Servicing Brief – Bachman Terrace Residential Development

Project # 160401069

Prepared for:
Tega Developments

Prepared by:
Stantec Consulting Ltd.

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Prepared by (signature)

Sheridan Gillis

Reviewed by (signature)

Dustin Thiffault, P.Eng.

Stantec

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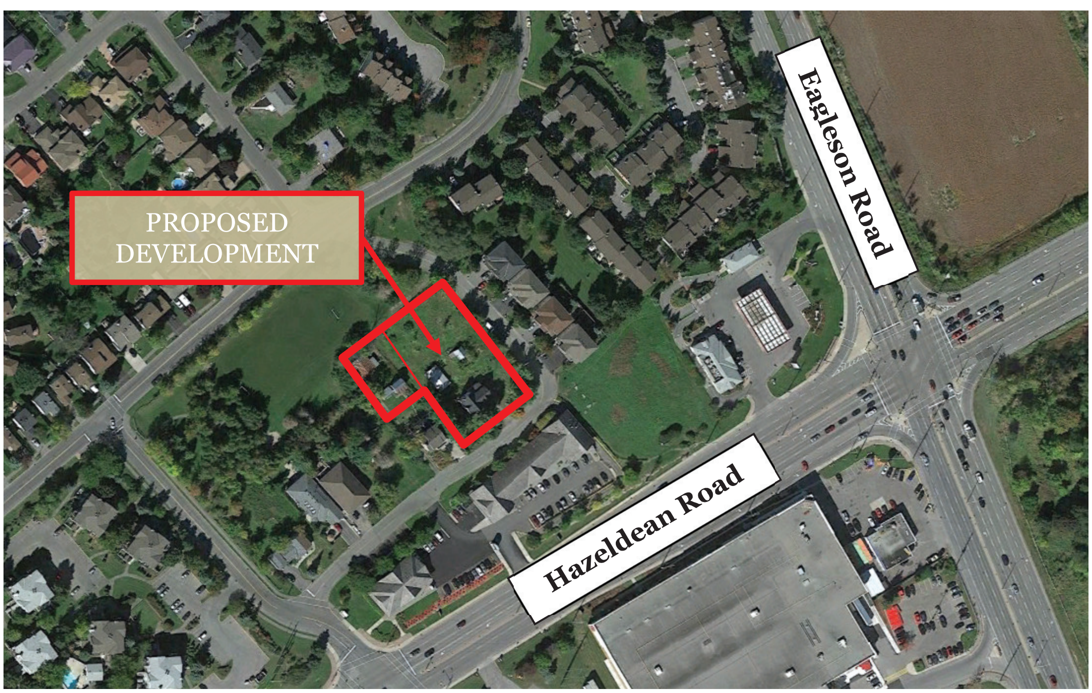
1.0 Introduction

Stantec Consulting Ltd. has been commissioned by Tega Developments to prepare the following Servicing Brief in support of the proposed twenty-three (23) unit residential development at 19 & 23 Bachman Terrace. The subject property is located within the City of Ottawa (formerly Kanata) and is currently zoned Residential (R1M). The development is bordered by Bachman Terrace to the north and east, existing residential to the south, and Irwin Gate Park to the west. The property comprises approximately 0.34ha of land and is indicated in **Figure 1-1**.

The existing properties are currently developed with single family homes and outbuildings on each parcel. 23 Bachman Terrace is currently serviced by a drilled well and septic system, which will be removed/decommissioned as part of construction activities; the existing home and out-buildings on 23 Bachman Terrace will also be removed. A severance application is proposed at the rear of 19 Bachman Terrace will retain the existing residence but remove existing out-buildings within the severed portion of the property. The proposed development includes eighteen (18) units located at 23 Bachman Terrace, with the remaining five (5) located within the future severance at the rear of 19 Bachman Terrace. The proposed townhouse dwellings will consist of slab on grade construction and be serviced by municipal sewer and water.

The intent of this report is to provide a servicing scenario for the site that is free of conflicts, provides on-site servicing in accordance with City of Ottawa design guidelines, and utilizes the existing local infrastructure in accordance with background drawings and as per consultation with City staff.

Figure 1-1: Approximate Location of Proposed Residential Development



2.0 Background

Documents referenced in preparing the site design for the redevelopment of 19 and 23 Bachman Terrace include:

- *Sewer Design Guidelines 2nd Edition,* City of Ottawa, October 2012
- *Water Distribution Design Guidelines 1st Edition,* City of Ottawa, July 2010
- *Design Guidelines for Drinking Water Systems*, Ministry of the Environment, 2008
- *Design Guidelines for Sewage Works*, Ministry of the Environment, 2008
- *Stormwater Management Planning and Design Manual,* Ministry of the Environment, March 2003
- *Shirley's Brook and Watts Creek Subwatershed Study*, Dillon Consulting, , September 1999
- *Geotechnical Investigation – Proposed Redevelopment 19 & 23 Bachman Terrace,* Houle Chevrier, June 25, 2014

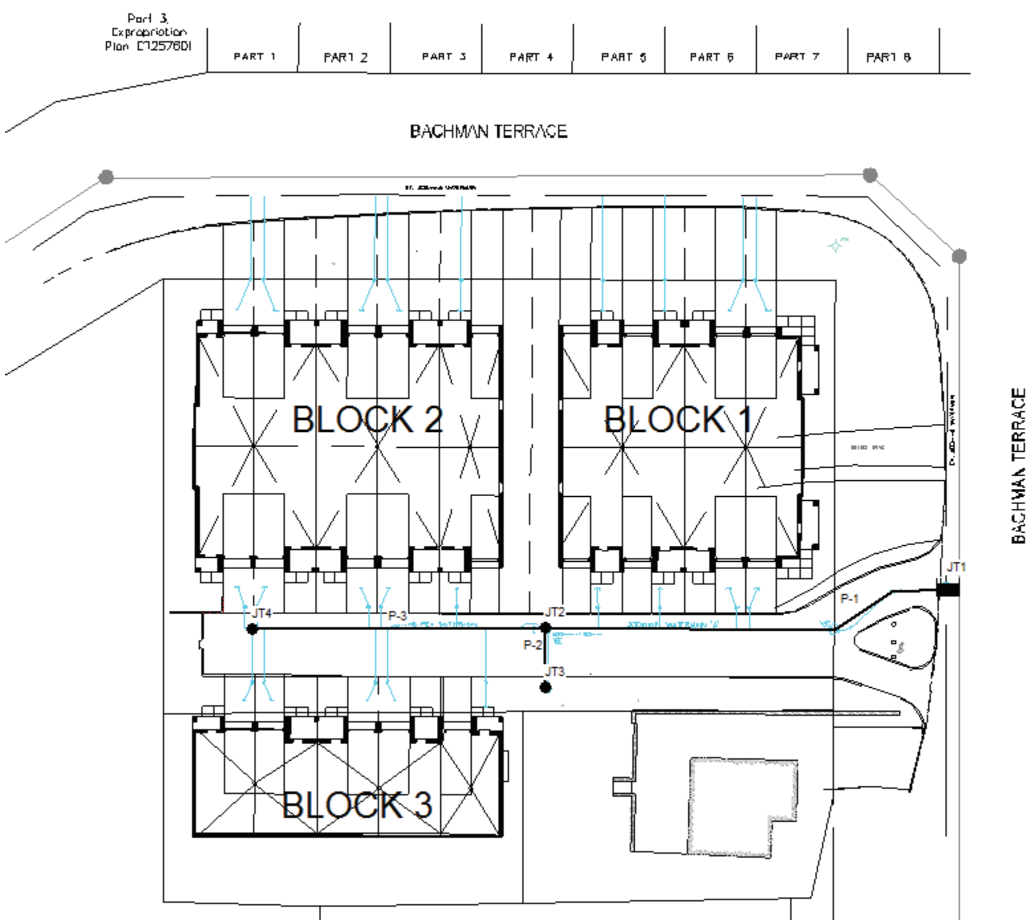
A Stormwater Management Report for the development has been submitted by Stantec Consulting Ltd. under a separate cover.

3.0 Water Supply Servicing

3.1 BACKGROUND

The proposed development includes 23 residential units, with potable water for 9 units serviced directly from the existing 400mm diameter main within Bachman Terrace, and the remaining 14 units being serviced off the proposed private street. Servicing within the proposed private street will include a 300mm watermain from the proposed hydrant to a TVS Chamber connection at the existing 400mm watermain within Bachman Terrace. In addition a 50mm water service beyond the proposed hydrant will provide domestic supply to the remaining units. A sketch of the proposed water servicing infrastructure is included in Figure 3-1.

Figure 3-1: Proposed Water Supply Servicing Plan



3.2 BOUNDARY CONDITIONS

Water demands for the proposed development were provided to the City of Ottawa in order to determine boundary conditions for the site. Hydraulic gradeline (HGL) values for each of the demand scenarios were then obtained by the City. The HGL values provided by the City are summarized below.

Table 1: Boundary Conditions

	Proposed Connection
Max HGL:	165.1m
Peak Hour (PKHR):	155.3m
Max Day + Fire Flow (MXDY+FF):	150.9m

3.3 WATER DEMANDS

Water demands for the development were estimated using the City of Ottawa's Water Distribution Design Guidelines. The estimated household size of an average townhome is **2.7 persons,** therefore the total projected population for the proposed residential development will be **62** persons.

For residential developments, the average day per capita water demand is **350 L/(cap*d)**.

The average day demand (AVDY) for the entire site was determined to be 0.25 L/s. The maximum daily demand (MXDY) is 2.5 times the AVDY which equals 0.63 L/s. The peak hour demand (PKHR) is 2.2 times the MXDY, totaling 1.38 L/s.

3.4 HYDRAULIC MODEL RESULTS

A hydraulic model was created to assess available pressures for the proposed 300mm diameter watermain within the proposed private street. The software package used to carry out the analysis was EPANET Version 0.5. Hazen-Williams carrying capacity coefficients were applied to the new watermain in accordance with the July 2010 Watermain Design Guidelines, which state in section 4.2.12 of that document: *the following "C" values shall be used for the design of water distribution systems regardless of pipe materials:*

Pipe Diameter (mm)	**C-Factor**
150	100
200 to 250	110
300 to 600	120
over 600	130

The Guidelines also specify the actual inside diameters that are to be used when nominal pipe sizes are used:

Pipe Diameters to be used in modeling (in mm):

Nominal Size Allowed (C900 & C905 PVC Pipe)	PVC, DI &C303 ID to be used for Modeling
50	50
150	155
300	297

The model was tested under three different domestic demand conditions: average day (AVDY), peak hour (PKHR) and the emergency condition of maximum day plus fire flow (MXDY + FF).

Junction demands, elevations, and the results of the model runs are provided in **Appendix A.3,** and are described in the following sections.

3.4.1 Average Day Results

AVDY output reports included in **Appendix A.3** contain the complete model output results for the average day demand analysis. The expected operating pressure for the proposed development is 455 kPa (66 psi). The pressures are within the allowable pressure range of 345 kPa to 480 kPa (50 to 70 psi) as recommended by the Ottawa Design Guidelines for Water Distribution.

3.4.2 Peak Hour Results

PKHR output reports included in **Appendix A.3** contain the complete model output results for the peak hour demand analysis. The expected peak hour pressure for the proposed development is 359 kPa (52 psi). The pressures are within the allowable pressure range of 345 kPa to 480 kPa (50 to 70 psi) as recommended by the Ottawa Design Guidelines for Water Distribution.

3.4.3 Maximum Day + Fire Flow Results

The City of Ottawa's design guidelines for water distribution systems require a minimum pressure of 140 kPa (20 psi) to be maintained at all points in the distribution system under a condition of maximum day and fire flow demand.

Based on calculations using the Fire Underwriters Survey (FUS) 1999 publication Water Supply for Public Fire Protection the required fire flow for this development is 317 L/s. The maximum day demand for the development was earlier assessed as 0.63L/s. The hydraulic model was used to analyze the maximum day + Fire flow demands, with appropriate fire flow demands attached to each hydrant. The MXDY+FF output reports (**Appendix A.3**) show residual pressure with this fire flow demand. Residual pressures have been calculated at 145 kpa (21 psi), which is above the minimum pressure of 20 psi.

3.5 SUMMARY OF FINDINGS

The proposed residential development is located in an area of the City's water distribution system that has sufficient capacity to provide both the required domestic and emergency fire flows. Based on computer modeling results, fire flows are available for this development which provide adequate water supply in accordance with the Fire Underwriters Survey (FUS) 1999 Guidelines.

The analysis shows a minimum pressure during peak demand of 359 kPa (52 psi) which is within the recommended design guidelines for pressure.

The analysis shows a maximum pressure during peak demand of 455 kPa (66 psi) which does not exceed the maximum allowable pressure for the system.

4.0 Wastewater Servicing

4.1 BACKGROUND

Wastewater infrastructure currently exists within the Bachman Terrace ROW to service residences in the area. The existing 250mm diameter gravity main drains north to Bellview Drive and ultimately outlets to the 1200mm diameter Glen Cairn Trunk along Eagleson Road. 23 Bachman Terrace is currently serviced via a private septic system (to be removed), while the residence at 19 Bachman Terrace is currently connected to the aforementioned 250mm diameter mainline.

4.2 DESIGN CRITERIA

As outlined in the City's "Ottawa Sewer Design Guidelines" the following design guidelines were used to calculate estimated wastewater flow rates and to size the sanitary sewers

- Minimum Velocity – 0.6 m/s
- Maximum Velocity – 3.0 m/s
- Manning roughness coefficient for all smooth wall pipes – 0.013
- Townhouse Population per Unit – 2.7
- Extraneous Flow Allowance – 0.28 l/s/ha
- Manhole Spacing – 120 m
- Minimum Cover – 2.5m

In addition, a residential peak factor based on Harmon's Equation was used to determine the peak design flows.

4.3 PROPOSED SERVICING

The proposed development requires sanitary servicing for twenty-three (23) townhouse units. Nine (9) units fronting Bachman Terrace will be serviced via new 135mm diameter PVC service laterals to the existing 250mm diameter sanitary main within Bachman Terrace. The remaining fourteen (14) units will front the proposed private street and will be serviced via new 135mm diameter PVC lateral connecting to a new proposed 300mm diameter PVC main. The proposed mainline within the private street will include a manhole at property line for monitoring purposes, and connect via a PVC tee to the 250mm diameter sanitary main within Bachman Terrace.

Daily Sanitary Peak flows have been calculated for the development as 1.1l/sec. This includes 0.1l/sec of extraneous flow. Due to the minimal additional contribution from the proposed infill development it is expected that the sanitary main within Bachman Terrace and associated downstream infrastructure will have adequate capacity to service the site.

The proposed drainage pattern is detailed on **Drawing SA-1**. A Sanitary sewer design sheet is included in **Appendix B**.

5.0 Stormwater Management

5.1 OBJECTIVES

The City of Ottawa has required that the post-development peak rate of site runoff not exceed the predevelopment release rate for the site. Stormwater may be detained, if necessary, to ensure that the allowable release rate is not exceeded.

Stormwater management facilities currently do not exist on-site. The stormwater management design aims to provide on-site storage to ensure that the allowable site release rate has not been exceeded in accordance with the criteria and constraints listed below.

5.2 SWM CRITERIA AND CONSTRAINTS

The stormwater management criteria for the proposed site are based on City of Ottawa Sewer Design Guidelines (2004) and on a pre-consultation meeting with City of Ottawa Staff. The following summarizes the criteria used in the preparation of this stormwater management plan:

- Maximum discharge during the 5 and 100 year storms to be restricted to that of predevelopment conditions.
- Maximum 100 year ponding depth of 0.30 m in parking and access areas.
- Provide adequate emergency overflow conveyance off-site.
- Size storm sewers to convey 5 year storm event, assuming only roof controls are imposed (i.e. provide capacity for system without inlet-control devices installed).
- Size storm culverts to convey 50 year storm event (local urban road, over 6m culvert span).
- Size storm sewers using an inlet time of concentration (Tc) of 10 minutes.
- On-site quality control not required for development (pre-consultation meeting).

5.3 GEOTECHNICAL CONSIDERATIONS

A geotechnical investigation titled *Geotechnical Investigation – Proposed Redevelopment – 19 and 23 Bachman Terrace* has been prepared for the subject site (Houle Chevrier, June 2014). The report indicates the presence of bedrock within approximately 1.0m from the surface of the existing site. In consideration of this and the lack of storm sewers within the Bachman Terrace ROW, subsurface storage has not been considered for use within the subject site.

5.4 DESIGN METHODOLOGY

The intent of the stormwater management plan presented herein is to mitigate any negative impact that the proposed development will have on the existing storm sewer infrastructure, while providing adequate levels of service to the proposed buildings and access areas. The proposed stormwater management plan is designed to detain runoff on the rooftop and the surface to ensure that peak flows after construction will not exceed the predevelopment flow rates from the site.

5.5 WATER QUANTITY CONTROL

The Modified Rational Method was employed to assess the quantity and volume of runoff generated during post-development conditions. The site was subdivided into subcatchments (subareas) tributary to stormwater controls as defined by the location of inlet control devices, and used in culvert design. Sub catchment areas and runoff coefficients are identified on **Drawing SD-1**.

5.6 ALLOWABLE RELEASE RATE

The predevelopment release rate for the area has been determined using the rational method. Existing buildings and access areas were considered as hard surfaces (C=0.9), while the remainder of the site is grassed (C=0.2). A time of concentration for the predevelopment area (15 minutes) was assigned during a pre-consultation meeting with City of Ottawa staff. C coefficient values have been increase by 25% for the 100-year storm event per City of Ottawa guidelines. Peak flow rates have been calculated using the rational method as follows:

Q = 2.78 CiA

Where: Q = peak flow rate, L/s
A = drainage area, ha
I = rainfall intensity, mm/hr (per Ottawa IDF curves)
C = site runoff coefficient

Target release rates for the site have been summarized in the table below:

Table 2: Target Release Rates

Design Storm	Target Flow Rate (L/s)
5-Year	22.7
100-Year	48.6

5.7 STORAGE REQUIREMENTS

The site requires quantity control measures to meet the restrictive stormwater release criteria. It is proposed that an inlet-control device in combination with surface grading (for ponding) and rooftop drain restrictions be used to reduce the peak flow. To provide the necessary controls, surface and roof storage were maximized across the site.

5.8 UNCONTROLLED TRIBUTARY CATCHMENTS

Due to grading constraints, one catchment has been designed without a storage component (Uncon). This area flows offsite to the Bachman Terrace ROW uncontrolled. Areas that discharge offsite without entering the proposed stormwater management system must be compensated for in areas with controls. **Table 3** summarizes the peak uncontrolled 5 and 100 year catchment release rates for catchments that are released to Bachman Terrace uncontrolled.

Table 3: 5 and 100 Year Discharge From Uncontrolled Catchments

Catchment ID	5-Year Peak Discharge (L/s)	100-Year Peak Discharge (L/s)
Uncon	9.2	19.7

5.9 ROOFTOP STORAGE

It is proposed to retain stormwater on the rooftops by installing restricted flow roof drains. The following calculations assumes the roof will be equipped with a standard Zurn Model Z-105-5 Control-Flow Single Notch Roof Drain, see Appendix C for details.

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

Table 4 and **Table 5** provide details regarding the retention of stormwater on the proposed rooftops during the 5 and 100 year storm events. Refer to **Appendix C** for details. Both buildings are tributary to the upstream control (via roof leader) for area A-1 and as such their discharges are further controlled at that location.

Table 4: Summary of Rooftop Storage (5-Year)

Location	Depth (mm)	Discharge (L/s)	$V_{required}$ (m^3)	Drawdown Time (h)	$V_{available}$ (m^3)
Block 1	90	2.7	7	1.1	20
Block 2	87	4.0	9	0.9	26
Block 3	72	3.3	3	0.3	12

1. Buildings 1 and 3are tributary to the downstream control at A-1.

Table 5: Summary of Rooftop Storage (100-Year)

Location	Depth (mm)	Discharge (L/s)	$V_{required}$ (m^3)	Drawdown Time (h)	$V_{available}$ (m^3)
Block 1	133	4.0	16	1.7	20
Block 2	129	5.9	19	1.4	26
Block 3	109	5.0	7	0.5	12

1. Buildings 1 and 3 are tributary to the downstream control at A-1.

5.10 SURFACE STORAGE

In addition to rooftop storage, it is proposed to detain stormwater on the surface in parking and landscape areas. The modified rational method was employed to determine the peak volume stored in surface ponding areas. Inlet control devices were sized based on the available storage volumes during the 5 and 100 year storm events.

Table 6 summarizes the estimated storm release rates and storage volumes during the 5 and 100 events.

Table 6: 5 and 100 Year Peak Surface Volume and Controlled Discharge Summary

		5-Year Event			100-Year Event		
Tributary Area ID	**ICD (mm)**	**Discharge (L/s)**	**$V_{required}$ (m^3)**	**$V_{available}$ (m^3)**	**Discharge (L/s)**	**$V_{required}$ (m^3)**	**$V_{available}$ (m^3)**
A-1	135	11.6	17.4	23.4	21.2	20.7	23.4

1. 100-year volume available based on subsurface storage and a maximum spill depth not to exceed 0.30m, where required.

The inlet control device (ICD) was sized with the following orifice equation:

$$Q = C_d A(2gh)^{1/2}$$

Where, C_d = 0.61
A = Area of Orifice (m2)
g = 9.81 m/s2
h = design head (m)

The design head used to determine restricted flow rates through the proposed orifices was measured from the downstream water level up to the level of surface ponding in the catchment area. Downstream water levels were considered to be at the obvert of the receiving culvert within the Bachman Terrace ROW. Refer to **Appendix C** for details.

5.11 RESULTS

Table 7 demonstrates that the proposed stormwater management plan provides adequate attenuation storage to meet the target peak outflow rates for the site.

Table 7: Summary of Total 5 and 100 Year Event Release Rates

	5-Year Peak Discharge (L/s)	100-Year Peak Discharge (L/s)
Uncontrolled	9.2	19.7
Controlled – Roof	10.0	14.9
Controlled - Surface	11.6	21.2
Total	**24.8**	**46.8**
Target	22.7	48.6

*Note that roof discharge rates may not be summed directly to the total site peak discharge, as the release rates of Blocks 1 and 3 are included within the controlled surface discharge values due to downstream controls.

5.12 WATER QUALITY CONTROL

Conversations with City of Ottawa staff have not identified the need for on-site stormwater quality control measures. As roof leaders and controlled discharge areas are routed to grassed swales at the perimeter of the property, and site drainage is routed in its entirety to grassed swales and culverts within the Bachman Terrace ROW over the entire site frontage, it is assumed that suspended solids within runoff generated by the site will not have a deleterious impact on downstream watercourses (Watt's Creek).

6.0 Grading and Drainage

The existing topography of the site is generally flat with overland sheet flow draining from south to north to a shallow roadside ditch on the south side of Bachman Terrace. The existing roadside ditch is not well defined but also serves to collect road drainage from a portion of Bachman Terrace, and then direct flows to a 400m CSP culvert at the northwest corner of the site. Previous reports have identified localized ponding in the area, likely due to the lack of storm sewer within the Bachman Terrace ROW, the shallow nature of the existing roadside ditch, and the lack of positive drainage on the existing paved portion of the ROW.

Several key considerations were applied to develop the proposed grading strategy for the site.

i. Ensure appropriate depth of cover is provided for site services in accordance with Ottawa Sewer Design Guidelines.

ii. Ensure drainage patterns and overland flow route direct flows to the existing CSP culvert outlet as per predevelopment conditions.

iii. Review of impacts to existing bedrock, and mitigate removal where possible.

iv. Adhere to all applicable City of Ottawa grading and drainage criteria for the development.

v. Maintain the existing boundary conditions for the site, specifically with respect to elevations around existing mature trees to be retained.

The proposed site grading plan, including direction of overland flow is depicted on **Drawing GP-1.**

7.0 Utilities

7.1 HYDRO

Existing Hydro infrastructure exists within the Bachman Terrace ROW, including an existing pad mounted transformer at the entrance to the future private street. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Consultation with Hydro Ottawa will occur throughout the Composite Utility Planning process. Exact size, location, and routing of hydro utilities, as well as required transformer locations, will be finalized after design circulation.

7.2 GAS

Similarly to Hydro, gas infrastructure exists within the Bachman Terrace ROW. Exact size, location and routing of gas infrastructure will be finalized as part of the Composite Utility Planning process, following design circulation.

7.3 TELECOMMUNICATIONS

Both Bell and Rogers are expected to be able to service the proposed site via underground utility infrastructure within the Bachman Terrace ROWs. Bell and Rogers may require easements for their respective cabinets and vaults. Easement requirements and location of telecommunication infrastructure will be determined as part of the Composite Utility Planning process, following design circulation.

8.0 Approvals

Pre-consultation with Ontario Ministry of Environment (MOE) staff concerning Environmental Compliance Approvals (ECAs, formerly Certificates of Approval (CofA)) under the Ontario Water Resources Act is forthcoming, and is expected to confirm that an ECA Approval will be required for the storm and sanitary sewer system. The ECA application is expected to fall under the transfer of review agreement with the City of Ottawa.

A MOE Permit to Take Water (PTTW) may be required for the site. The geotechnical consultant shall confirm at the time of application that a PTTW is required.

9.0 Erosion Control During Construction

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

1. Implement best management practices to provide appropriate protection of the existing and proposed drainage system and the receiving water course(s).
2. Limit extent of exposed soils at any given time.
3. Re-vegetate exposed areas as soon as possible.
4. Minimize the area to be cleared and grubbed.
5. Protect exposed slopes with plastic or synthetic mulches.
6. Provide sediment traps and basins during dewatering.
7. Plan construction at proper time to avoid flooding.

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

8. Verification that water is not flowing under silt barriers.

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

10.0 Geotechnical Investigation

A geotechnical investigation was completed by Houle Chevrier Engineering in February of 2014 and revised in June 2014. The report summarizes the existing soil conditions within the subject area and provides construction recommendations. For details which are not summarized below, please see the original Houle Chevrier report dated June 25, 2014.

Field Investigation for the site was carried out March 7th, 2013, with a total of four (4) test pits advanced to between 0.4 and 1.2 meters below ground. Subsurface conditions in the test pits were identified by visual and tactile examination of the material exposed on the sides and bottom of the test pits.

No groundwater inflow was observed by Houle Chevrier personnel at the time of test pit excavation on March 7th, 2013. However it is noted that groundwater levels fluctuate seasonally and may be higher during wet periods of the year, or following heavy precipitation.

A layer of fractured/weathered bedrock was encountered below the glacial till below two (2) test pits, at elevations of 117.0 and 117.6 meters geodetic respectively. It is expected that the structure footings and proposed servicing will require excavation of existing bedrock to facilitate construction. Proposed structures may be founded on spread footings placed either directly on the surface of the bedrock or on a pad of engineered fill above the bedrock. The footing recommendations outlined in the Houle Chevrier report should be reviewed in detail. Additionally, field input from the geotechnical consultant should be obtained to confirm current assumptions at time of excavation.

The required pavement structure for the private roadways is outlined in Table 8 below.

Table 8: Pavement Structure – Car Only Parking Areas

Thickness (mm)	Material Description
50	**Wear Course** – HL-3 or Superpave 12.5 Asphaltic Concrete
40	**Wear Course** – HL-3 or Superpave 19.0 Asphaltic Concrete
150	**Base** – OPSS Granular A Crushed Stone
375	**Subbase** – OPSS Granular B Type II
-	**Subgrade** – Either fill, in situ soil, select subgrade material or OPSS Granular B Type I or II material placed over in situ soil or fill.

11.0 Conclusions

11.1 WATER SERVICING

Based on the results of the hydraulic analysis presented in this report, the proposed servicing in this development will provide sufficient capacity to sustain both the required domestic demands and emergency fire flow demands. Based on computer modeling results, fire flows greater than those required per the Fire Underwriters Survey (FUS) guidelines are available.

The minimum and maximum pressures found within the model output results during peak demand are 359kPa (52 psi) and 455kPa (66 psi) respectively, which are both within the recommended design guidelines for minimum and maximum pressure for the system.

11.2 SANITARY SERVICING

Daily Sanitary Peak flows have been calculated for the development as 1.10l/sec. This includes 0.10l/sec of extraneous flow. Due to the minimal additional contribution from the proposed infill development it is expected that the sanitary main within Bachman Terrace and associated downstream infrastructure will have adequate capacity to service the site.

11.3 STORMWATER SERVICING

Based on the preceding report and in conjunction with the Stormwater Management Report for 19 and 23 Bachman Terrace, the following conclusions can be drawn:

- Quantity control is provided via a combination of rooftop and surface storage.
- The site discharges stormwater to the existing storm sewer infrastructure without exceeding the 100-year event calculated allowable release rate.
- 100 year ponding depths have been maintained at a maximum of 0.055 m on rooftops and 0.30 m in surface catchments within parking areas.
- 100 year volumes for controlled catchments are contained on-site.
- Major overland flow paths have been provided to relieve the parking and access areas during emergency conditions or extreme rainfall events.

Additional stormwater quality control measures are not required

11.4 GRADING

Grading for the site has been designed as per City of Ottawa requirements and provides for outlet of emergency overland flow under extreme flood conditions. Erosion and sediment control measures will be implemented during construction to reduce the impact on existing facilities.

11.5 UTILITIES

Utility infrastructure exists within the Bachman Terrace ROW at the north and east property boundaries of the proposed site. It is anticipated that existing infrastructure will be sufficient to provide a means of distribution for the proposed site. Exact size, location and routing of utilities will be finalized after design circulation.

11.6 APPROVAL / PERMITS

MOE Environmental Compliance Approvals are expected to be required for the subject site as the site will be developed as freehold condominium with multiple ownership. The ECA application is expected to fall under the transfer of review program with the City of Ottawa. Forthcoming pre-consultation with the MOE will confirm the above statement. A Permit to Take Water may be required for pumping requirements for sewer installation. No other approval requirements from other regulatory agencies are anticipated.

Appendix A : Water Supply Servicing

A.1 DOMESTIC WATER DEMAND ESTIMATES

Estimated Water Demand

Water demand may be estimated based on the City of Ottawa Watermain Distribution Guidelines, July 2010.

Estimated Population:

23 Townhomes x 2.7 p / unit = 62p

Average Daily Demand:

$$Q_{avg} = 62\,p \times \frac{350L}{p \cdot d} = 21{,}700\frac{L}{d} = 0.25\frac{L}{s}$$

Maximum Daily Demand:

$$Q_{\max\_daily} = 21{,}700\frac{L}{d} \times 2.5 = 54{,}250\frac{L}{d} \times \frac{1d}{86{,}400s} = 0.63\frac{L}{s}$$

Peak Hourly:

$$Q_{peak\_hourly} = 54{,}250\frac{L}{d} \times 2.2 \times = 119{,}350\frac{L}{d} \times \frac{1d}{86{,}400s} = 1.38\frac{L}{s}$$

A.2 FIRE FLOW REQUIREMENTS PER FUS GUIDELINES (1999)

FUS Fire Flow Calculations

Stantec

Stantec Project #: 160401069
Project Name: 23 Bachman Terrace
Date: June 4, 2015
Data input by: Sheridan Gillis
Reviewed by: Dustin Thiffault, P.Eng.

Calculations Based on 1999 Publication "*Water Supply for Public Fire Protection*" by Fire Underwriters' Survey (FUS)

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

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

Step	Task	Term	Options	Multiplier Associated with Option	Choose:	Value Used	Unit	Total Fire Flow (L/min)
		Framing Material						
1	Choose Frame Used for Construction of Unit	Coefficient related to type of construction (C)	Wood Frame	1.5	Wood Frame	1.5	m	
			Ordinary construction	1				
			Non-combustible construction	0.8				
			Fire resistive construction (< 2 hrs)	0.7				
			Fire resistive construction (> 2 hrs)	0.6				
		Floor Space Area						
2	Choose Type of Housing (if TH, Enter Number of Units Per TH Block)	Type of Housing	Single Family	1	Townhouse - indicate # of units	8	Units	
			Townhouse - indicate # of units	8				
			Other (Comm, Ind, etc.)	1				
2.2	# of Storeys	Number of Floors/ Storeys in the Unit (do not include basement):			3	3	Storeys	
3	Enter Ground Floor Area of One Unit	Enter Ground Floor Area (A) of One Unit Only :			66	1,584	Area in Square Meters (m^2)	
		Measurement Units	Square Feet (ft^2)	0.09290304	Square Metres (m2)			
			Square Metres (m^2)	1				
			Hectares (ha)	10000				
4	Obtain Required Fire Flow without Reductions	Required Fire Flow(without reductions or increases per FUS) (F = 220 * C * √A) Round to nearest 1000L/min						13,000
5	Apply Factors Affecting Burning	**Reductions/Increases Due to Factors Affecting Burning**						
5.1	Choose Combustibility of Building Contents	Occupancy content hazard reduction or surcharge	Non-combustible	-0.25	Combustible	0	N/A	13,000
			Limited combustible	-0.15				
			Combustible	0				
			Free burning	0.15				
			Rapid burning	0.25				
5.2	Choose Reduction Due to Presence of Sprinklers	Sprinkler reduction	Complete Automatic Sprinkler Protection	-0.3	None	0	N/A	0
			None	0				
5.3	Choose Separation Distance Between Units	Exposure Distance Between Units	North Side	30.1 to 45.0m	0.05	0.45	m	5,850
			East Side	30.1 to 45.0m	0.05			
			South Side	10.1 to 20.0m	0.15			
			West Side	3.1 to 10.0m	0.2			
6	Obtain Required Fire Flow, Duration & Volume	*Total Required Fire Flow, rounded to nearest 1000 L/min, with max/min limits applied:*						**19,000**
		Total Required Fire Flow (above) in L/s:						**317**
		Required Duration of Fire Flow (hrs)						**4.25**
		Required Volume of Fire Flow (m^3)						**4,845**

Note: The most current FUS document should be referenced before design to ensure that the above figures are consistent with the intent of the Guideline

Legend	
(green)	Drop down menu - choose option, or enter value.
(hatched)	No Information, No input required.

A.3 HYDRAULIC ANALYSIS RESULTS

EPANET HYDRAULIC MODELLING RESULTS

NODE RESULTS

FILE NUMBERS: 1604-01069

SITE:

23 BACHMAN TERRACE

Hydraulic Analysis

DATE: June 3, 2015
REVISION:
DESIGNED BY: SGG
CHECKED BY: DCT

Average daily

Node ID	Elevation	Demand	Head	Pressure		
	m	LPS	m	m	psi	kPa
Junc JT3	118.65	0.00	155.30	36.65	52	359
Junc JT2	118.20	0.00	155.30	37.10	53	365
Junc JT4	118.20	0.25	155.27	37.07	53	365
Resvr JT1	155.30	-0.25	155.30	0.00	0	0

Peak hour

Node ID	Elevation	Demand	Head	Pressure		
	m	LPS	m	m	psi	kPa
Junc JT3	118.65	0.00	155.30	36.65	52	359
Junc JT2	118.20	0.00	155.30	37.10	53	365
Junc JT4	118.20	1.38	154.62	36.42	52	359
Resvr JT1	155.30	-1.38	155.30	0.00	0	0

Max day & FF

FF=18,000L/min

Node ID	Elevation	Demand	Head	Pressure		
	m	LPS	m	m	psi	kPa
Junc JT3	118.65	317.00	135.05	14.69	21	145
Junc JT2	118.20	0.00	148.61	30.17	43	296
Junc JT4	118.20	0.63	148.45	30.01	43	296
Resvr JT1	150.90	-317.63	150.90	0.00	0	0

pressure check

Node ID	Elevation	Demand	Head	Pressure		
	m	LPS	m	m	psi	kPa
Junc JT3	118.65	0.00	165.10	46.45	66	455
Junc JT2	118.20	0.00	165.10	46.90	67	462
Junc JT4	118.20	0.25	165.07	46.87	67	462
Resvr JT1	165.10	-0.25	165.10	0.00	0	0

EPANET HYDRAULIC MODELLING RESULTS

LINK RESULTS

FILE NUMBERS: 1604-01069

SUBDIVISION:

23 BACHMAN TERRACE

Hydraulic Analysis

DATE: June 3, 2015
REVISION:
DESIGNED BY: SGG
CHECKED BY: DCT

Average daily

	Length	Diameter	Roughness	Velocity	Unit Headloss
Link ID	m	mm		m/s	m/km
Pipe P-2	5.800	150.000	100.000	0.000	0.000
Pipe P-1	40.000	300.000	120.000	0.000	0.000
Pipe P-3	29.500	50.000	100.000	0.130	0.980

Peak hour

	Length	Diameter	Roughness	Velocity	Unit Headloss
Link ID	m	mm		m/s	m/km
Pipe P-2	5.800	150.000	100.000	0.000	0.000
Pipe P-1	40.000	300.000	120.000	0.020	0.000
Pipe P-3	29.500	50.000	100.000	0.700	23.140

Max day & FF

FF=7,500L/min

		Diameter	Roughness	Velocity	Unit Headloss
Link ID	Length	mm		m/s	m/km
Pipe P-2	5.800	150.000	100.000	17.940	2589.720
Pipe P-1	40.000	300.000	120.000	4.490	63.370
Pipe P-3	29.500	50.000	100.000	0.320	5.420

Appendix B : Wastewater Servicing

B.1 SANITARY SEWER DESIGN SHEET

SITE:

19 & 23 BACHMAN TERRACE - OTTAWA, ONTARIO

DATE:	28-Feb-14
REVISION:	7-Apr-14
DESIGNED BY:	SG
CHECKED BY:	DT

SANITARY SEWER DESIGN SHEET
(City of Ottawa)

FILE NUMBER: 160401069

DESIGN PARAMETERS

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	350 l/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	50,000 l/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 l/ha/day	MANNINGS n	0.013
PEAKING FACTOR (COMM., INST.):	1.5	INDUSTRIAL (LIGHT)	35,000 l/ha/day	BEDDING CLASS	B
PERSONS / SINGLE UNIT	3.4	INSTITUTIONAL	50,000 l/ha/day	MINIMUM COVER	2.50 m
PERSONS / TOWNHOME	2.7	INFILTRATION	0.28 l/s/Ha		
PERSONS / APARTMENT	2.3				

LOCATION			RESIDENTIAL AREA AND POPULATION									COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+I+I	INFILTRATION			TOTAL	PIPE								
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	SINGLE	UNITS TOWN	APT.	POP.	CUMULATIVE AREA (ha)	CUMULATIVE POP.	PEAK FACT.	PEAK FLOW (l/s)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	AREA (ha)	ACCU. AREA (ha)	PEAK FLOW (l/s)	TOTAL AREA (ha)	ACCU. AREA (ha)	INFILT. FLOW (l/s)	FLOW (l/s)	LENGTH (m)	DIA (mm)	MATERIAL	CLASS	SLOPE (%)	CAP. (FULL) (l/s)	CAP. V PEAK FLOW (%)	VEL. (FULL) (m/s)	VEL. (ACT.) (m/s)
SA101	102	101	0.23	0	15	0	41	0.23	41	4.00	0.66	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.23	0.23	0.06	0.72	58.6	200	PVC	SDR 35	0.65	27.0	**2.67%**	0.85	0.31
	101	MAIN	0.00	0	0	0	0	0.23	41	4.00	0.66	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	0.23	0.06	0.72	14.1	200	PVC	SDR 35	0.50	23.6	**3.05%**	0.74	0.28
EX 1	EX 4	EX 3	0.83	4	0	0	14	1.06	54	4.00	0.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.83	1.06	0.30	1.17	67.3	250	PVC	SDR 35	0.34	35.4	**3.32%**	0.71	0.27
	EX 3	EX 2	0.00	0	0	0	0	1.06	54	4.00	0.88	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.00	1.06	0.30	1.17	11.7	250	PVC	SDR 35	0.16	24.3	**4.84%**	0.49	0.21
SA102 & EX 2	EX 2	EX 1	0.47	8	10	0	54	1.53	108	4.00	1.75	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.0	0.47	1.53	0.43	2.18	74.0	250	PVC	SDR 35	0.33	34.8	**6.27%**	0.70	0.33

Note: Total flow generated from the site includes SA101 and SA102 for a total of 1.19l/sec (including 0.10 l/sec of infiltration flow)

Appendix C : Stormwater Management

C.1 MODIFIED RATIONAL METHOD STORAGE CALCULATIONS

Stormwater Management Design Parameters

5 yr Intensity $I = a/(t + b)^c$ a = 998.071
City of Ottawa b = 6.053
c = 0.814

5-Year Predevelopment Release Rate

Area (ha): 0.338
C: 0.29

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
15	83.6	22.7	22.7	0.0	0.0

100 yr Intensity $I = a/(t + b)^c$ a = 1735.688
City of Ottawa b = 6.014
c = 0.820

100-Year Predevelopment Release Rate

Area (ha): 0.338
C: 0.36

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
15	142.9	48.6	48.6	0.0	0.0

100-yr Target = 48.6 L/s *Site Runoff + External Runoff

Proposed Stormwater Management Plan

Uncontrolled Areas

Area ID: Uncon
Area (ha): 0.065
C: 0.49

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.2	9.2	9.2	0.0	0.0

Area ID: Uncon
Area (ha): 0.065
C: 0.61

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	19.7	19.7	0.0	0.0

Controlled Areas

Area ID: A-1
Area (ha): 0.126 *All Roof drains from Blocks 1 and 3
C: 0.56 tributary to Area A-1

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	
10	104.2	30.4	11.6	18.8	11.3	
15	83.6	26.4	11.6	14.8	13.3	
20	70.3	23.8	11.6	12.2	14.6	
30	53.9	20.6	11.6	9.0	16.1	
40	44.2	18.6	11.6	7.0	16.9	
50	37.7	17.4	11.6	5.8	17.3	
60	32.9	16.4	11.6	4.8	17.4	<--
70	29.4	15.7	11.6	4.1	17.4	
80	26.6	15.2	11.6	3.6	17.2	
90	24.3	14.7	11.6	3.1	17.0	
100	22.4	14.4	11.6	2.8	16.7	
110	20.8	14.1	11.6	2.5	16.3	
120	19.5	13.8	11.6	2.2	15.8	
130	18.3	13.6	11.6	2.0	15.4	

Orifice Equation: $Q = C_dA(2gh)^{1/2}$ Where C= 0.610
Orifice Diameter: 135 mm
Control Invert Elevation 118.00 m
T/G Elevation: 118.00 m
Downstream W/L: 118.00 m *Obvert of outlet sewer

Control Location	Ponding Elev.[1] (m)	Head[2] (m)	Discharge[3] (L/s)	$V_{required}$[4] (m^3)	$V_{available}$[5] (m^3)	Volume Check
A-1	118.09	0.09	11.6	17.4	23.4	ok

Area ID: A-1
Area (ha): 0.126 *All Roof drains from Blocks 1 and 3
C: 0.70 tributary to Area A-1

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	
10	178.6	52.8	21.2	31.6	18.9	
15	142.9	44.0	21.2	22.8	20.6	
20	120.0	38.4	21.2	17.2	20.7	<--
30	91.9	31.5	21.2	10.3	18.6	
40	75.1	27.4	21.2	6.2	15.0	
50	64.0	24.7	21.2	3.5	10.5	
60	55.9	22.7	21.2	1.5	5.5	
70	49.8	21.2	21.2	0.0	0.1	
80	45.0	20.0	20.0	0.0	0.0	
90	41.1	19.1	19.1	0.0	0.0	
100	37.9	18.3	18.3	0.0	0.0	
110	35.2	17.6	17.6	0.0	0.0	
120	32.9	17.1	17.1	0.0	0.0	
130	30.9	16.6	16.6	0.0	0.0	

Orifice Equation: $Q = C_dA(2gh)^{1/2}$ Where C= 0.610
Orifice Diameter: 135 mm
Control Invert Elevation 118.00 m
T/G Elevation: 118.00 m
Downstream W/L: 118.00 m *Obvert of outlet sewer

Control Location	Ponding Elev.[1] (m)	Head[2] (m)	Discharge[3] (L/s)	$V_{required}$[4] (m^3)	$V_{available}$[5] (m^3)	Volume Check
A-1	118.30	0.30	21.2	20.7	23.4	ok

Inlet Control Notes:
1. Max allowable water depth during storm event
2. Based on the difference between the 'Storage Depth' less the greater of either the 'Orifice Invert Elevation' or 'Downstream W/L'
3. Based on the orifice equation as shown above.
4. Volume required per Modified Rational Method.
5. Volume available as illustrated on dwg SD-1.

Roof Control Areas

Area (ha)	Depth (mm)	Discharge (L/s)	Vreq (m^3)	Est. Drawdown	Vavail (m^3)
0.051	90	2.7	7	1.1	20

Area (ha)	Depth (mm)	Discharge (L/s)	Vreq (m^3)	Est. Drawdown	Vavail (m^3)
0.065	87	4.0	9	0.9	26

Area (ha)	Depth (mm)	Discharge (L/s)	Vreq (m^3)	Est. Drawdown	Vavail (m^3)
0.031	72	3.3	3	0.3	12

Area (ha)	Depth (mm)	Discharge (L/s)	Vreq (m^3)	Est. Drawdown	Vavail (m^3)
0.051	133	4.0	16	1.7	20

Area (ha)	Depth (mm)	Discharge (L/s)	Vreq (m^3)	Est. Drawdown	Vavail (m^3)
0.065	129	5.9	19	1.4	26

Area (ha)	Depth (mm)	Discharge (L/s)	Vreq (m^3)	Est. Drawdown	Vavail (m^3)
0.031	109	5.0	7	0.5	12

Estimated Discharge from Site

Total Uncontrolled Area	0.065 ha
5-year Uncontrolled Release	9.2 L/s
Total Controlled Area	0.126 ha
Storage Volume	17.4 m^3
5-year Controlled Release	11.6 L/s
Total Controlled Roof Area	0.147 ha
Roof Storage Volume	19.3 m^3
5-year Controlled Roof Release	10.0 L/s
Total Area	**0.338 ha**
Total Volume Retained	**37 m^3**
Total 5-year Release	**24.8 L/s**

Total Uncontrolled Area	0.065 ha
100-year Uncontrolled Release	19.7 L/s
Total Controlled Area	0.126 ha
Storage Volume	20.7 m^3
100-year Controlled Release	21.2 L/s
Total Controlled Roof Area	0.147 ha
Roof Storage Volume	42.1 m^3
100-year Controlled Roof Release	14.9 L/s
Total Area	**0.338 ha**
Total Volume Retained	**63 m^3**
Total 100-year Release	**46.8 L/s**

Proposed Bachman Terrace Building (Block 1)

Rating Curve				Volume Estimation (Conical)				
Elevation (m)	Discharge Rate (m^3/s)	Outlet Discharge (m^3/s)	Storage (m^3)	Elevation (m)	Area (m^2)	Volume (m^3) Increment	Volume (m^3) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0.0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0.6	0.025	68	0.6	0.6	0.025
0.050	0.0008	0.0015	2.3	0.050	136	1.7	2.3	0.050
0.075	0.0011	0.0023	5.1	0.075	204	2.8	5.1	0.075
0.100	0.0015	0.0030	9.1	0.100	272	4.0	9.1	0.100
0.125	0.0019	0.0038	14.2	0.125	340	5.1	14.2	0.125
0.150	0.0023	0.0046	20.4	0.150	408	6.2	20.4	0.150

Drawdown Estimate

Total Volume (m^3)	Total Time (sec)	Vol (m^3)	Detention Time (hr)
0.6	746	0.6	0.2
2.3	1,104	1.7	0.5
5.1	1,243	2.8	0.9
9.1	1,305	4.0	1.2
14.2	1,342	5.1	1.6
20.4	1,367	6.2	2.0

Roof-top Storage Summary		
Total Building Area (ha)		0.051
Total Building Area (m^2)		510
Assume Available Roof Area (m^2)	80%	408
Roof Imperviousness		100%
Roof Drain Requirement (m^2/notch)		232
Number of Roof Notches*		2
Maximum Allowable Depth of Roof Ponding (m)		0.15
Maximum Available Storage (m^3)		20
Estimated 100 Year Drawdown Time (h)		1.7

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

Modelling Results

	5yr	100yr	Available
Qresult (m^3/s)	0.0027	0.0040	-
Depth (m)	0.090	0.133	0.15
Volume (m^3)	7	16	20
Draintime (hrs)	1.1	1.7	-

Modified Rational Method Calculations

5 yr Intensity
City of Ottawa

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

a = 998.071
b = 6.053
c = 0.814

Area (ha): 0.051
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.2	13.3	2.7	10.6	6.3
15	83.6	10.7	2.7	7.9	7.1
20	70.3	9.0	2.7	6.2	7.5
30	53.9	6.9	2.7	4.1	7.5
40	44.2	5.6	2.7	2.9	7.0
50	37.7	4.8	2.7	2.1	6.2
60	32.9	4.2	2.7	1.5	5.3
70	29.4	3.7	2.7	1.0	4.3
80	26.6	3.4	2.7	0.7	3.1
90	24.3	3.1	2.7	0.4	2.0
100	22.4	2.9	2.7	0.1	0.7
110	20.8	2.7	2.7	0.0	0.0
120	19.5	2.5	2.5	0.0	0.0
130	18.3	2.3	2.3	0.0	0.0
140	17.3	2.2	2.2	0.0	0.0

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (m^3)	Vavail (m^3)	Discharge Check
5-year Water Level	90	0.090	2.74	7	20	0.0

100 yr Intensity
City of Ottawa

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

a = 1735.688
b = 6.014
c = 0.820

Area (ha): 0.051
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	25.3	4.0	21.3	12.8
15	142.9	20.3	4.0	16.2	14.6
20	120.0	17.0	4.0	13.0	15.6
30	91.9	13.0	4.0	9.0	16.2
40	75.1	10.7	4.0	6.6	15.9
50	64.0	9.1	4.0	5.0	15.1
60	55.9	7.9	4.0	3.9	14.0
70	49.8	7.1	4.0	3.0	12.7
80	45.0	6.4	4.0	2.3	11.2
90	41.1	5.8	4.0	1.8	9.7
100	37.9	5.4	4.0	1.3	8.0
110	35.2	5.0	4.0	1.0	6.3
120	32.9	4.7	4.0	0.6	4.5
130	30.9	4.4	4.0	0.3	2.7
140	29.2	4.1	4.0	0.1	0.8

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (m^3)	Vavail (m^3)	Discharge Check
100-year Water Level	133	0.133	4.04	16	20	0.0

Stantec

Proposed Bachman Terrace Building (Block 2)

Rating Curve				Volume Estimation (Conical)				
Elevation (m)	Discharge Rate (m^3/s)	Outlet Discharge (m^3/s)	Storage (m^3)	Elevation (m)	Area (m^2)	Volume (m^3) Increment	Volume (m^3) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0.0	0.000	0	0	0	0.000
0.025	0.0004	0.0011	0.7	0.025	87	0.7	0.7	0.025
0.050	0.0008	0.0023	2.9	0.050	173	2.2	2.9	0.050
0.075	0.0011	0.0034	6.5	0.075	260	3.6	6.5	0.075
0.100	0.0015	0.0046	11.6	0.100	347	5.1	11.6	0.100
0.125	0.0019	0.0057	18.1	0.125	433	6.5	18.1	0.125
0.150	0.0023	0.0068	26.0	0.150	520	7.9	26.0	0.150

Drawdown Estimate

Total Volume (m^3)	Total Time (sec)	Vol (m^3)	Detention Time (hr)
0.7	634	0.7	0.2
2.9	938	2.2	0.4
6.5	1,056	3.6	0.7
11.6	1,109	5.1	1.0
18.1	1,140	6.5	1.4
26.0	1,161	7.9	1.7

Roof-top Storage Summary		
Total Building Area (ha)		0.065
Total Building Area (m^2)		650
Assume Available Roof Area (m^2)	80%	520
Roof Imperviousness		100%
Roof Drain Requirement (m^2/notch)		232
Number of Roof Notches*		3
Maximum Allowable Depth of Roof Ponding (m)		0.15
Maximum Available Storage (m^3)		26
Estimated 100 Year Drawdown Time (h)		1.4

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

Modelling Results

	5yr	100yr	Available
Qresult (m^3/s)	0.0040	0.0059	-
Depth (m)	0.087	0.129	0.15
Volume (m^3)	9	19	26
Draintime (hrs)	0.9	1.4	-

Modified Rational Method Calculations

5 yr Intensity
City of Ottawa

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

a = 998.071
b = 6.053
c = 0.814

Area (ha): 0.065
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.2	16.9	4.0	13.0	7.8
15	83.6	13.6	4.0	9.6	8.7
20	70.3	11.4	4.0	7.5	8.9
30	53.9	8.8	4.0	4.8	8.6
40	44.2	7.2	4.0	3.2	7.7
50	37.7	6.1	4.0	2.2	6.5
60	32.9	5.4	4.0	1.4	5.0
70	29.4	4.8	4.0	0.8	3.4
80	26.6	4.3	4.0	0.3	1.7
90	24.3	4.0	4.0	0.0	0.0
100	22.4	3.6	3.6	0.0	0.0
110	20.8	3.4	3.4	0.0	0.0
120	19.5	3.2	3.2	0.0	0.0
130	18.3	3.0	3.0	0.0	0.0
140	17.3	2.8	2.8	0.0	0.0

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (m^3)	Vavail (m^3)	Discharge Check
5-year Water Level	87	0.087	3.97	9	26	0.0

100 yr Intensity
City of Ottawa

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

a = 1735.688
b = 6.014
c = 0.820

Area (ha): 0.065
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	32.3	5.9	26.4	15.8
15	142.9	25.8	5.9	19.9	17.9
20	120.0	21.7	5.9	15.8	19.0
30	91.9	16.6	5.9	10.7	19.3
40	75.1	13.6	5.9	7.7	18.5
50	64.0	11.6	5.9	5.7	17.0
60	55.9	10.1	5.9	4.2	15.2
70	49.8	9.0	5.9	3.1	13.1
80	45.0	8.1	5.9	2.2	10.8
90	41.1	7.4	5.9	1.5	8.4
100	37.9	6.8	5.9	1.0	5.8
110	35.2	6.4	5.9	0.5	3.2
120	32.9	5.9	5.9	0.1	0.5
130	30.9	5.6	5.6	0.0	0.0
140	29.2	5.3	5.3	0.0	0.0

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (m^3)	Vavail (m^3)	Discharge Check
100-year Water Level	129	0.129	5.88	19	26	0.0

Stantec

Proposed Bachman Terrace Building (Block 3)

Rating Curve				Volume Estimation (Conical)				
Elevation (m)	Discharge Rate (m^3/s)	Outlet Discharge (m^3/s)	Storage (m^3)	Elevation (m)	Area (m^2)	Volume (m^3) Increment	Volume (m^3) Accumulated	Water Depth (m)
0.000	0.0000	0.0000	0.0	0.000	0	0	0	0.000
0.025	0.0004	0.0011	0.3	0.025	41	0.3	0.3	0.025
0.050	0.0008	0.0023	1.4	0.050	83	1.0	1.4	0.050
0.075	0.0011	0.0034	3.1	0.075	124	1.7	3.1	0.075
0.100	0.0015	0.0046	5.5	0.100	165	2.4	5.5	0.100
0.125	0.0019	0.0057	8.6	0.125	207	3.1	8.6	0.125
0.150	0.0023	0.0068	12.4	0.150	248	3.8	12.4	0.150

Drawdown Estimate

Total Volume (m^3)	Total Time (sec)	Vol (m^3)	Detention Time (hr)
0.3	302	0.3	0.1
1.4	447	1.0	0.2
3.1	504	1.7	0.3
5.5	529	2.4	0.5
8.6	544	3.1	0.6
12.4	554	3.8	0.8

Roof-top Storage Summary		
Total Building Area (ha)		0.031
Total Building Area (m^2)		310
Assume Available Roof Area (m^2)	80%	248
Roof Imperviousness		100%
Roof Drain Requirement (m^2/notch)		232
Number of Roof Notches*		3
Maximum Allowable Depth of Roof Ponding (m)		0.15
Maximum Available Storage (m^3)		12
Estimated 100 Year Drawdown Time (h)		0.5

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

Modelling Results

	5yr	100yr	Available
Qresult (m^3/s)	0.0033	0.0050	-
Depth (m)	0.072	0.109	0.15
Volume (m^3)	3	7	12
Draintime (hrs)	0.3	0.5	-

Modified Rational Method Calculations

5 yr Intensity
City of Ottawa

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

a = 998.071
b = 6.053
c = 0.814

Area (ha): 0.031
C: 0.90

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	104.2	8.1	3.3	4.8	2.9
15	83.6	6.5	3.3	3.2	2.9
20	70.3	5.4	3.3	2.2	2.6
30	53.9	4.2	3.3	0.9	1.6
40	44.2	3.4	3.3	0.2	0.4
50	37.7	2.9	2.9	0.0	0.0
60	32.9	2.6	2.6	0.0	0.0
70	29.4	2.3	2.3	0.0	0.0
80	26.6	2.1	2.1	0.0	0.0
90	24.3	1.9	1.9	0.0	0.0
100	22.4	1.7	1.7	0.0	0.0
110	20.8	1.6	1.6	0.0	0.0
120	19.5	1.5	1.5	0.0	0.0
130	18.3	1.4	1.4	0.0	0.0
140	17.3	1.3	1.3	0.0	0.0

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (m^3)	Vavail (m^3)	Discharge Check
5-year Water Level	72	0.072	3.28	3	12	0.0

100 yr Intensity
City of Ottawa

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

a = 1735.688
b = 6.014
c = 0.820

Area (ha): 0.031
C: 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)
10	178.6	15.4	5.0	10.4	6.3
15	142.9	12.3	5.0	7.3	6.6
20	120.0	10.3	5.0	5.4	6.4
30	91.9	7.9	5.0	3.0	5.3
40	75.1	6.5	5.0	1.5	3.6
50	64.0	5.5	5.0	0.5	1.6
60	55.9	4.8	4.8	0.0	0.0
70	49.8	4.3	4.3	0.0	0.0
80	45.0	3.9	3.9	0.0	0.0
90	41.1	3.5	3.5	0.0	0.0
100	37.9	3.3	3.3	0.0	0.0
110	35.2	3.0	3.0	0.0	0.0
120	32.9	2.8	2.8	0.0	0.0
130	30.9	2.7	2.7	0.0	0.0
140	29.2	2.5	2.5	0.0	0.0

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (m^3)	Vavail (m^3)	Discharge Check
100-year Water Level	109	0.109	4.97	7	12	0.0

C.2 CULVERT DESIGN SHEET

STORM DESIGN SHEET
(City of Ottawa)

SITE: **BACHMAN TERRACE**

REVISION DATE:
DATE: 6/4/2015
DESIGNED BY: SGG
CHECKED BY: DCT

FILE: 160401069

DESIGN PARAMETERS

DESIGN STORM **1 in 50 Years(Culverts)**

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

a=	1569.58	MANNING'S n =	0.024	CMP (unpaved)
b=	6.014	TIME OF ENTRY	10	min
c=	0.8200			

LOCATION			DRAINAGE AREA								CULVERT PROPERTIES							
STREET	FROM	TO	AREA (ha)	C	ACCUM. AREA	A x C (ha)	ACCUM. AxC (ha)	T of C (min)	I (mm/h)	Q* (l/s)	LENGTH (m)	Pipe Size (mm)	MATERIAL	SLOPE %	Q_{CAP} (FULL) (L/s)	Q_{ACT} / Q_{CAP} % FULL	VELOCITY (m/s)	TIME OF FLOW (min)
Bachman Terrace	A	B	0.185	0.55	0.185	0.10	0.102	10.00 10.30	161.47	**51.52**	8.0	400.0	CSP	0.25	56.40	91.34%	0.45	0.30

*NOTE: Peak flow rate includes 5.88L/s max roof release rate from Block 2

Appendix D : City Correspondence

D.1 DESIGN CRITERIA

Gillis, Sheridan

From: Whittaker, Damien <Damien.Whittaker@ottawa.ca>
Sent: Wednesday, March 19, 2014 1:08 PM
To: Gillis, Sheridan
Cc: Wilkie, Tim
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Sheridan,

On the basis that the architect states that the exterior is cladding brick and siding the proposal does not appear to fully satisfy the requirement of masonry or non-combustible material.

Please feel free to ask for clarification, or further information, on any of the comments above.

Thank you,

Damien Whittaker, P.Eng ▪ **Project Manager** ▪ **Development Review, Suburban West Sub-unit**
City of Ottawa ▪ 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
☎ 613-580-2424 x16968 ▪ damien.whittaker@ottawa.ca ▪ 01-14

From: Gillis, Sheridan [mailto:Sheridan.Gillis@stantec.com]
Sent: Monday, March 17, 2014 3:25 PM
To: Whittaker, Damien
Cc: Wilkie, Tim
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Hi Damien,
I'm wondering if you can make a ruling on the construction type for the proposed Bachman Terrace townhouse units. Our architect has provided us with the following details:

Bachman Townhomes

-OBC- Part 9

- Occupancy type-C, Townhomes
- Each townhome separated by a 1Hr. fire separation
- Building is divided into 3 buildings by a 2 Hr. firewall each having a building area of less than 600 m sq.
- No dwelling unit above another dwelling unit
- 3 storey building height
- Combustible construction
- Exterior cladding brick & siding

I've attached the FUS criteria as well (though I'm sure you have a copy on hand). The language is somewhat ambiguous, but ultimately if it's wood frame we can make it work, but it will mean a larger main within the private street and a larger chamber at our connection point. I'll give you a call to discuss further.
Thanks,
Sheridan

From: Whittaker, Damien [mailto:Damien.Whittaker@ottawa.ca]
Sent: Wednesday, March 12, 2014 8:33 AM
To: Gillis, Sheridan

Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Sheridan,

Please find boundary conditions below.

Boundary conditions for existing system based on hydraulic modelling are as follows for both connection points:

Max HGL = 165.1m
PKHR = 155.3m
MXDY+Fire (FUS) = 150.9m
MXDY+Fire (OBC) = 156.8m

Notes:

- boundary conditions are relatively insensitive to fire demand because of the 16” local main.
- Multiple hydrants will be required to deliver the fire flow.

Please feel free to ask for clarification, or further information, on any of the comments above.

Thank you,

Damien Whittaker, P.Eng ▪ **Project Manager** ▪ **Development Review, Suburban West Sub-unit**
City of Ottawa ▪ 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
☎ 613-580-2424 x16968 ▪ damien.whittaker@ottawa.ca ▪ 01-14

From: Gillis, Sheridan [mailto:Sheridan.Gillis@stantec.com]
Sent: Monday, March 10, 2014 3:54 PM
To: Whittaker, Damien
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Damien,
Thanks for the phone call to clarify. Attached is the updated FUS requirements with construction coefficient of 1.5 (wood frame). I think there’s a case to be made for ordinary construction for these units but at this point we really just need the boundary conditions to get an idea of where we are with respect to pressures. Once we know where we are we can look at measures to mitigate requirements if necessary (sprinkler/additional fire wall/construction materials). Again, we’d like the boundary conditions for both the OBC and FUS requirements.
Thanks again,
Sheridan

From: Whittaker, Damien [mailto:Damien.Whittaker@ottawa.ca]
Sent: Monday, March 10, 2014 12:52 PM
To: Gillis, Sheridan
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Sheridan,

The boundary condition request came back unanswered based on the comments of February 21. Copied again here; Please note that using C=1 (ordinary construction) requires supporting discussion as houses are assumed by City staff as wood frame with C=1.5. Please address the parameter usage in your reponse.

Please feel free to ask for clarification, or further information, on any of the comments above.

Thank you,

Damien Whittaker, P.Eng ▪ **Project Manager** ▪ **Development Review, Suburban West Sub-unit**

City of Ottawa ▪ 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
☎ 613-580-2424 x16968 ▪ damien.whittaker@ottawa.ca ▪ 01-14

From: Gillis, Sheridan [mailto:Sheridan.Gillis@stantec.com]
Sent: Tuesday, February 25, 2014 11:35 AM
To: Whittaker, Damien
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Hi Damien,
As requested, attached is the FUS calculation for Bachman Terrace. We'd actually like to receive boundary conditions for both the OBC requirements as well as the FUS requirements. I know there's some ongoing discussion regarding the application of the FUS criteria for this type of development but I suppose at this point we'll get boundary conditions for both and see what the analysis determines with respect to pressures. Anyway, whenever you get a free minute could you give me a call, I'd like to quickly discuss this and a couple of other minor related items.
Best Regards,
Sheridan

From: Whittaker, Damien [mailto:Damien.Whittaker@ottawa.ca]
Sent: Monday, February 24, 2014 8:44 AM
To: Gillis, Sheridan
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Sheridan,

Please provide fire demand based on FUS as per clause 4.2.11 of the Ottawa Design Guidelines - Water Distribution, First Edition, Document WDG001, July 2010, City of Ottawa (Water Guidelines), including Technical Bulletin ISD-2010-2, December 15, 2010 for the pipe sizing connecting to the property rather than the Ontario Building Code that applies to private property only.

Thank you,

Damien Whittaker, P.Eng ▪ **Project Manager** ▪ **Development Review, Suburban West Sub-unit**
City of Ottawa ▪ 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
☎ 613-580-2424 x16968 ▪ damien.whittaker@ottawa.ca ▪ 01-14

From: Gillis, Sheridan [mailto:Sheridan.Gillis@stantec.com]
Sent: Friday, February 21, 2014 4:35 PM
To: Whittaker, Damien
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Hi Damien,
Apologies, I should have included the calc's in the original e-mail. Attached are the requested fire demand calculations using the obc guidelines to determine the fire flow. Just let us know if you (or water group) has any questions,
Have a great weekend,
Sheridan

From: Whittaker, Damien [mailto:Damien.Whittaker@ottawa.ca]
Sent: Friday, February 21, 2014 3:52 PM
To: Gillis, Sheridan
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: RE: Water Boundary Conditions for 23 Bachman Terrace

Sheridan,

My request to the unit supplying boundary conditions was returned unanswered as no fire demand calculations were provided. Please note that using C=1 (ordinary construction) requires supporting discussion as houses are assumed by City staff as wood frame with C=1.5.

Please feel free to ask for clarification, or further information, on any of the comments above.

Thank you,

Damien Whittaker, P.Eng ▪ **Project Manager** ▪ **Development Review, Suburban West Sub-unit**
City of Ottawa ▪ 110 Laurier Avenue West, Ottawa, Ontario K1P 1J1
☎ 613-580-2424 x16968 ▪ damien.whittaker@ottawa.ca ▪ 01-14

From: Gillis, Sheridan [mailto:Sheridan.Gillis@stantec.com]
Sent: Friday, February 14, 2014 1:44 PM
To: Whittaker, Damien
Cc: Wilkie, Tim; Thiffault, Dustin
Subject: Water Boundary Conditions for 23 Bachman Terrace

Hi Damien,
We've run through some preliminary water demand calculations for the Tega Development at 23 Bachman Terrace, and we're hoping you can provide us with the existing boundary conditions based on our numbers below. I've attached a sketch of the development showing our proposed wtm servicing concept for your reference. The preliminary water demands are as follows:

- Avg. Day = 0.27l/s
- Max. Day = 0.69l/s
- Peak Hour = 1.52l/s
- Fire Flow = 6,300l/min

If you have any questions, or need any additional information feel free to call,
Thanks,
Sheridan

Sheridan Gillis
Senior Designer, Urban Land Engineering

Stantec
400 - 1331 Clyde Avenue Ottawa ON K2C 3G4
Phone: (613) 722-4420
sheridan.gillis@stantec.com

Houle Chevrier Engineering

Houle Chevrier Engineering Ltd.
180 Wescar Lane
Ottawa, Ontario
K0A 1L0

tel: 613.836.1422
fax: 613.836.9731
info@hceng.ca
www.hceng.ca

Tega Developments
66 Colonnade Road, Suite 200
Ottawa, Ontario
K2E 7K7

June 25, 2014
Project: 13-042

Attention: Mr. Spyro Dimitrakopoulos

**Re: Response to City of Ottawa Comments
Proposed Redevelopment
19 and 23 Bachman Terrace
Ottawa, Ontario**

The following provides our response to the City of Ottawa letter dated June 4, 2014. The letter was prepared by City of Ottawa staff, in part, following a review of our geotechnical report titled: “Geotechnical Investigation Report, Proposed Redevelopment, 19 and 23 Bachman Terrace, Ottawa, Ontario”, dated February 12, 2014.

The comments raised by City of Ottawa staff with respect to our geotechnical report are italicized below and we have provided our response for each item raised by the City of Ottawa.

Item 1

Please revise section 2.1 that does not specifically mention the current and future state of all buildings on the addresses.

We have added the number and type of existing buildings to Section 2.1, including which of the existing buildings will be demolished to accommodate the proposed development.

Item 2

Please embed a picture of the geology map referenced in section 2.2 or provide a reference.

We have added a reference to the urban geology database of Canada’s National Capital Region (Geological Survey of Canada, Open File 2878, 1994) to Section 2.2.

Item 3

Please revise the first bullet point of section 3 that references six proposed dwellings.

Reference to “six proposed dwellings” has been removed from Section 3.0

Item 4

Please provide engineering justification for the OHSA classification for Type 3 within section 5.3.1.

The overburden at this site was classified as Type 3 soil based on our visual and physical examination of the soil exposed in the test pits. Furthermore, fill material (i.e., previously excavated soil) was encountered within some of the test pits (e.g., test pit 3), which, according to the Occupational Health and Safety Act is classified as Type 3 soil. Justification for the classification for Type 3 soil has been added to Section 5.3.1.

Item 5

Please clarify the term sumps in section 5.3.1.

Sumps are low points within the base of the excavation where water collects. Clarification has been made in Section 5.3.1.

Item 6

The proposed pavement design of section 5.5.2 does not match that presented in the drawings set for the project.

Refer to our response to Item 8.

Item 7

It is suggested that the statistical risk of the conditions presented in section 5.5.4 occurring are higher than low. Please revise the pavement structure and prescribe a specific, revised pavement design.

The pavement design provided in our report is appropriate for the intended use of the access roadway. However, as indicated in Section 5.5.4, if the subgrade surface becomes disturbed or wetted due to construction operations or precipitation, it may be necessary to increase the thickness of the subbase and/or to incorporate a woven geotextile separator. Since the degree of subgrade disturbance and weather conditions cannot be predicted, in our opinion, the preferred approach from a geotechnical point of view is to:

- Proof roll the subgrade conditions at the time of construction under the supervision of experienced geotechnical personnel.

- Adjust the thickness of the subbase material and include a nonwoven geotextile separator, as required, at the time of construction. Unit rate allowances should be made in the contract for subexcavation and replacement with OPSS Granular B Type II and a woven geotextile.

In our opinion, it is over-conservative to assume “worst case” conditions (e.g., wet and poor subgrade conditions) at the pavement design stage and doing so may result in a thicker (and much more costly) pavement structure than actually required at the time of construction.

No revisions have been made to pavement structure; however, clarification has been added to Section 5.5.4.

Item 8

Please provide certification that the geotechnical engineer is satisfied with the civil design as per section 5.8.

We will review the design drawings once they are provided to us. Our comments will be provided in a separate letter.

We trust that this letter is sufficient for your purposes. If you have any questions concerning this information, please call.

Johnathan A. Cholewa, Ph.D., P.Eng.

A.C. Houle, M.Eng., P.Eng.
Principal

Appendix E : Geotechnical Investigation

E.1 REPORT EXCERPTS: GEOTECHNICAL INVESTIGATION – PROPOSED REDEVELOPMENT 19 AND 23 BACHMAN TERRACE OTTAWA, ONTARIO – JUNE 25, 2014

Houle Chevrier Engineering Ltd.
180 Wescar Lane
R.R. 2
Carp, Ontario K0A 1L0
Tel: (613) 836-1422
Fax: (613) 836-9731
www.hceng.ca

REPORT ON

GEOTECHNICAL INVESTIGATION
PROPOSED REDEVELOPMENT
19 AND 23 BACHMAN TERRACE
OTTAWA, ONTARIO

Submitted to:

Tega Developments
66 Colonnade Road, Suite 200
Ottawa, Ontario
K2E 7K7

DISTRIBUTION:

5 bound copies - Tega Developments
1 electronic copy - Tega Developments
2 bound copies - Houle Chevrier Engineering Ltd.

June 25, 2014 Our ref: 13-042

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation carried out at the site of the proposed twenty-five (25) row house development at 19 and 23 Bachman Terrace in Ottawa, Ontario. The purpose of the investigation was to identify the general subsurface conditions at the site by means of a limited number of test pits and, based on the factual information obtained, to provide engineering guidelines on the geotechnical design aspects of the proposed development, including construction considerations that could influence design decisions.

This investigation was carried out in accordance with our proposal dated January 10, 2013.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

Plans are being prepared to construct twenty-five (25) row house residential dwellings at 19 and 23 Bachman Terrace in Ottawa, Ontario (see Key Plan, Figure 1). Twenty (20) units will be constructed on the property located at 23 Bachman Terrace, and the remaining five (5) to be constructed on a portion of property located at 19 Bachman Terrace. Currently, the following buildings exist on 23 Bachman Terrace:

- One (1) two storey single family residential dwelling services, which is serviced by a drilled well and septic system; and
- Two (2) accessory buildings (i.e., sheds).

The following buildings currently exist on 19 Bachman Terrace:

- One (1) two storey single family residential dwelling; and
- Three (3) accessory buildings.

It is understood that the proposed dwellings will consist of slab on grade construction and will be serviced by municipal water and sewer. A new access roadway is also included in the scope of the project. With the exception of the existing two storey residential dwelling on 19 Bachman Terrace, all of the existing buildings on 19 and 23 Bachman Terrace are to be demolished.

2.2 Review of Geology Maps

Based on the geology maps provided in the urban geology database of Canada's National Capital Region (Geological Survey of Canada, Open File 2878, 1994), the subject site is underlain by shallow/exposed interbedded sandstone and dolostone bedrock of the March formation.

3.0 SUBSURFACE INVESTIGATION

The field work for this investigation was carried out on March 7, 2013. During that time, a total of four (4) test pits were excavated across the site using a 4 ton CAT 304 track mounted excavator supplied and operated by KingEx Landscaping and Excavating of Kemptville, Ontario. Details for the test pits are provided below:

- Four (4) test pits, numbered 13-1 to 13-4, inclusive, were advanced to between 0.4 and 1.2 metres below ground surface within 23 Bachman Terrace for foundation design purposes.

The subsurface conditions in the test pits were identified by visual and tactile examination of the materials exposed on the sides and bottom of the test pits. The groundwater conditions in the open test pits were observed on completion of excavating. The field work was observed throughout by a member of our engineering staff, who directed the excavation and logged the test pits.

Descriptions of the subsurface conditions logged in the test pits are provided on the Record of Test Pit sheets in Appendix A. The approximate locations of the test pits are shown on the Test Pit Location Plan, Figure 2.

The test pit locations were selected by Houle Chevrier Engineering Ltd. personnel and positioned at the site using a Trimble R8 GPS survey instrument. The ground surface elevations at the test pits were also determined using a Trimble R8 GPS survey instrument. The elevations are referenced to Geodetic datum.

4.0 SUBSURFACE CONDITIONS

4.1 General

The soil and groundwater conditions logged in the test pits are given on the Record of Test Pit sheets in Appendix A. The test pit logs indicate the subsurface conditions at the specific test locations only. Boundaries between zones on the logs are often not distinct, but rather are transitional and have been interpreted. Subsurface conditions at other than the test pit locations may vary from the conditions encountered in the test pits. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site.

The soil descriptions in this report are based on commonly accepted methods of classification and identification employed in geotechnical practice. Classification and identification of soil involves judgement and Houle Chevrier Engineering Ltd. does not guarantee descriptions as exact, but infers accuracy to the extent that is common in current geotechnical practice.

The following presents an overview of the subsurface conditions encountered in the test pits advanced during this investigation.

4.2 Topsoil

A surficial layer of topsoil, having a thickness of about 0.3 metres, was encountered in test pit 13-2.

4.3 Fill Material

A surficial layer of topsoil fill, having a thickness of about 0.1 and 0.4 metres, was encountered in test pits 13-1, 13-3, and 13-4.

At test pit 13-3, the topsoil fill is underlain by about 0.7 metres of fill material composed of brown silty sand followed by reddish grey to grey brown gravel with variable amounts of sand and silt. At test pit 13-4, the topsoil fill is underlain by 0.1 metres of brown silty sand fill containing miscellaneous debris.

4.4 Former Topsoil

A 0.1 metre thick former topsoil layer, composed of dark brown sandy silt with variable amounts of organic material, was encountered in test pit 13-4 below the fill material at 0.2 metres below ground surface.

4.5 Glacial Till

At test pits 13-2 and 13-4, a deposit of glacial till was encountered below the topsoil and former topsoil at 0.3 metres below ground surface. The glacial till can generally be described as brown to grey brown silty sand with variable amounts of gravel. Cobbles and boulders should also be expected in the glacial till. At test pits 13-2 and 13-4, the glacial till has a thickness of about 0.6 and 0.3 metres, respectively.

4.6 Fractured/Inferred Bedrock

A layer of fractured/weathered bedrock was encountered below the glacial till in test pits 13-2 and 13-4 at 0.9 and 0.6 metres below ground surface, respectively (elevation 117.0 and 117.6 metres, geodetic datum). The fractured/weathered bedrock was excavated with the 4 ton CAT 304 track mounted excavator with little effort. At test pits 13-2 and 13-4, the fractured/weathered bedrock has a thickness of 0.3 metres.

Practical excavator refusal to further advancement of test pits 13-1 to 13-4, inclusive, occurred on the inferred surface of the bedrock occurred at depths ranging between 0.4 and 1.2 metres below ground surface (elevation 116.7 to 117.5 metres, geodetic datum).

It should be noted that practical excavator refusal can sometimes occur within cobbles and boulders and may not necessarily be representative of the upper surface of the bedrock.

4.7 Groundwater Conditions

No groundwater inflow was observed in the test pits during the relatively short period of time they were left open following excavation on March 7, 2013.

It should be noted that groundwater levels will fluctuate seasonally and may be higher during wet periods of the year, such as the early spring or fall, or following periods of heavy precipitation.

5.0 GEOTECHNICAL DESIGN GUIDELINES

5.1 General

The information in the following sections is provided for the guidance of the design engineers and is intended for the design of this project only. While the results of the geotechnical investigation carried out at the site by Houle Chevrier Engineering Ltd. are considered adequate to provide geotechnical recommendations for the proposed development, it is noted that the subsurface conditions (e.g., thickness of fill material, bedrock depth) present within 19 Bachman Terrace could vary from the subsurface conditions encountered in test pits 13-1 to 13-4, inclusive.

Contractors bidding on or undertaking the works should examine the factual results of the investigation, satisfy themselves as to the adequacy of the information for construction, and make their own interpretation of the factual data as it affects their construction techniques, schedule, safety and equipment capabilities.

The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous uses or activities of this site or adjacent properties, and/or resulting from the introduction onto the site of materials from off site sources are outside the terms of reference for this report.

5.2 Removal of Existing Septic System

The existing septic tank, and associated fill materials, deleterious material or topsoil should be removed from below any foundations and concrete slabs to expose undisturbed native soil or bedrock. Furthermore, any distribution piping should also be removed. The grade below the proposed building could then be raised with imported granular material conforming to OPSS Granular B Type II compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the proposed building be composed of virgin material only for environmental reasons. To provide adequate spread of load below the footings, the material should extend at least 0.5 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

If the existing septic tank is not located below foundations or hard surfaced areas (concrete slabs on grade, pavement etc.), the existing tank could be filled with 19 millimetre clear stone and the access lids placed back in place. Alternatively, the tank and distribution piping could be removed as described above.

5.3 Proposed Buildings

5.3.1 Excavation

The excavation for the proposed buildings will be carried out through topsoil, fill material, glacial till, and possibly fractured/weathered bedrock. For excavations exceeding 1.2 metres depth, the sides should be sloped in accordance with the requirements in Ontario Regulation 213/91 under the Occupational Health and Safety Act. Based on our visual and physical examination of the soil exposed in the test pits, and given that portions of the overburden consists of previously excavated soil (fill material), in our opinion, the soils at this site can be classified as Type 3. As such, open cut excavations within the overburden having a depth of greater than 1.2 metres should be carried out with walls sloped at 1 horizontal to 1 vertical, or flatter, from the base of the excavation

The groundwater inflow from the overburden deposits, if any, should be controlled by pumping from sumps within the excavation (i.e., low points within the base of the excavation where water collects).

5.3.2 Spread Footing Design

Based on the results of the investigation, the proposed structures could be founded on spread footings. The fill materials, topsoil and former topsoil is considered to be highly compressible and should be removed from below the foundations and concrete slabs. Furthermore, foundation elements from the existing structure, building rubble, the existing septic tank and distribution piping, and any fill materials should also be removed from the building areas.

The following alternatives could be considered for the spread footings:

1) Spread footings bearing on or within native, undisturbed native glacial till deposits or engineered fill above native, undisturbed soil deposits; OR
2) Spread footings bearing on or within bedrock.

Alternative 1: Bearing on Native Soil or Engineered Fill above Native Soil

The native soil deposits (i.e., glacial till) could be considered for the support for the proposed structures. If required, the grade below the proposed buildings could be raised with imported granular material conforming to OPSS Granular B Type II compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the proposed building be composed of virgin material only for environmental reasons. To provide adequate spread of load below the footings, the granular material should extend at least 0.3 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.

Spread footing foundations bearing on or within native undisturbed deposits of glacial till, or on a pad of compacted granular fill above native, undisturbed glacial till deposits should be sized using an allowable bearing pressure of 150 kilopascals.

Alternative 2: Bearing on Bedrock

Based on the results of the investigation, the proposed structure could be founded on conventional spread footings placed either directly on the surface of the bedrock or on a pad of engineered fill above bedrock (i.e., at or above elevation 117.0 to 117.5 metres, geodetic datum).

In areas where the underside of footing level is above the level of the bedrock or where significant undulations exist in the bedrock, the grade below the proposed building could be raised with imported granular material conforming to OPSS Granular B Type II compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor dry density value. OPSS documents allow recycled asphaltic concrete and concrete to be used in Granular

B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the proposed building be composed of virgin material only for environmental reasons. To provide adequate spread of load below the footings, the material should extend at least 0.5 metres horizontally beyond the edge of the footings and down and out from this point at 1 horizontal to 1 vertical, or flatter.
Spread footings founded on a pad of compacted granular material above competent bedrock, or spread footings bearing directly on undisturbed weathered bedrock, should be sized using an allowable bearing pressure of 500 kilopascals. This bearing pressure assumes that all loose or disturbed soil and bedrock is removed from the bearing surfaces and that the pad of compacted granular material is prepared as described in this report.

In contrast, spread footings founded on or within competent bedrock should be sized using an allowable bearing pressure of 1,000 kilopascals. This bearing pressure should be confirmed at the time of construction and assumes that all soil and any fractured or disturbed bedrock is removed from the bearing surface.

The bearing pressures provided above are summarized in the table below:

Subgrade Material	Allowable Bearing Pressure (kilopascals)[1]
Glacial till, or on a pad of compacted crushed stone above glacial till	150
Pad of compacted granular material above undisturbed, fractured or competent bedrock	500[2]
Fractured bedrock	500[3]
Bedrock	1,000[3]

Notes:
1. These bearing pressures assume the subgrade surface is prepared as described in this report
2. The engineered fill must placed compacted as described in this report.
3. The bedrock should be inspected and approved by geotechnical personnel.

To reduce the potential for cracking in the footings, foundation walls, and concrete slabs on grade where the footings transition between different subgrade materials, the foundation walls should be suitably reinforced as specified by the structural engineer.

5.3.3 Frost Protection Requirements for Foundations

All exterior footings should be provided with at least 1.5 metres of earth cover for frost protection purposes. Isolated (unheated) piers that are located in areas that are to be cleared of snow should also be provided with at least 1.8 metres of earth cover for frost protection purposes. Alternatively, the required frost protection could be provided by means of a combination of earth cover and extruded polystyrene insulation. An insulation detail could be provided upon request.

The required frost protection could be waived for footings on relatively sound bedrock. Inspection of the bedrock by geotechnical personnel would be required to reduce or waive the frost protection.

5.3.4 Foundation Wall Backfill and Drainage

The native soils at this site are frost susceptible and should not be used as backfill against foundations. To avoid frost adhesion and possible heaving, the foundations should be backfilled with imported, free-draining, non-frost susceptible granular material such as that meeting OPSS Granular B Type I or II requirements.

Where the backfill will ultimately support areas of hard surfacing (sidewalks or other similar surfaces), the backfill should be placed in maximum 200 millimetre thick lifts and should be compacted to at least 95 percent of the standard Proctor maximum dry density value using suitable vibratory compaction equipment. Where future landscaped areas will exist next to the proposed structures and if some settlement of the backfill is acceptable, the backfill could be compacted to at least 90 percent of the standard Proctor maximum dry density value.

Where areas of hard surfacing (concrete, sidewalk, pavement, etc.) abut the proposed building, all topsoil, and fill material (including septic system materials etc.), should be removed to the level of relatively undisturbed native soil or bedrock. In the event that the hard surfaced areas are underlain by frost susceptible material, a gradual transition should be provided between those areas of hard surfacing underlain by non-frost susceptible granular wall backfill and those areas underlain by existing frost susceptible native materials to reduce the effects of differential frost heaving. It is suggested that granular frost tapers be constructed from the bedrock surface to the underside of the granular base/subbase material for the hard surfaced areas. The frost tapers should be sloped at 1 horizontal to 1 vertical, or flatter.

Perimeter foundation drainage is not considered necessary for a slab on grade structure at this site, provided that the floor slab level is above the finished exterior ground surface level.

5.3.5 Slab on Grade Support

Based on the test pits advanced during this investigation, the area of the proposed building is underlain by topsoil, fill material, and former topsoil followed by glacial till and bedrock. The topsoil, fill material, former topsoil and septic system materials are not considered suitable for the support of the slab on grade and should be removed from the area of the proposed building.

The grade within the proposed building could be raised, where necessary, with granular material meeting OPSS requirements for Granular B Type I or II. The granular base for the proposed slab on grade should consist of at least 150 millimetres of OPSS Granular A. City of Ottawa documents allow recycled asphaltic concrete and concrete to be used in Granular A and Granular B Type II materials. Since the source of recycled material cannot be determined, it is suggested that any granular materials used beneath the floor slab be composed of virgin material (100 percent crushed rock) only, for environmental reasons.

All imported granular materials placed below the proposed floor slab should be compacted in maximum 200 millimetre thick lifts to at least 95 percent of the standard Proctor maximum dry density value.

Underfloor drainage is not considered necessary provided that the floor slab level is above the finished exterior ground surface level.

If any areas of the buildings are to remain unheated during the winter period, thermal protection of the materials beneath the slab on grade may be required. Further details on the insulation requirements could be provided, if necessary. The required frost protection could be waived if the floor slab is underlain by non frost susceptible imported granular materials over bedrock. Inspection of the bedrock by geotechnical personnel would be required to reduce or waive the frost protection.

5.4 Site Services

Probable bedrock was encountered across the site at depths ranging between 0.4 and 0.9 metres below existing surface grade (about elevation 117.0 to 117.6 metres). As such, bedrock excavation may be required in order to install the site services.

Removal of the fractured/weathered bedrock could be carried out using large hydraulic excavation equipment. In contrast, the competent bedrock will likely require rock hammering (i.e., hoe ramming equipment). It is noted that the bedrock likely contains near vertical joints and bedding planes. Therefore, some vertical and horizontal over break of the bedrock should be expected and allowance should be made for the use of additional granular bedding. In addition, the bedrock type in this area is known to be hard and abrasive, and significant equipment wear should be expected.

Provided that good bedrock excavation techniques are used, the bedrock could be excavated using vertical side walls.

Flexible service pipes should be installed in accordance with Ontario Provincial Standard Drawing (OPSD) 802.013 for Type 1 Soil. The excavation for rigid service pipes should be in accordance with OPSD 802.033 for Type 1 Soil.

5.5 Proposed Access Roadway

5.5.1 Subgrade Preparation

In preparation for the construction of the access roadway, all surficial topsoil, and any loose/soft, wet, organic or deleterious materials should be removed from the proposed subgrade surface. This need not include the removal of the existing fill provided that some minor post construction settlement of the flexible (asphaltic concrete) pavement can be accommodated. Any settlement of the asphaltic concrete paving could be corrected by padding with asphaltic concrete, if necessary.

The subgrade surface for the pavement areas should be proof rolled with a large (10 tonne minimum) steel drum roller under dry conditions. Any soft areas exposed from the proof rolling should be subexcavated and replaced with suitable earth borrow. An assessment of the subgrade conditions within the roadway should be made by the geotechnical engineer at the time of construction.

The grade within the parking lot could be raised, where necessary, using suitable earth borrow or OPSS Select Subgrade Material. The earth borrow and Select Subgrade Material should be placed in maximum 300 millimetre thick lifts and compacted to at least 95 percent of the standard Proctor maximum dry density value using vibratory compaction equipment. The fill type and placement should be uniform to reduce the potential for differential frost heaving of the pavement.

The subgrade surface should be crowned and shaped to promote drainage of the granular materials to the catch basins, as discussed in Section 5.5.6.

5.5.2 Proposed Pavement Structure

It is suggested that parking areas to be used by light vehicles (cars, etc.) be constructed using the following minimum pavement structure:

50 millimetres of Superpave 12.5 asphaltic concrete, over
150 millimetres of OPSS Granular A base, over
300 millimetres of OPSS Granular B Type II subbase

The thickness of the Granular B Type II subbase could be reduced to a minimum of 150 millimetres where bedrock is encountered at subgrade level.

For any access roadways which will be used by truck traffic or fire trucks, the asphaltic concrete surfacing thickness should be increased to 80 millimetres (40 millimetres of Superpave 12.5 (Traffic Level B) over 40 millimetres of Superpave 19.0 (Traffic Level B)) and the thickness of the subbase layer increased to 375 millimetres.

In accordance with current practice in the City of Ottawa, performance graded PG 58-34 asphaltic concrete should be specified. An assessment of the subgrade conditions within the parking areas should be made by the geotechnical engineer at the time of construction.

5.5.3 Compaction Requirements

The granular base and subbase materials should be compacted in maximum 200 millimetre thick lifts to at least 98 percent of the standard Proctor maximum dry density value.

5.5.4 Effects of Soil Disturbance and Construction Traffic

The pavement structure provided above assumes that:

- The trench backfill is properly reinstated and adequately compacted as described in this report;
- The roadway subgrade surface is properly prepared and proof rolled at the time of construction (i.e., the subgrade is not disturbed or wetted due to construction operations or precipitation); and
- The base and subbase materials are not subjected to heavy construction truck traffic during the duration of the project and the contractor is made responsible for maintaining access during construction.

If the roadway subgrade surface becomes disturbed or wetted due to construction operations or precipitation, or the granular pavement materials are used as a haul road, the Granular B Type II thickness given above may not be adequate and it may be necessary to increase the thickness of the Granular B Type II subbase and/or to incorporate a woven geotextile separator between the roadway subgrade surface and the granular subbase material.

The required thickness of the subbase materials will depend on a number of factors, including contractor workmanship and schedule, contractor methodology, soil types and weather conditions. Since the degree of subgrade disturbance and weather conditions cannot be predicted, in our opinion, the preferred approach from a geotechnical point of view is to:

- Proof roll the subgrade conditions at the time of construction under the supervision of experienced geotechnical personnel.
- Adjust the thickness of the subbase material and include a nonwoven geotextile separator, as required. Unit rate allowances should be made in the contract for subexcavation and replacement with OPSS Granular B Type II and a woven geotextile.

5.5.5 Pavement Transitions

In areas where the new pavement will abut the existing pavement along Bachman Terrace, the depths of the granular materials should taper up or down at 5 horizontal to 1 vertical, or flatter, to match the depths of the granular material(s) exposed in the existing pavement.

5.5.6 Drainage of the Granular Materials

Adequate drainage of the pavement granular materials and subgrade is important for the long term performance of the pavement at this site. Where storm sewers are used to convey surface water runoff, catch basins should be provided with minimum 3 metre long perforated stub drains which extend in at least two directions from each catch basin at pavement subgrade level. Perimeter drainage is also suggested, where practicable.

5.6 Effects of Construction Induced Vibration

Some of the construction operations (such as granular material compaction, overburden excavation, rock hammering etc.) will cause ground vibration on and off of the site. The vibrations will attenuate with distance from the source, but may be felt at nearby structures. The vibration effects of excavator ramming are usually minor and localized. Monitoring of the hoe ramming could be carried out, at least initially, to measure the vibrations to ensure that they are below the acceptable threshold value.

5.7 Winter Construction

In the event that construction is required during freezing temperatures, the frost susceptible subgrade below the footings and slabs should be protected immediately from freezing using straw, propane heaters, polystyrene insulation, insulated tarpaulins, or other suitable means. Inspection of the bedrock by geotechnical personnel would be required to determine if the bedrock is frost susceptible.

5.8 Design Review and Construction Observation

The details for the proposed construction were not available to us at the time of preparation of this report. It is recommended that the design drawings be reviewed by the geotechnical engineer as the design progresses to ensure that the guidelines provided in this report have been interpreted as intended.

The engagement of the services of the geotechnical consultant during construction is recommended to confirm that the subsurface conditions throughout the proposed excavations do not materially differ from those given in the report and that the construction activities do not adversely affect the intent of the design. The subgrade surfaces for the buildings should be inspected by experienced geotechnical personnel to ensure that suitable materials have been reached and properly prepared. The placing and compaction of earth fill and imported granular materials should be inspected to ensure that the materials used conform to the grading and compaction specifications.

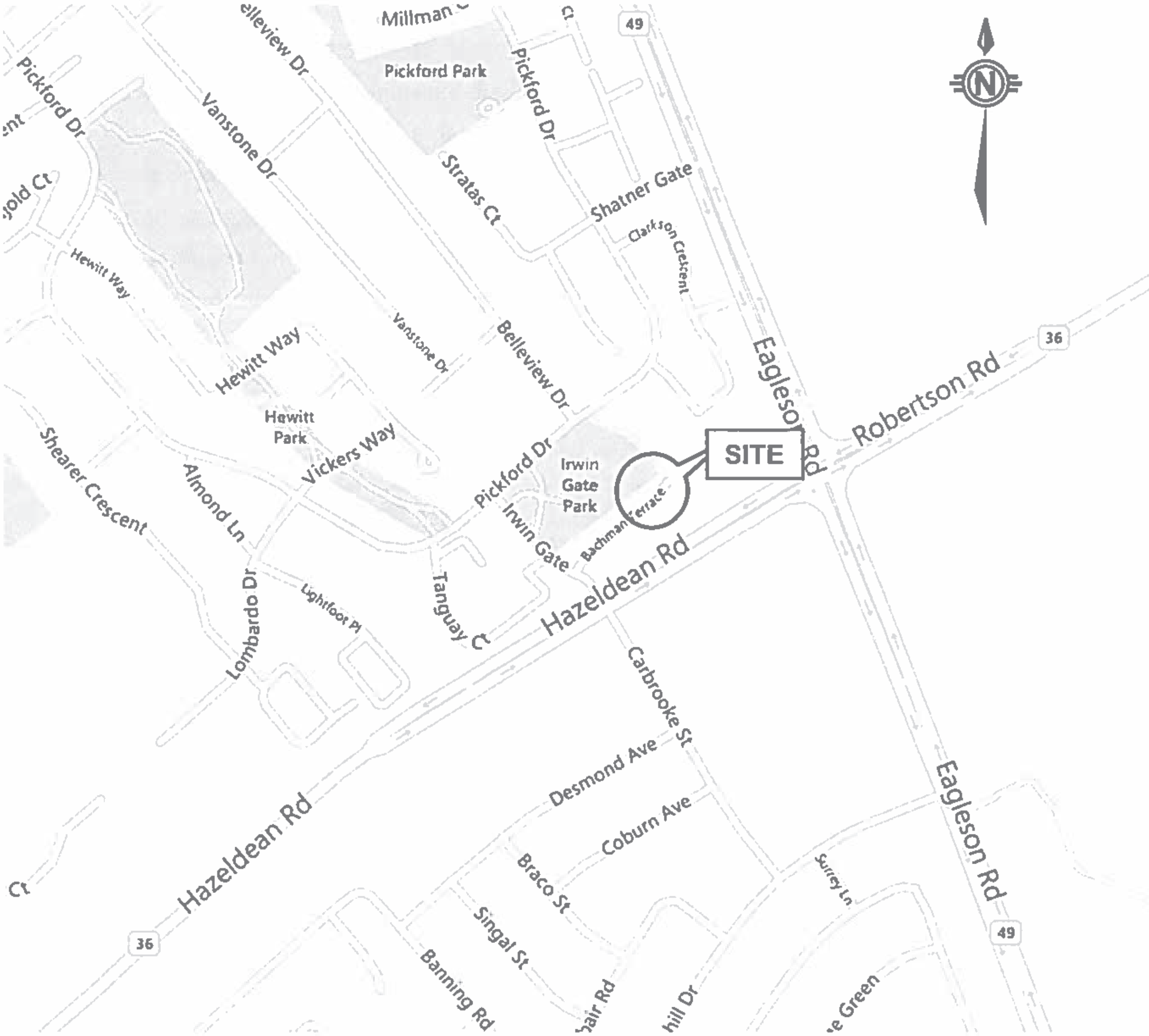
We trust this report provides sufficient information for your present purposes. If you have any questions concerning this report, please do not hesitate to contact our office.

Johnathan A. Cholewa, Ph.D., P.Eng.

LICENSED PROFESSIONAL ENGINEER
J. A. CHOLEWA
100119055
25 June 2014
PROVINCE OF ONTARIO

A.C. Houle, M.Eng., P.Eng.
Principal

KEY PLAN	FIGURE 1

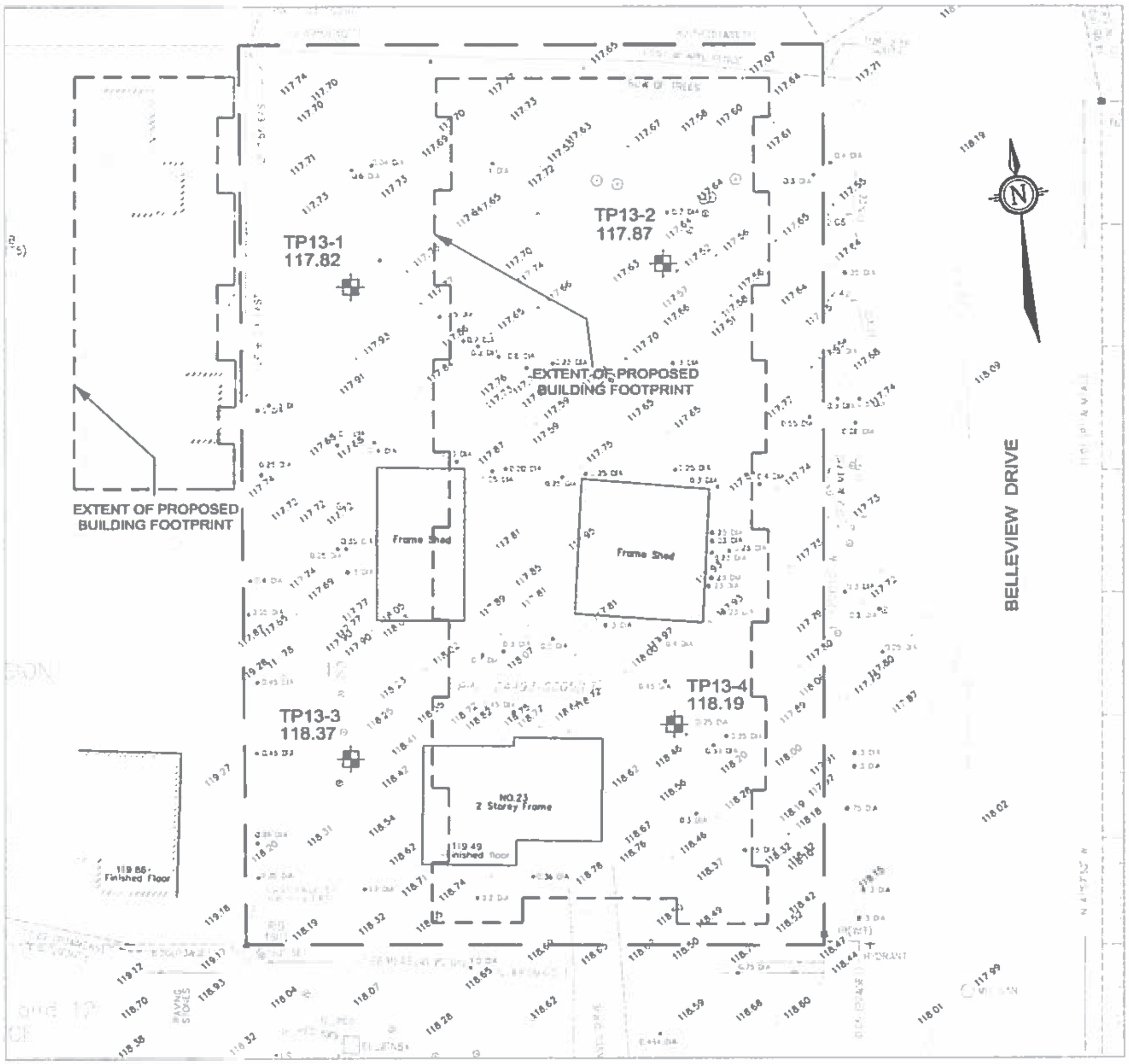


N.T.S

Houle Chevrier Engineering

Date: June 2014

Project: 13-042



LEGEND

TP13-1 APPROXIMATE TEST PIT LOCATION IN PLAN, CURRENT INVESTIGATION BY HOULE CHEVRIER ENGINEERING LTD.

118.37 GEODETIC ELEVATION AT GROUND SURFACE OF TEST PIT, MEASURED IN METRES.

REFERENCE: PLAN PREPARED USING SITE PLAN PROVIDED BY J. D. BARNES SURVEYING LTD.

Client NINAVA LTD.		Location 23 BACHMAN TERRACE OTTAWA, ON	Revision 0
Drawn by M.J.L.	Approved by J.A.C	Project No. 13-042	Scale 1:300
Houle Chevrier Engineering	Title TEST PIT LOCATION PLAN		
	Date June 2014		FIGURE 2

APPENDIX A

LIST OF ABBREVIATIONS AND TERMINOLOGY
RECORD OF TEST PIT SHEETS

LIST OF ABBREVIATIONS AND TERMINOLOGY

SAMPLE TYPES

AS	auger sample
CS	chunk sample
DO	drive open
MS	manual sample
RC	rock core
ST	slotted tube
TO	thin-walled open Shelby tube
TP	thin-walled piston Shelby tube
WS	wash sample

PENETRATION RESISTANCE

Standard Penetration Resistance, N

The number of blows by a 63.5 kg hammer dropped 760 millimetres required to drive a 50 mm drive open sampler for a distance of 300 mm. For split spoon samples where less than 300 mm of penetration was achieved, the number of blows is reported over the sampler penetration in mm.

Dynamic Penetration Resistance

The number of blows by a 63.5 kg hammer dropped 760 mm to drive a 50 mm diameter, 60° cone attached to 'A' size drill rods for a distance of 300 mm.

WH

Sampler advanced by static weight of hammer and drill rods.

WR

Sampler advanced by static weight of drill rods.

PH

Sampler advanced by hydraulic pressure from drill rig.

PM

Sampler advanced by manual pressure.

SOIL TESTS

C	consolidation test
H	hydrometer analysis
M	sieve analysis
MH	sieve and hydrometer analysis
U	unconfined compression test
Q	undrained triaxial test
V	field vane, undisturbed and remoulded shear strength

SOIL DESCRIPTIONS

Relative Density	'N' Value
Very Loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very Dense	over 50

Consistency	Undrained Shear Strength (kPa)
Very soft	0 to 12
Soft	12 to 25
Firm	25 to 50
Stiff	50 to 100
Very Stiff	over 100

LIST OF COMMON SYMBOLS

c_u	undrained shear strength
e	void ratio
C_c	compression index
c_v	coefficient of consolidation
k	coefficient of permeability
I_p	plasticity index
n	porosity
u	pore pressure
w	moisture content
w_L	liquid limit
w_P	plastic limit
ϕ^1	effective angle of friction
γ	unit weight of soil
γ^1	unit weight of submerged soil
σ	normal stress

RECORD OF TEST PIT 13-1

PROJECT: 13-042
LOCATION: See Test Pit Location Plan, Figure 2
DATE OF EXCAVATION: March 7, 2013

SHEET 1 OF 1
DATUM: Geodetic
TYPE OF EXCAVATOR: Hydraulic Shovel

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		117.82					
	Dark brown sandy silt, some gravel with organic material and cobbles (TOPSOIL FILL)			1				Backfilled with excavated material
	End of Test Pit Excavator Refusal		117.45 0.37					Test pit dry upon completion on March 7, 2013
1								
2								
3								

DEPTH SCALE
1 to 15

Houle Chevrier Engineering Ltd.

LOGGED: M.L.
CHECKED:

TESTPIT RECORD 2012 13-042 GINT TP.GPJ HCE DATA TEMPLATE.GDT 21/3/13

RECORD OF TEST PIT 13-2

PROJECT: 13-042
LOCATION: See Test Pit Location Plan, Figure 2
DATE OF EXCAVATION: March 7, 2013

SHEET 1 OF 1
DATUM: Geodetic
TYPE OF EXCAVATOR: Hydraulic Shovel

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural. V - + Remoulded. V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp ⊢ W ⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		117.87					
	Dark brown sandy silt with organic material (TOPSOIL)			1				
			117.55 0.32					
	Brown to grey brown silty sand, some gravel with probable cobbles and boulders (GLACIAL TILL)			2				Backfilled with excavated material
			116.97 0.90					
1	Grey, weathered/fractured bedrock			3				
			116.67 1.20					
	End of Test Pit Excavator Refusal							Test pit dry upon completion on March 7, 2013
2								
3								

DEPTH SCALE
1 to 15

Houle Chevrier Engineering Ltd.

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RECORD OF TEST PIT 13-3

PROJECT: 13-042
LOCATION: See Test Pit Location Plan, Figure 2
DATE OF EXCAVATION: March 7, 2013

SHEET 1 OF 1
DATUM: Geodetic
TYPE OF EXCAVATOR: Hydraulic Shovel

DEPTH SCALE METRES	SOIL PROFILE: DESCRIPTION	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural, V - + Remoulded, V - ⊕ (20 40 60 80)	WATER CONTENT (PERCENT) Wp ⊢—○ W—⊣ Wl (20 40 60 80)	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface	118.37					
	Dark brown sandy silt with organic material (TOPSOIL FILL)	118.17 0.20	1				Backfilled with excavated material
	Brown silty sand, trace gravel (FILL MATERIAL)	117.67 0.70	2				
	Reddish grey to grey brown, gravel, some sand and silt (FILL MATERIAL)	117.52 0.85					
	End of Test Pit Excavator Refusal						Test pit dry upon completion on March 7, 2013
1							
2							
3							

DEPTH SCALE
1 to 15

Houle Chevrier Engineering Ltd.

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TESTPIT RECORD 2012 13-042 GINT TP.GPJ HCE DATA TEMPLATE.GDT 21/3/13

PROJECT: 13-042

RECORD OF TEST PIT 13-4

SHEET 1 OF 1

LOCATION: See Test Pit Location Plan, Figure 2

DATUM: Geodetic

DATE OF EXCAVATION: March 7, 2013

TYPE OF EXCAVATOR: Hydraulic Shovel

DEPTH SCALE METRES	SOIL PROFILE DESCRIPTION	STRATA PLOT	ELEV. DEPTH (m)	SAMPLE NUMBER	SHEAR STRENGTH, Cu (kPa) Natural, V - + Remoulded, V - ⊕ 20 40 60 80	WATER CONTENT (PERCENT) Wp ⊢—○W—⊣ Wl 20 40 60 80	ADDITIONAL LAB. TESTING	WATER LEVEL IN OPEN TEST PIT OR STANDPIPE INSTALLATION
0	Ground Surface		118.19					
	Dark brown sandy silt with organic material (TOPSOIL FILL)		118.09 / 0.10	1				
	Grey silty sand with pieces of plastic (FILL MATERIAL)		117.97 / 0.22	2				
	Dark brown sandy silt with organic material (FORMER TOPSOIL)		117.86 / 0.33	3				
	Brown to grey brown silty sand, some gravel with probable cobbles and boulders (GLACIAL TILL)		117.57 / 0.62	4				Backfilled with excavated material
	Grey, weathered/fractured bedrock		117.26 / 0.93	5				
1	End of Test Pit Excavator Refusal							Test pit dry upon completion on March 7, 2013
2								
3								

DEPTH SCALE 1 to 15

Houle Chevrier Engineering Ltd.

LOGGED: M.L.

CHECKED: [signature]