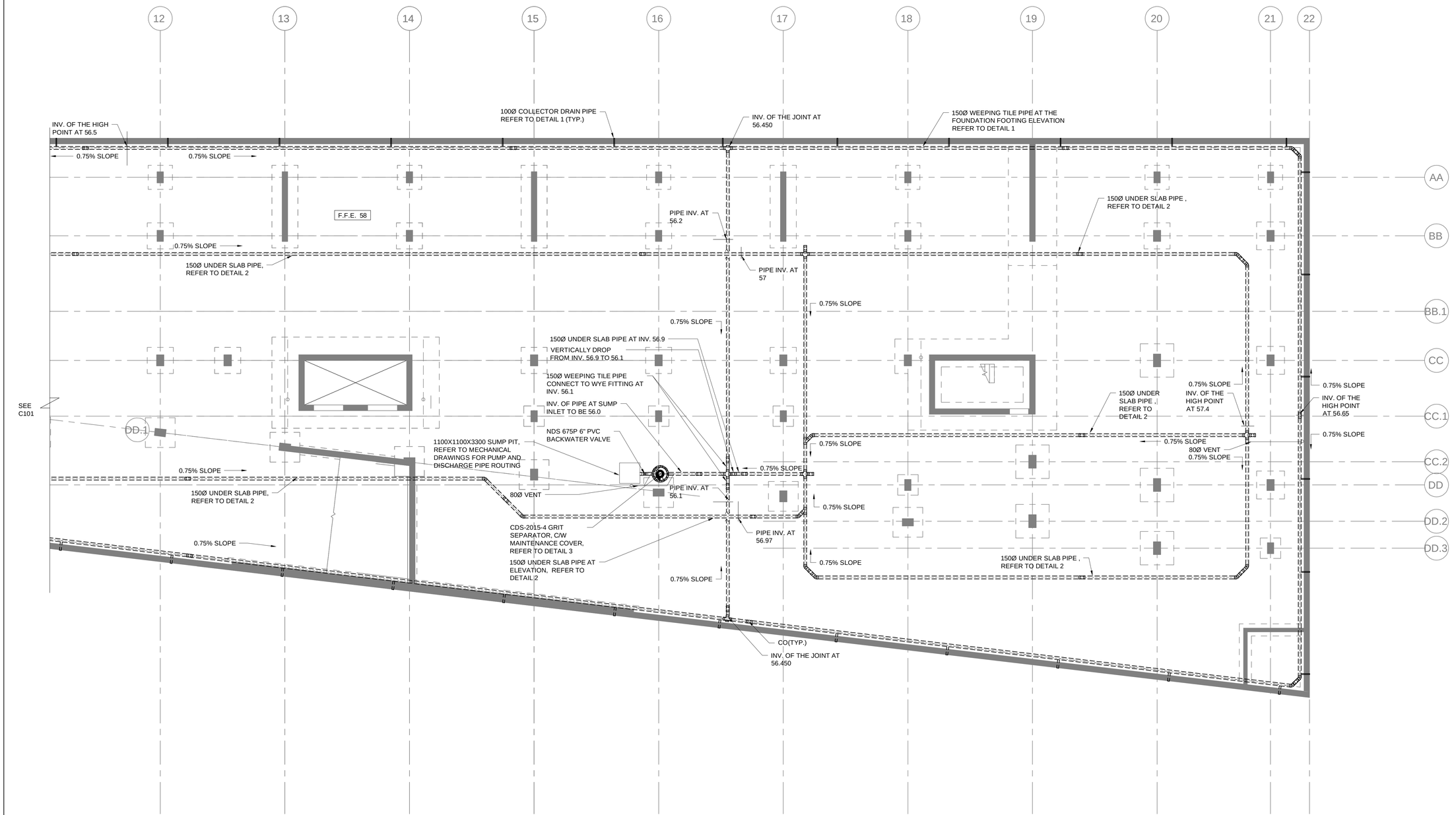
Preliminary DO NOT USE FOR CONSTRUCTION

DESSINÉ PAR Drawn by: KP
VÉRIFIÉ PAR Checked by: BJM
DATE: 12/24/20
ÉCHELLE Scale: AS SHOWN

TITRE DU DESSIN Drawing Title

FOUNDATION DRAIN PLAN VIEW 1

RÉVISION Revision: **0**
NO. DESSIN Dwg Number: **C101**



1 C102 BASEMENT 2 FOUNDATION DRAIN PLAN VIEW
1 : 100

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R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
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Goodkey, Weedmark & Associates Limited (GWAL)
1688 Woodward Drive
Ottawa, ON K2C 3R8
T 613 727 5111

URBANISTE Urban Planner
Fotenn Planning + Design
396 Cooper Street - Suite 300
Ottawa, ON, K2P 2H7
T 613 730 5709

ARCHITECTES Architect
NEUF architect(e)s SENCRL
630, boul. René-Lévesque O. 32e étages, Montréal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

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B. J. MAJANO
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ÉCHELLE Scale: AS SHOWN

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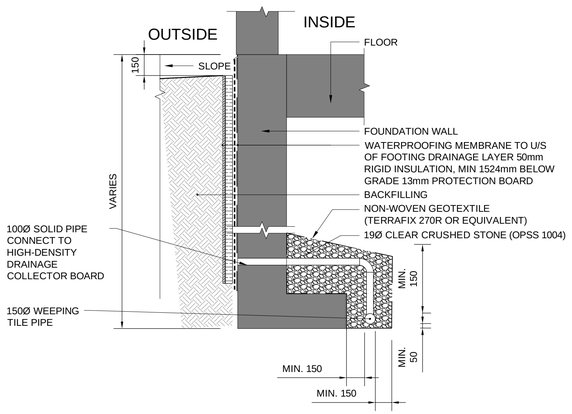
RÉVISION Revision: 0
NO. DESSIN Dwg Number: C102



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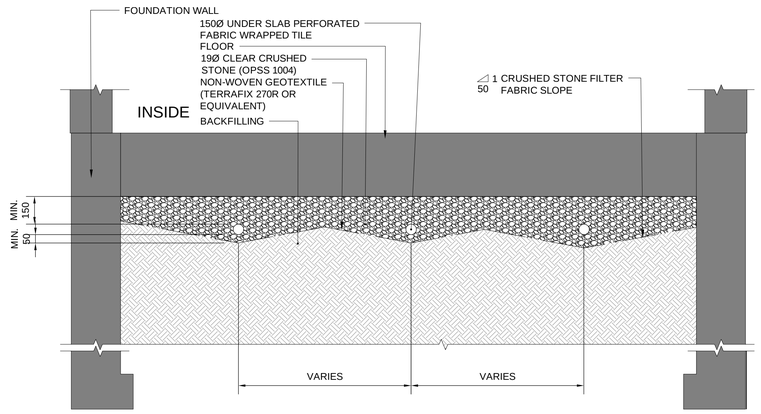

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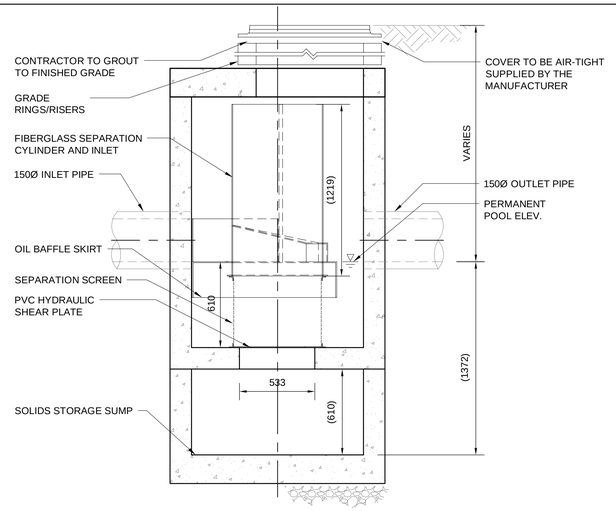


3 C103 DETAIL 3
NTS

PIPING SPECIFICATION

PART 1 - GENERAL

1.01 WORK INCLUDED:

A. PROVIDE BURIED OR IN SLAB GRAVITY PVC PRESSURE SEWER PIPE INCLUDING VENT AND FITTINGS AS SHOWN ON THE DRAWINGS AND SPECIFIED HEREIN.

B. PROVIDE RELATED GASKETS, FASTENERS, ANCHORS, AND SUPPORTS.

C. COMPLY WITH THE ONTARIO BUILDING CODE FOR MATERIAL, SUPPLY AND INSTALLATION.

1.02 SUBMITTALS:

A. SUBMIT SHOP DRAWINGS OF PIPING GIVING DETAILS, DIMENSIONS, INVERTS, LAYOUTS, JOINT TYPES, METHOD OF CONSTRUCTION, PIPE SUPPORTS, SPACING TYPE AND ALL PRODUCTS SPECIFIED PRIOR TO ORDERING OF ANY ITEM.

PART 2 - PRODUCTS

2.01 PVC SEWER PIPE AND FITTINGS:

A. PIPE: 100 - 900 MM DIAMETER PVC RING-TITE GASKETED JOINTED PIPE EQUAL TO SCHEDULE 40 PRESSURE PIPE, AND MEET THE ASTM D1785M AND ASTM D2655 DWV FITTINGS STANDARDS.

B. PIPE 80-100 MM DIAMETER PVC FOR VENT, AND MEET THE CSA B182.1 STANDARDS.

C. FITTINGS: 100 -900 MM DIAMETER, INJECTION MOLDED GASKETED PVC FITTINGS EQUAL TO IPEX DR35 FABRICATED FITTINGS, AND MEET THE CSA B182.1, CSA 182.2, ASTM F679 STANDARDS

D. JOINTS: SEALING GASKETS SHALL MEET THE CSA 182.2, ASTM F477 STANDARDS.

E. PERFORATIONS: THE PIPE SHALL HAVE A 2 ROWS OF 12.5MM DIAMETER HOLES , 160 DEGREE, 125 MM APART ON CENTERED

2.02 CLEAN OUTS:

A. SUPPLIERS:

1. ZURN DRAINAGE PRODUCTS ARE SPECIFIED. EQUIVALENT ENPOCO / ANCON OR J.R. SMITH WILL BE CONSIDERED EQUAL.

B. CLEANOUTS IN EXTERIOR GROUND LEVEL SHALL BE ZURN ZN-1400-SZ1-DP WITH DURA COATED CAST IRON BODY, WITH GAS AND WATER TIGHT PLUG, STAINLESS STEEL DECK PLATE AND TOP ASSEMBLY.

C. CLEANOUTS IN FLOORS SHALL BE ZURN ZN-1400-2 WITH DURA COATED CAST IRON BODY, ADJUSTABLE COLLAR, NON-SEIZING SEALING PLUG, T-HANDLE SERVICING WRENCH AND ROUND SOLID NICKEL BRONZE FRAME AND COVER TO SUIT FLOOR FINISHING MATERIAL.

D. ACCESS COVERS IN WALLS SHALL BE ZURN Z-1403 WITH ROUND STAINLESS-STEEL WALL ACCESS COVER WITH CENTER SECURING SCREW, FOR PIPE SIZES AS INDICATED.

PART 3 - EXECUTION

3.01 INSTALLATION:

A. BEDDING CLASSIFICATION TO CONFORM TO ASTM D-2321 - "RECOMMENDED PRACTICE FOR UNDERGROUND INSTALLATION OF FLEXIBLE THERMO PLASTIC SEWER PIPE".

B. USE MANUFACTURER SUPPLIED LUBRICANT AND GUIDELINES FOR JOINTING THE GASKETED PIPE SECTIONS. KEEP JOINTS FREE OF DIRT.

C. SELECT INITIAL BACKFILL AND HAND TAMP FIRST 300 MM BEFORE FINAL BACKFILLING IS DONE BY MACHINE.

D. COORDINATE PLUMBING INSPECTOR REVIEW PRIOR TO BACKFILL.

E. CLEANOUTS SHALL BE INSTALLED AS FOLLOWS:

1. AS SHOWN ON THE DRAWINGS.
2. WHERE A DRAIN CHANGES DIRECTION BY MORE THAN 45 DEGREES.
3. AT THE BASE OF EACH WASTE, SOIL STACK AND RAINWATER LEADER.
4. WHERE NOT SHOWN ON THE CONTRACT DRAWINGS BUT REQUIRED BY ANY OF THE OBC

3.02 SLOPE:

A. SLOPE DRAINS NOT LESS THAN THE MINIMUM SLOPE INDICATED ON THE DRAWINGS.

3.03 FIELD TESTING:

A. CARRY OUT A BALL TEST ON A SANITARY BUILDING DRAIN/SEWER AND A STORM BUILDING DRAIN/SEWER PIPING OF 100 MM IN SIZE OR LARGER BURIED UNDERGROUND.

B. CONSIDER PIPING IS PERFORATED FOR TESTING.

GRIT SEPARATOR SPECIFICATION

PART 1 - GENERAL

1.01 WORK INCLUDED:

A. THIS ITEM SHALL GOVERN THE FURNISHING AND INSTALLATION OF THE GRIT SEPARATOR, COMPLETE AND OPERABLE AS SHOWN AND AS SPECIFIED HEREIN, IN ACCORDANCE WITH THE REQUIREMENTS OF THE PLANS AND CONTRACT DOCUMENTS.

B. THE CONTRACTOR SHALL FURNISH ALL LABOR, EQUIPMENT AND MATERIALS NECESSARY TO INSTALL THE STORM WATER TREATMENT DEVICE(S) (SWTD) AND APPURTENANCES SPECIFIED IN THE DRAWINGS AND THESE SPECIFICATIONS.

1.02 SUBMITTALS:

A. SUBMIT SHOP DRAWINGS FOR PRODUCTS SPECIFIED PRIOR TO ORDERING OF ANY ITEM.

PART 2 - PRODUCTS

2.01 HOUSING UNIT OF STORMWATER TREATMENT DEVICE SHALL BE CONSTRUCTED OF PRE-CAST OR CAST-IN-PLACE CONCRETE, NO EXCEPTIONS. PRECAST CONCRETE COMPONENTS SHALL CONFORM TO APPLICABLE SECTIONS OF ASTM C 478, ASTM C 857 AND ASTM C 858 AND THE FOLLOWING:

A. CONCRETE SHALL ACHIEVE A MINIMUM 28-DAY COMPRESSIVE STRENGTH OF 4,000 POUNDS PER SQUARE-INCH (PSI).

B. UNLESS OTHERWISE NOTED, THE PRECAST CONCRETE SECTIONS SHALL BE DESIGNED TO WITHSTAND LATERAL EARTH AND AASHTO H-20 TRAFFIC LOADS.

C. CEMENT SHALL BE TYPE III PORTLAND CEMENT CONFORMING TO ASTM C 150.

D. AGGREGATES SHALL CONFORM TO ASTM C 33.

E. REINFORCING STEEL SHALL BE DEFORMED BILLET-STEEL BARS, WELDED STEEL WIRE OR DEFORMED WELDED STEEL WIRE CONFORMING TO ASTM A 615, A 185, OR A 497.

F. JOINTS SHALL BE SEALED WITH PREFORMED JOINT SEALING COMPOUND CONFORMING TO ASTM C 990.

G. SHIPPING OF COMPONENTS SHALL NOT BE INITIATED UNTIL A MINIMUM COMPRESSIVE STRENGTH OF 4,000 PSI IS ATTAINED OR FIVE (5) CALENDAR DAYS AFTER FABRICATION HAS EXPIRED, WHICHEVER OCCURS FIRST.

2.02 INTERNAL COMPONENTS AND APPURTENANCES SHALL CONFORM TO THE FOLLOWING:

A. SCREEN AND SUPPORT STRUCTURE SHALL BE MANUFACTURED OF TYPE 316 AND 316L STAINLESS STEEL CONFORMING TO ASTM F 1267-01.

B. HARDWARE SHALL BE MANUFACTURED OF TYPE 316 STAINLESS STEEL CONFORMING TO ASTM A 320.

C. FIBERGLASS COMPONENTS SHALL CONFORM TO THE ASTM D-4097.

D. ACCESS SYSTEM(S) CONFORM TO THE FOLLOWING:

D.A. MANHOLE CASTINGS SHALL BE DESIGNED TO WITHSTAND AASHTO H-20 LOADINGS AND MANUFACTURED OF CAST-IRON CONFORMING TO ASTM A 48 CLASS 30.

2.03 PERFORMANCE:

A. THE SWTD SHALL BE SIZED TO EITHER ACHIEVE AN 80 PERCENT AVERAGE ANNUAL REDUCTION IN THE TOTAL SUSPENDED SOLID LOAD OR TREAT A FLOW RATE DESIGNATED BY THE JURISDICTION IN WHICH THE PROJECT IS LOCATED. BOTH METHODS SHOULD BE SIZED USING A PARTICLE SIZE DISTRIBUTION HAVING A MEAN PARTICLE SIZE (D_{50}) OF 125 MICRONS UNLESS OTHERWISE STATED.

B. THE SWTD SHALL BE CAPABLE OF CAPTURING AND RETAINING 100 PERCENT OF POLLUTANTS GREATER THAN OR EQUAL TO 2.4 MILLIMETERS (MM) REGARDLESS OF THE POLLUTANT'S SPECIFIC GRAVITY (I.E.: FLOATABLE AND NEUTRALLY BUOYANT MATERIALS) FOR FLOWS UP TO THE DEVICE'S RATED-TREATMENT CAPACITY. THE SWTD SHALL BE DESIGNED TO RETAIN ALL PREVIOUSLY CAPTURED POLLUTANTS ADDRESSED BY THIS SUBSECTION UNDER ALL FLOW CONDITIONS. THE SWTD SHALL BE CAPABLE OF CAPTURING AND RETAINING TOTAL PETROLEUM HYDROCARBONS. THE SWTD SHALL BE CAPABLE OF ACHIEVING A REMOVAL EFFICIENCY OF 92 AND 78 PERCENT WHEN THE DEVICE IS OPERATING AT 25 AND 50 PERCENT OF ITS RATED-TREATMENT CAPACITY. THESE REMOVAL EFFICIENCIES SHALL BE BASED ON INDEPENDENT THIRD-PARTY RESEARCH FOR INFLUENT OIL CONCENTRATIONS REPRESENTATIVE OF STORM WATER RUNOFF (20 ± 5 MG/L). THE SWTD SHALL BE GREATER THAN 99 PERCENT EFFECTIVE IN CONTROLLING DRY-WEATHER ACCIDENTAL OIL SPILLS.

C. THE SWTD SHALL BE DESIGNED WITH A SUMP CHAMBER FOR THE STORAGE OF CAPTURED SEDIMENTS AND OTHER NEGATIVELY BUOYANT POLLUTANTS IN BETWEEN MAINTENANCE CYCLES. THE MINIMUM STORAGE CAPACITY PROVIDED BY THE SUMP CHAMBER SHALL BE IN ACCORDANCE WITH THE VOLUME ADEQUATE FOR THE DESIGN FLOW. THE BOUNDARIES OF THE SUMP CHAMBER SHALL BE LIMITED TO THAT WHICH DO NOT DEGRADE THE SWTD'S TREATMENT EFFICIENCY AS CAPTURED POLLUTANTS ACCUMULATE. THE SUMP CHAMBER SHALL BE SEPARATE FROM THE TREATMENT PROCESSING PORTION(S) OF THE SWTD TO MINIMIZE THE PROBABILITY OF FINE PARTICLE RE-SUSPENSION. IN ORDER TO NOT RESTRICT THE OWNER'S ABILITY TO MAINTAIN THE SWTD, THE MINIMUM DIMENSION PROVIDING ACCESS FROM THE GROUND SURFACE TO THE SUMP CHAMBER SHALL BE 16 INCHES IN DIAMETER.

D. THE SWTD SHALL BE DESIGNED TO CAPTURE AND RETAIN TOTAL PETROLEUM HYDROCARBONS GENERATED BY WET-WEATHER FLOW AND DRY-WEATHER GROSS SPILLS AND HAVE A CAPACITY ADEQUATE FOR THE DESIGN FLOW..

E. THE SWTD SHALL CONVEY THE FLOW FROM THE PEAK DESIGN FLOW OF 1 L/S OF THE DRAINAGE NETWORK, IN ACCORDANCE WITH REQUIRED HYDRAULIC UPSTREAM CONDITIONS AS DEFINED BY THE ENGINEER. IF A SUBSTITUTE SWTD IS PROPOSED, SUPPORTING DOCUMENTATION SHALL BE SUBMITTED THAT DEMONSTRATES EQUAL OR BETTER UPSTREAM HYDRAULIC CONDITIONS COMPARED TO THAT SPECIFIED HEREIN. THIS DOCUMENTATION SHALL BE SIGNED AND SEALED BY A PROFESSIONAL ENGINEER REGISTERED IN THE STATE OF THE WORK. ALL COSTS ASSOCIATED WITH PREPARING AND CERTIFYING THIS DOCUMENTATION SHALL BE BORN SOLELY BY THE CONTRACTOR.

F. THE SWTD SHALL HAVE COMPLETED FIELD TESTED FOLLOWING TARP TIER II PROTOCOL REQUIREMENTS

PART 3 - EXECUTION

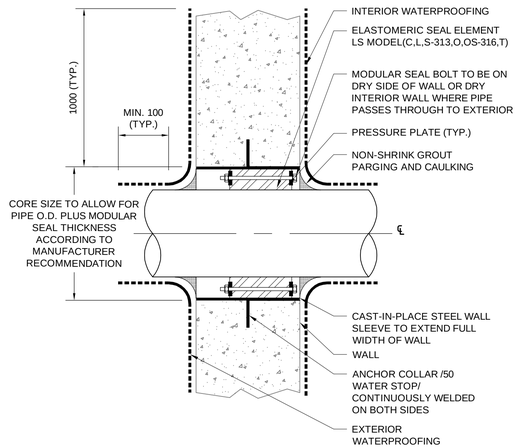
3.01 INSTALLATION:

A. THE CONTRACTOR SHALL EXERCISE CARE IN THE STORAGE AND HANDLING OF THE SWTD COMPONENTS PRIOR TO AND DURING INSTALLATION. ANY REPAIR OR REPLACEMENT COSTS ASSOCIATED WITH EVENTS OCCURRING AFTER DELIVERY IS ACCEPTED AND UNLOADING HAS COMMENCED SHALL BE BORNE BY THE CONTRACTOR.

B. THE SWTD SHALL BE INSTALLED IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS AND RELATED SECTIONS OF THE CONTRACT DOCUMENTS. THE MANUFACTURER SHALL PROVIDE THE CONTRACTOR INSTALLATION INSTRUCTIONS AND OFFER ON-SITE GUIDANCE DURING THE IMPORTANT STAGES OF THE INSTALLATION AS IDENTIFIED BY THE MANUFACTURER AT NO ADDITIONAL EXPENSE. A MINIMUM OF 72 HOURS NOTICE SHALL BE PROVIDED TO THE MANUFACTURER PRIOR TO THEIR PERFORMANCE OF THE SERVICES INCLUDED UNDER THIS SUBSECTION.

C. THE CONTRACTOR SHALL FILL ALL VOIDS ASSOCIATED WITH LIFTING PROVISIONS PROVIDED BY THE MANUFACTURER. THESE VOIDS SHALL BE FILLED WITH NON-SHRINKING GROUT PROVIDING A FINISHED SURFACE CONSISTENT WITH ADJACENT SURFACES. THE CONTRACTOR SHALL TRIM ALL PROTRUDING LIFTING PROVISIONS FLUSH WITH THE ADJACENT CONCRETE SURFACE IN A MANNER, WHICH LEAVES NO SHARP POINTS OR EDGES.

D. THE CONTRACTOR SHALL REMOVAL ALL LOOSE MATERIAL AND POOLING WATER FROM THE SWTD PRIOR TO THE TRANSFER OF OPERATIONAL RESPONSIBILITY TO THE OWNER.



4 C103 GRIT SEPARATOR PIPE CONNECTION DETAIL
NTS

NOTES GÉNÉRALES General Notes

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STRUCTURE Structural
Adjeleian Allen Rubeli Ltd. (AAR)
75 Albert Street, Suite 1005
Ottawa, ON K1P 5E7
T 613 232 5706

ARCHITECTURE DE PAYSAGE Landscape Architect
James B. Lennox & Associates Inc.
3332 Carling Ave.
Ottawa, ON K2H 5A8
T 613 722 5188

CIVIL Civil
R.V. Anderson Associates Limited
1750 Courtwood Crescent, Suite 220
Ottawa, ON K2C 2B5
T 613 226 1844

HYDRAULIQUE/ÉLECTRIQUE Hydraulic/Electrical
Goodkey, Weedmark & Associates Limited (GWAL)
1688 Woodward Drive
Ottawa, ON K2C 3R8
T 613 727 5111

URBANISTE Urban Planner
Fotenn Planning + Design
396 Cooper Street - Suite 300
Ottawa, ON, K2P 2H7
T 613 730 5709

ARCHITECTES Architect
NEUF architect(e)s SENCRL
630, boul. René-Lévesque O. 32e étages, Montréal QC H3B 1S6
T 514 847 1117 NEUFarchitectes.com

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OUVRAGE Project

320 McRAE AVENUE

EMPLACEMENT Location
1976 SCOTT ST & 320 McRAE AVE
OTTAWA, ON

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TITRE DU DESSIN Drawing Title

DETAILS AND SPECIFICATION

RÉVISION Revision: **0**
NO. DESSIN Dwg Number: **C103**