

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix A Water Supply Servicing  
March 29, 2022

## **Appendix A**    **WATER SUPPLY SERVICING**

### **A.1**    **DOMESTIC WATER DEMAND ESTIMATE**

**Wellings of Stittsville Phase 2 - 20 Cedarow Court - Domestic Water Demand Estimates**

- Based on Wellings of Stittsville Site Phase 2 (160401511)

Building ID	Area (m <sup>2</sup> )	Population	Daily Rate of Demand <sup>1</sup>	Avg Day Demand		Max Day Demand <sup>2,3</sup>		Peak Hour Demand <sup>2,3</sup>	
				(L/min)	(L/s)	(L/min)	(L/s)	(L/min)	(L/s)
<b>Phase 2 and Phase 3</b>									
Residential	-	441	350	107.2	1.79	268.0	4.47	589.5	9.83
Commercial and communal Amenity Areas	4726	-	28,000	9.2	0.15	13.8	0.23	24.8	0.41
<b>Phase 4</b>									
Residential	-	312	350	75.9	1.26	189.7	3.16	417.4	6.96
<b>Total Site :</b>				<b>192.3</b>	<b>3.20</b>	<b>471.5</b>	<b>7.86</b>	<b>1031.7</b>	<b>17.19</b>

- 28,000 L/gross ha/day is used to calculate water demand for retail, restaurants and office space.
- The City of Ottawa water demand criteria used to estimate peak demand rates for commercial space are as follows:  
 maximum day demand rate = 1.5 x average day demand rate  
 maximum hour demand rate = 1.8 x maximum day demand rate
- The City of Ottawa water demand criteria used to estimate peak demand rates for residential areas are as follows:  
 maximum day demand rate = 2.5 x average day demand rate  
 maximum hour demand rate = 2.2 x maximum day demand rate

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**A.2 FIRE FLOW REQUIREMENTS PER FUS**



FUS Fire Flow Calculation Sheet

Stantec Project #: 160401317  
 Project Name: 20 Cedarow Court  
 Date: 9/1/2021

Fire Flow Calculation #: 1  
 Description: Phase 2 and 3

Notes: 6 storey building with 2hr horizontal firewalls between each floor

Step	Task	Notes	Value Used	Req'd Fire Flow (L/min)					
1	Determine Type of Construction	Non-Combustible Construction	0.8	-					
2	Determine Ground Floor Area of One Unit	-	4456	-					
	Determine Number of Adjoining Units	-	1	-					
3	Determine Height in Storeys	Does not include floors >50% below grade or open attic space	1	-					
4	Determine Required Fire Flow	(F = 220 x C x A <sup>1/2</sup> ). Round to nearest 1000 L/min	-	12000					
5	Determine Occupancy Charge	Limited Combustible	-15%	10200					
6	Determine Sprinkler Reduction	Conforms to NFPA 13	-30%	-4080					
		Standard Water Supply	-10%						
		Not Fully Supervised or N/A	0%						
		% Coverage of Sprinkler System	100%						
7	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	-	-
		North	10.1 to 20	30	6	> 120	Wood Frame or Non-Combustible	15%	3978
		East	20.1 to 30	82	5	> 120	Wood Frame or Non-Combustible	10%	
		South	> 45	123	1	> 120	Wood Frame or Non-Combustible	0%	
		West	10.1 to 20	82	1	61-90	Wood Frame or Non-Combustible	14%	
8	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min			10000				
		Total Required Fire Flow in L/s			166.7				
		Required Duration of Fire Flow (hrs)			2.00				
		Required Volume of Fire Flow (m <sup>3</sup> )			1200				



FUS Fire Flow Calculation Sheet

Stantec Project #: 160401317  
 Project Name: 20 Cedarow Court  
 Date: 9/1/2021

Fire Flow Calculation #: 1  
 Description: Phase 4

Notes: 6 storey building with 2hr horizontal firewalls between each floor

Step	Task	Notes	Value Used	Req'd Fire Flow (L/min)					
1	Determine Type of Construction	Non-Combustible Construction	0.8	-					
2	Determine Ground Floor Area of One Unit	-	3192	-					
	Determine Number of Adjoining Units	-	1	-					
3	Determine Height in Storeys	Does not include floors >50% below grade or open attic space	1	-					
4	Determine Required Fire Flow	( $F = 220 \times C \times A^{1/2}$ ). Round to nearest 1000 L/min	-	10000					
5	Determine Occupancy Charge	Limited Combustible	-15%	8500					
6	Determine Sprinkler Reduction	Conforms to NFPA 13	-30%	-3400					
		Standard Water Supply	-10%						
		Not Fully Supervised or N/A	0%						
		% Coverage of Sprinkler System	100%						
7	Determine Increase for Exposures (Max. 75%)	Direction	Exposure Distance (m)	Exposed Length (m)	Exposed Height (Stories)	Length-Height Factor (m x stories)	Construction of Adjacent Wall	-	-
		North	> 45	122	1	> 120	Wood Frame or Non-Combustible	0%	2125
		East	30.1 to 45	54	5	> 120	Wood Frame or Non-Combustible	5%	
		South	10.1 to 20	122	6	> 120	Wood Frame or Non-Combustible	15%	
		West	30.1 to 45	28	1	0-30	Wood Frame or Non-Combustible	5%	
8	Determine Final Required Fire Flow	Total Required Fire Flow in L/min, Rounded to Nearest 1000L/min							7000
		Total Required Fire Flow in L/s							116.7
		Required Duration of Fire Flow (hrs)							2.00
		Required Volume of Fire Flow (m <sup>3</sup> )							840

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**A.3 BOUNDARY CONDITIONS**

## Boundary Conditions - 20 Cedarow Court

Date Provided

October-19

Scenario	Demand	
	L/min	L/s
Average Daily Demand	156	2.60
Maximum Daily Demand	388	6.46
Peak Hour	850	14.17
Fire Flow Demand #1	16,020	267

# of connections

2

### Location:



## Results:

### Connection 1 - Cedarow Crescent

Demand Scenario	Head (m)	Pressure <sup>1</sup> (psi)
Maximum HGL	161.1	80.3
Peak Hour	157.7	75.5
Max Day plus Fire 1	150.2	64.8

<sup>1</sup> Ground Elevation = 104.6m

### Connection 2 - Wellings Pvt

Demand Scenario	Head (m)	Pressure <sup>1</sup> (psi)
Maximum HGL	161.1	80.3
Peak Hour	157.7	75.4
Max Day plus Fire 1	149.6	63.9

<sup>1</sup> Ground Elevation = 104.7m

## Notes:

1. Pressure reducing valve is required since the maximum pressure exceeds 80 psi.
2. Looping of the watermain is required to decrease vulnerability of the water system in case of breaks.
3. Confirm the ownership of the watermain on Wellings Private.

## Disclaimer

*The boundary condition information is based on current operation of the city water distribution system. The computer model simulation is based on the best information available at the time. The operation of the water distribution system can change on a regular basis, resulting in a variation in boundary conditions. The physical properties of watermains deteriorate over time, as such must be assumed in the absence of actual field test data. The variation in physical watermain properties can therefore alter the results of the computer model simulation. Fire Flow analysis is a reflection of available flow in the watermain; there may be additional restrictions that occur between the watermain and the hydrant that the model cannot take into account.*



**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
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Appendix B Wastewater Servicing  
March 29, 2022

## **Appendix B WASTEWATER SERVICING**

### **B.1 SANITARY SEWER DESIGN**



SUBDIVISION:  
**Nautical Lands Group - Wellings of Stittsville Senior's Living and Extencicare L.T.C.**  
 DATE: 8/31/2021  
 REVISION: 2  
 DESIGNED BY: TR  
 CHECKED BY: DT

**SANITARY SEWER DESIGN SHEET**  
 (City of Ottawa)

FILE NUMBER: 1604-01511

**DESIGN PARAMETERS**

MAX PEAK FACTOR (RES.)=	4.0	AVG. DAILY FLOW / PERSON	280 L/p/day	MINIMUM VELOCITY	0.60 m/s
MIN PEAK FACTOR (RES.)=	2.0	COMMERCIAL	28,000 L/ha/day	MAXIMUM VELOCITY	3.00 m/s
PEAKING FACTOR (INDUSTRIAL):	2.4	INDUSTRIAL (HEAVY)	55,000 L/ha/day	MANNINGS n	0.013
PEAKING FACTOR (COMM., INST.):	1.5	INDUSTRIAL (LIGHT)	35,000 L/ha/day	BEDDING CLASS	B
STUDIO APARTMENT	1.4	INSTITUTIONAL	28,000 L/ha/day	MINIMUM COVER	2.50 m
1 BEDROOM	1.4	INFILTRATION	0.33 L/s/ha		
2 BEDROOM	2.1				

LOCATION			RESIDENTIAL AREA AND POPULATION								COMMERCIAL		INDUSTRIAL (L)		INDUSTRIAL (H)		INSTITUTIONAL		GREEN / UNUSED		C+H	INFILTRATION			TOTAL	PIPE										
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (ha)	Studio	Single 1 Bedroom Units	2 Bedroom	POP.	CUMULATIVE AREA (ha)	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)	
<b>Wellings of Stittsville Ph2</b>																																				
ENTIRE SITE	STUB	MAIN	1.82	0	376	108	753	1.82	753	3.88	9.5	0.47	0.47	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.2	2.29	2.29	0.8	10.4	23.1	300 675	PVC	SDR 35	0.40	60.7	17.20%	0.86	0.53	

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Appendix B Wastewater Servicing  
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**B.2 SANITARY EXCERPTS FROM THE KWMSS**

## 4.0 SANITARY SEWER SERVICING

### 4.1 Introduction

This section outlines the evaluation criteria for wastewater servicing options, describes the alternative wastewater servicing alignments, summarizes the evaluation process, and compares the recommended alternatives to select the preferred option.

### 4.2 Evaluation Criteria and Weightings

The evaluation of alternatives is based, in part, on criteria previously developed for the Regional Master Plan for Water, Wastewater and Transportation, which can be found in Volume 2 of the "Planning and Environmental Assessment Summary Report" prepared by the former Region of Ottawa-Carleton.

The criteria are divided into four categories. The first three categories consider environmental, social, and economic impacts of the project on the Study area. The fourth category (Constructability/Functionality) considers project-specific criteria assessing the technical aspects and impacts of the project on the Study area. A list of each criteria and its respective category, as well as an explanation of their indicators, is provided in **Table 4.1-1**.

TABLE 4.1-1 Evaluation Criteria		
Category	Criteria	Indicator
<b>Constructability/Functionality</b>		
CO1.1	Geotechnical Issues and Construction Risks	Potential for encountering poor soils and/or elevated groundwater conditions.
CO1.2	Infrastructure Requirements	Extent of works required.
CO1.3	Operational Impacts	Amount of maintenance intensive infrastructure required.
CO1.4	Construction Scheduling	Impact of construction on development timing/phasing.
CO1.5	Property Acquisition	Ease of property acquisition. Depends on status of required and adjacent lands (i.e. vacant, leased or owner occupied).
CO1.6	System Reliability	Proximity of a storm sewer, SWM or other surface water for emergency overflow.
CO1.7	System Flexibility	Ease of accommodating potential changes in servicing plans.
<b>Economy</b>		
E1	Potential to Use Combined Service Corridor	Length and area of combined service corridor.
E2	Efficiency of use of existing infrastructure	Use of existing capacity.
E3	Energy consumption	Pumping requirements.
E5	Impact on Agriculture	Agricultural area likely to be affected by infrastructure.

E9	Construction Cost	Estimated construction cost.
<b>Caring and Healthy Community</b>		
C3	Displacement of Residents, Community/Recreation Features and Institutions	Affects on residential areas, institutions or businesses.
C4	Disruption to Existing Community	Extent of works affecting existing residences and businesses.
C9	Consistency with Planned Land Use and Infrastructure	Compatibility with City land use, design guidelines and infrastructure servicing corridor planning (Kanata West Transportation Master Plan Report and Storm Sewer and Watermain Needs).
<b>Natural Environment</b>		
N1	Impact on Significant Natural Features	Loss of natural areas due to installation of works.
N3	Impact on Aquatic Systems	Potential impact on fish habitat due to installation of works.
N4	Impact on Quality and Quantity of Surface Water and Groundwater	Potential impact on water quality in the Carp River resulting from rare emergency overflows to the SWM pond due to pumping station failure.
N5	Impact on Global Warming	Difference in carbon dioxide emissions resulting from occasional use of diesel generator.
N6	Effects on Urban Green Space, Open Space and Vegetation	Disruption to green space and trees.

#### 4.2.1 Description of Evaluation Categories

Presented below is a description of the categories used to assess each of the three servicing alternatives. The four categories were selected to ensure that the various servicing alternatives were evaluated in a consistent and comprehensive manner. Further details on the criteria and weightings for each category are provided in Appendix 2.1.

##### **Constructability/Functionality (C/F) – 36%**

Wastewater infrastructure is required to facilitate the development of Kanata West. The infrastructure needs to provide a flexible servicing solution to accommodate the orderly development of the entire area in a series of phases. It is important that the construction of the wastewater servicing be coordinated with other major infrastructure projects such as storm sewers, watermain and transportation, to ensure all services are available when required. Various alignment alternatives, construction techniques and phasing options will be assessed.

##### **Economy (E) – 25 %**

The Kanata West area is recognized within the City of Ottawa Official Plan as a future growth area comprised of a mixture of residential, business, and commercial lands. The accelerated rate of development and design concerns within the Study area requires a cost-effective solution to providing municipal wastewater services.

**Caring and Healthy Communities (CHC) – 25 %**

Impact to the surrounding community is an important factor when determining the preferred servicing alternative. The selected alignment and construction techniques are evaluated to minimize disruption to surrounding communities. It is anticipated that impacts will be limited to the time of construction for the off-site servicing.

**Natural Environment (NE) – 14%**

The majority of the required wastewater infrastructure is aligned within existing or proposed public roads to limit the impacts to the natural environment. Servicing alignments selected outside of roadways were chosen to minimize impacts where possible. Construction of the wastewater services will be performed in conjunction with other servicing projects required as part of the overall development. Further information on the environmental impacts of the proposed road allowances are documented in the Kanata West Transportation Master Plan Report.

In the rare event that the pump station overflows, impacts to surface water quality are anticipated to be minimal. All discharges from the overflow will be directed to the stormwater management pond where they will be collected. Increases in CO<sub>2</sub> emissions from the emergency diesel generators during power failures or maintenance procedures will be negligible.

**4.2.2 Outlet Alternatives**

4.2.2.1 Description of Outlet Alternatives

To provide an adequate outlet for the KWCP wastewater system three servicing options were evaluated. Each of these options will ultimately discharge to the Tri Township Collector Sewer. The first servicing option utilizes a gravity sewer (tunnel), the remaining two options make use of a pumping station located at the intersection of Maple Grove Road and Silver Seven Road, with alternate forcemain alignments. **Figures 4.1-1, 4.1-2 and 4.1-3** illustrate the alternative outlet alignments, which are further described below:

- Alternative I (Gravity Outlet) – A gravity sewer (tunnel) along the Highway 417 corridor to the Tri Township Collector. The tunnel would be constructed within the existing road allowance, adjacent to the travel lanes. The alignment crosses Highway 417 east of Eagleson Road and parallels the Glen Cairn Collector. Refer to **Figure 4.1-1**.
- Alternative II (Forcemain Alignment 1) - A forcemain along the Highway 417 corridor from a proposed pumping station on Maple Grove Road, extending to the Glen Cairn Collector Sewer east of Eagleson Road. Refer to **Figure 4.1-2**.
- Alternative III (Forcemain Alignment 2) - A forcemain along Katimavik Road and Palladium Drive from a proposed Pumping Station on Maple Grove Road to the Glen Cairn Collector Sewer east of Eagleson Road. Refer to **Figure 4.1-3**.

#### 4.2.2.2 Evaluation of Outlet Alternatives

Evaluation of the criteria was completed using the “standard pair-wise comparison” methodology. The weightings assigned to each of the criteria were selected based on the weightings applied for past similar projects, knowledge of environmental constraints, community concerns and professional judgment. The scores for each category and criterion were summed to determine the overall category weighting. Evaluation results are summarized in **Table 4.1-2**. An explanation of the category rankings and weightings are provided below.

##### **Constructability/Functionality (C/F) (36%)**

A review of the three proposed servicing options indicates that the forcemain alternatives present fewer issues with respect to the geotechnical constraints when compared to the gravity sewer alternative. The forcemain alternatives would require a relatively shallow excavation, reducing the conflict with the shallow bedrock formations that exist along each forcemain alignment. The shallow depth of the forcemains would also minimize the technical difficulties arising from earth to rock transitions along the trench. The effort required to install either of the forcemain alternatives would be much less than the gravity outlet alternative because the need to tunnel would be eliminated.

When comparing the two forcemain alternatives, an obvious benefit of Alternative II is its location along Katimavik Road as compared to the location of Alternative III, along Highway 417. Katimavik Road has a lower classification than Highway 417, reducing traffic management issues during construction and routine maintenance operations. The central location of the Alternative II forcemain alignment in relation to the area to be serviced also improves the flexibility for developing internal servicing options. The various alignments available for Alternative II, west of Terry Fox Drive (see **Figure 4.1-3**), are all located within existing road allowances and are considered equal when evaluated with the prescribed criteria. The Alternative II alignment along Silver Seven Road also allows the opportunity for the services to be installed in an easement located immediately adjacent to the east side of the right-of-way. Construction in this easement would eliminate the need to reconstruct this portion of the road. The use of easements for construction of the necessary services was not factored into the evaluation criteria and therefore the ranking was not affected.

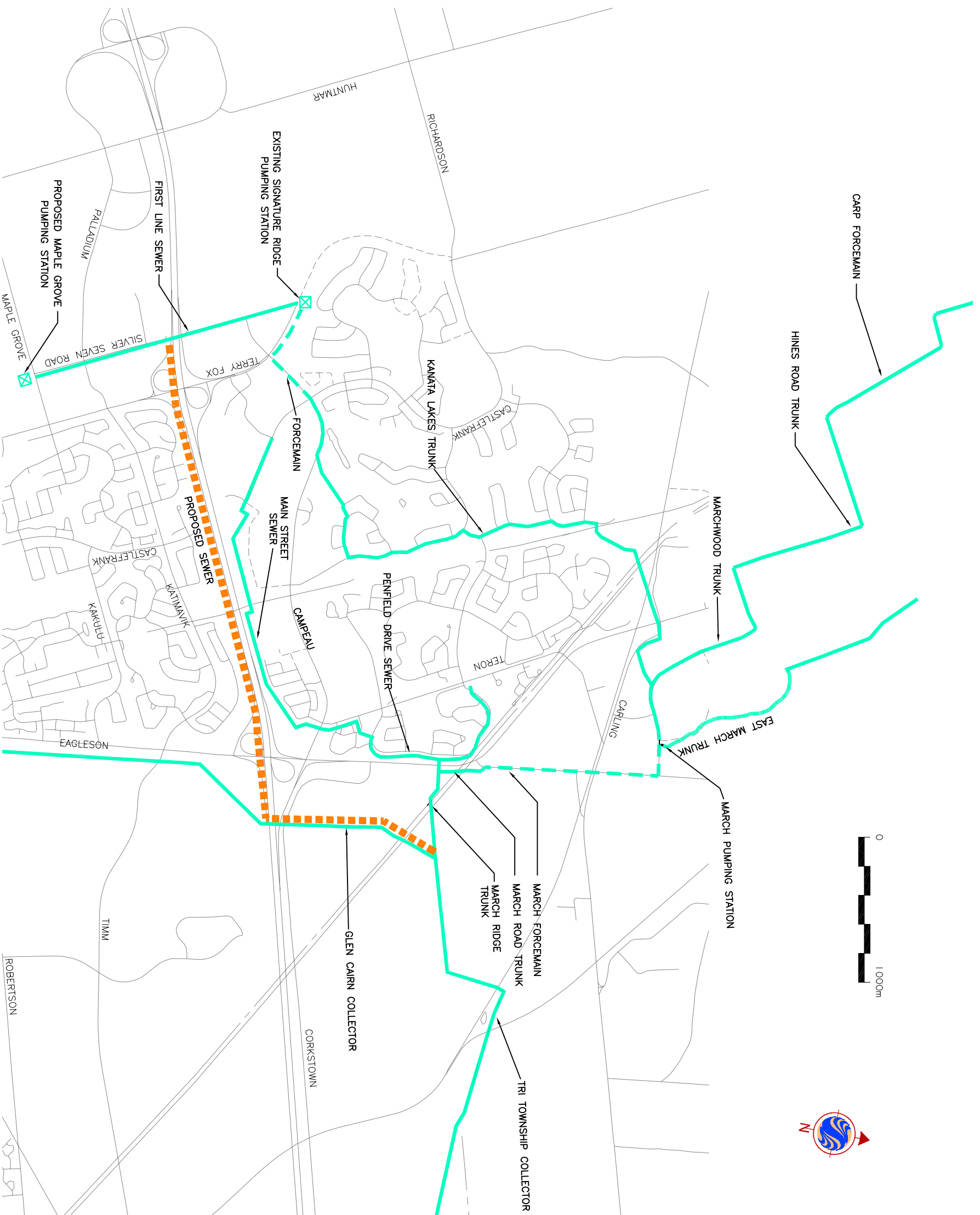
##### **Economy (E) (25%)**

The costs of both forcemain alternatives are similar and much less expensive than the gravity sewer alternative. The increased costs of the gravity sewer are attributed to the need to tunnel through the existing bedrock. The forcemain alternatives allow for a relatively shallow excavation over the entire length of the alignment. The level of effort required to construct the gravity sewer would also be significantly greater than the effort required to install either of the forcemain alternatives.

##### **Caring and Healthy Community (CHC) (25%)**

Both the gravity outlet and the Alternative I forcemain would have minimal impact on the community given that the majority of the work would occur within the Highway 417 road allowance. The Alignment II forcemain alignment along Katimavik Road would require open cut excavation and would have a temporary impact on the residents during construction.

# GRAVITY SANITARY OUTLET LOCATION



- Legend:**
- Gravity Outlet (Tunnel)
  - Existing Trunk Sewer

FIG. 4.1-1

MAY 2006

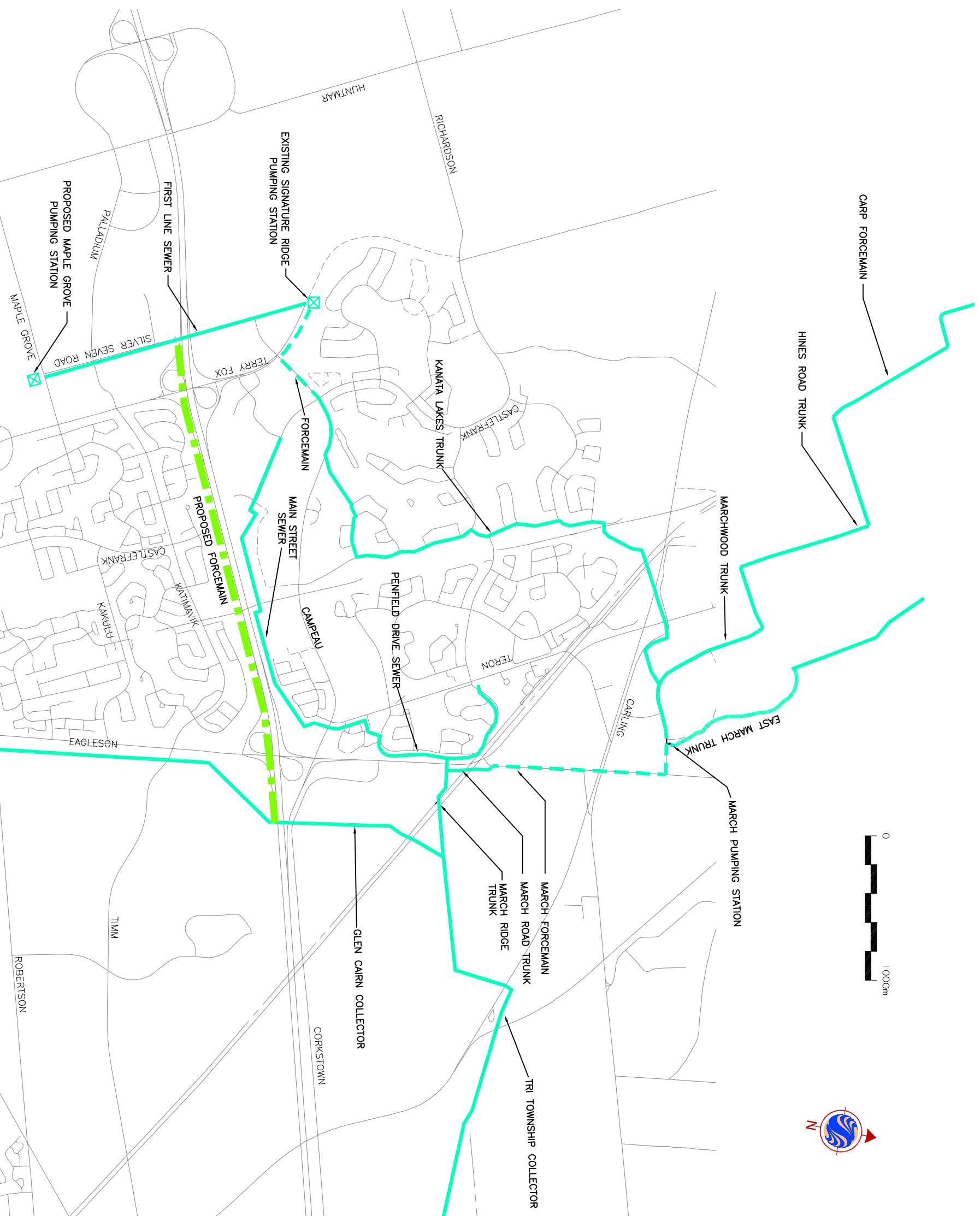
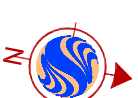


Startec



CCCL/BI  
Cummins Construction Limited





# SANITARY FORCEMAIN OUTLET ALIGNMENT 1 HWY 417

- Legend:**
-  Forcemain
  -  Existing Trunk Sewer

FIG. 4.1-2

MAY 2006



Stantec



CCCL/IBI

Consulting Civil Engineers Limited

**SANITARY  
FORCEMAIN OUTLET  
ALIGNMENT 2  
KATIMAVIK RD.  
(PREFERRED OPTION)**



- Legend:**
- Forcemain
  - - - Optional F.M. locations to accommodate internal servicing
  - Existing Trunk Sewer

**FIG. 4.1-3**

MAY 2006



Stantec



CCCL/BI  
Gumina Colectura Limited

The construction of both forcemain alternatives is compatible with existing City design standards and construction practices. However, only Alignment II can easily be integrated into other servicing or roadway improvements. The time required for the construction of the gravity outlet would be significantly longer than that of the forcemain alternatives.

#### **Natural Environment (NE) (14%)**

There are no significant differences on the impacts to the natural environment between the gravity outlet and forcemain Alternative II. The gravity outlet will be tunneled below ground for the majority of the alignment resulting in minimal impact to surface conditions. Forcemain Alternative II is located within the Katimavik Road allowance, which is already developed and has minimal environmental impact. Forcemain Alternative I has a greater impact on the natural areas located along the Highway 417 corridor than the gravity sewer.

#### **4.2.2.3 Selection of Preferred Outlet Alternative**

Based on the above evaluation Alternative II, the Katimavik Road alignment, is selected as the preferred outlet alternative. This alignment offers the greatest amount of flexibility for internal servicing design, uses a lower road classification corridor, which simplifies routine maintenance operations, and provides maximum separation from the sensitive natural areas located in the 417 corridor east of Terry Fox Drive.

While Forcemain Alignment II has a marginal cost increase over Alignment I, the benefits of improved internal servicing and phasing options more than offset this discrepancy.

### **4.2.3 Internal Servicing Alternatives**

#### **4.2.3.1 Description of Internal Servicing Alternatives**

The preliminary servicing report prepared in support of the approved Community Design Plan identified the need for two pumping stations for the wastewater discharge from KWCP. The two stations identified are required to satisfy phasing needs for construction of the overall development area and to produce a cost effective initial phasing scheme. The new sanitary pumping station(s) south of Highway 417 will be required to provide internal wastewater service to that portion of KWCP south of Highway 417.

Three potential servicing alternatives were considered for the configuration and location for the pumping station(s) required to service these lands south of Hwy. 417. Internal servicing alternatives were chosen based on their proximity to the preferred outlet described in Section 4.2.3.3. above, and accessibility to the servicing areas as illustrated in **Figures 4.1-4, 4.1-5, 4.1-6 and 4.1-6A**. A brief description of the alternative pumping station locations are as follows:

- Alternative I - Two pumping stations connected with a combination of gravity sewer and forcemain. One pumping station will be located on Silver Seven Road at Highway 417. The second station will be located on Maple Grove Road at the Carp River. Refer to **Figure 4.1-4**.
- Alternative II - Two pumping stations connected with a gravity sewer. One station will be located on Maple Grove Road near the Carp River and will discharge to the main station located near the Carp River south of Highway 417. A diversion sewer will also be required to intercept the existing Silver Seven Road sanitary sewer. Refer to **Figure 4.1-5**.

TABLE 4.1-2



Kanata West Wastewater - Outlet Alternatives

Criteria	Indicators	Weighting	Rationale for Relative Weights	Alternatives		
				Gravity Sewer Outlet	PS FM Alignment I	PS FM Alignment II
<b>CONSTRUCTABILITY/FUNCTIONALITY</b>			<b>36%</b>	<b>16</b>	<b>20</b>	<b>22</b>
CO1.1	Geotechnical Issues and Construction Risks	Potential for encountering poor soils and/or elevated groundwater conditions.	Alt. I has potential for poor soils conditions due to depth and tunnelling in and out of rock.	2	3	3
CO1.2	Infrastructure Requirements	Extent of works required.	Alt. I with tunnelling is a very large scale operation.	1	3	3
CO1.3	Operational Impacts	Amount of maintenance intensive infrastructure required.	Alt. II and III require more extensive maintenance due to pumping.	3	2	2
CO1.4	Construction Scheduling	Impact of construction on development timing.	Alt. I with tunnelling is an extended construction schedule.	1	3	3
CO1.5	Property Acquisition	Ease of property acquisition. (Depends on status of lands and adjacent lands, i.e. vacant, leased or owner occupied.)		4	4	4
CO1.6	System Reliability	Proximity of a storm sewer, SWM or other surface water for emergency overflow		3	3	3
CO1.7	Servicing Flexibility	Ease of accommodating potential changes in servicing plans.	Alt. I and II have fixed alignments along the north limit of the servicing area. Alt. II has some flexibility to be realigned within the development area, but Alt. III due to its more central location has maximum flexibility within Kanata West.	2	2	4
<b>ECONOMY</b>			<b>25%</b>	<b>9</b>	<b>13</b>	<b>15</b>
E1	Potential to Use Combined Service Corridor	Length and area of combined service corridor.	Alt. I with the requirement for tunnelling does not offer any potential to use combined corridors.	1	2	3
E2	Efficiency of Use of Existing Infrastructure	Use of existing capacity	Alt. I requires reconstruction beyond the closest connection point to the Glen Cairn Collector sewer.	1	3	4
E3	Energy Consumption	Pumping requirements	Alt. II & III require pumping.	3	2	2
E5	Impact on Agriculture	Agriculture area likely to be affected by infrastructure.		3	3	3
E9	Capital Cost	Estimated cost of construction.	Alt. I is very expensive due to the tunnelling requirement.	1	3	3
<b>CARING AND HEALTHY COMMUNITIES</b>			<b>25%</b>	<b>7</b>	<b>9</b>	<b>9</b>
C3	Displacement of Residents, Community/Recreation Features and Institutions.	Affects areas of residence, institutions or businesses.	Length of Construction for Alt. I will result in increased construction traffic, etc.	3	3	3
C4	Disruption to Existing Community	Extent of works affecting existing residences and businesses and visibility of additional infrastructure.		3	3	3
C9	Consistency with Planned Land Use and Infrastructure	Compatibility with City land use, design guidelines and infrastructure servicing corridor planning (Kanata West Roadwork Environmental Study Report and Storm Sewer and Watermain Needs).	Alt. I would provide service for larger area than the existing urban boundary due to size of pipe required to tunnel. Alt. II provides greater flexibility for internal servicing.	1	3	3
<b>NATURAL ENVIRONMENT</b>			<b>14%</b>	<b>16</b>	<b>12</b>	<b>14</b>
N1	Impact on Significant Natural Features	Loss of natural area due to installation of works.	Alt. I mostly tunnel therefore minimal impact. Alt. II in vicinity of ANSI in 417 corridor at Terry Fox.	4	1	3
N3	Impact on Aquatic Systems	Potential impact on fish habitat due to installation of works.		3	3	3
N4	Impact on Quality and Quantity of Surface Water and Groundwater	Potential impact on water quality in the Carp River resulting from rare emergency overflows to the SWM pond due to pump station failure.		3	3	3
N5	Impact on Global Warming	Difference in carbon dioxide emissions resulting from occasional use of diesel generator.	Alt. II and III require pumping in long term. Alt. I does not.	3	2	2
N6	Effects on Urban Greenspace, Open Space and Vegetation (i.e.trees,shrubs,etc.)	Disruption to greenspace and trees.		3	3	3
<b>Total Score</b>			<b>100%</b>	<b>2.17</b>	<b>2.75</b>	<b>3.01</b>
<b>Ranking</b>				<b>3</b>	<b>2</b>	<b>1</b>
<b>Estimated Capital Cost (in \$million)</b>				<b>30</b>	<b>8.8</b>	<b>9</b>

Description of Alternatives

Gravity Sewer Outlet  
 Pump Station - Forcemain Alignment I - HWY 417  
 Pump Station - Forcemain Alignment II - Katimavik Rd.

Evaluation Ranking

1 -2 High or Negative Impact  
 3 Moderate or No Impact  
 4-5 Low or Positive Impact

- Alternatives III and IIIA – Alternative III is a single pumping station with a gravity sewer intercepting the existing Silver Seven Road sanitary sewer. The gravity sewer alignment will be adjacent to the Carp River and connect to the pump station located at Maple Grove Road west of the Carp River. Alternative IIIA is a variation of this alternative utilizing a single pumping station and gravity sewer intercepting the existing Silver Seven Road sewer. The variation from Alternative III is that the gravity sewer will be located within a proposed road right-of-way, or an easement, north of Palladium Drive. Refer to **Figures 4.1-6 and 4.1-6A.**

#### 4.2.3.2 Evaluation of Internal Servicing Alternatives

The alternative internal servicing alignments were evaluated as discussed in Section 4.2. The results of the evaluation are summarized in **Table 4.1-3**. An explanation of the category rankings and weightings are provided below.

##### **Constructability/Functionality (C/F) (36%)**

All proposed alternatives use pumping stations to provide internal wastewater servicing. The use of pumps allows the sewer system to be constructed at a relatively shallow depth. This reduces the potential for contact with poor subsurface conditions during construction. Deep Excavations will be confined to a limited area in the vicinity of the pumping station.

A benefit of Alternatives III and IIIA is that a single pumping station is required to provide the internal servicing. This is advantageous from a constructability and operational point of view when compared to Alternatives I and II which require two pumping stations to service the same area. A further benefit of Alternatives III and IIIA is that either servicing scenario will eliminate the need for the existing Palladium siphon under the Carp River. Removing the siphon will improve the overall reliability to the system.

A benefit of Alternative IIIA over Alternative III is that work in the Carp River corridor is reduced to a single crossing at Palladium Drive. Both Alternatives are close to a stormwater management pond which can be used as an emergency overflow in the rare event of flooding. (see **Figures 4.1-6 and 4.1-6A**)

All alternatives are capable of satisfying a phased development process.

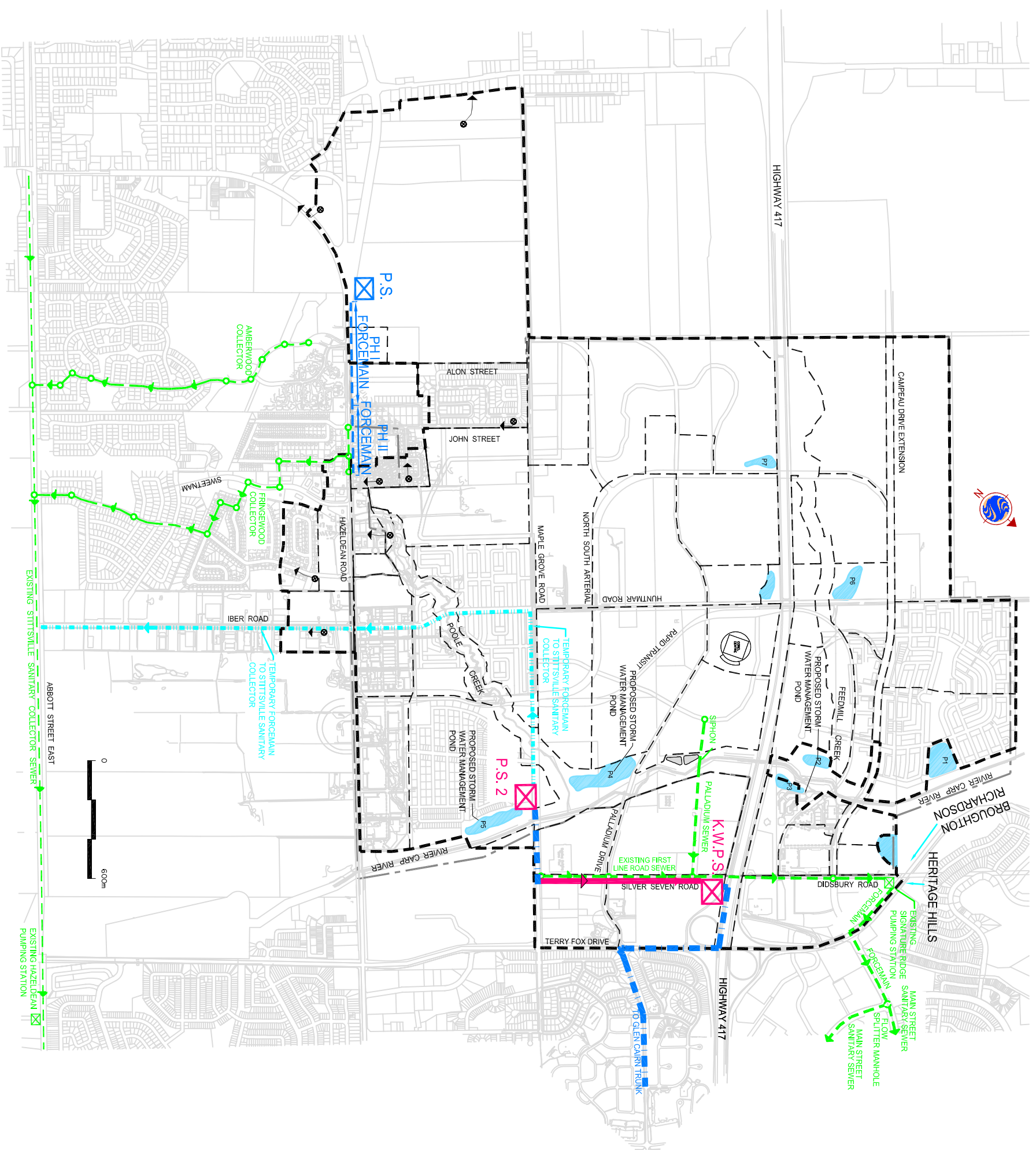
##### **Economy (E) (25%)**

Alternatives I and II use two pumping stations and are significantly more expensive than Alternatives III and IIIA which use a single pump station. The additional pump stations in Alternatives I and II also increase the energy demand over the remaining options. Alternatives III and IIIA are able to service the entire KWCP with a single pump station resulting in equal or fewer economic impacts.

##### **Caring and Healthy Community (CHC) (25%)**

In terms of impact on the community, there are no differences between the alternatives. All options require construction in the vicinity of existing businesses. Impacts are anticipated to be relatively short in duration (less than two years).

# INTERNAL SANITARY SERVICING ALTERNATIVE I



- Ultimate Major Drainage Limit
- Proposed Trunk Sewer
- Forcemain
- Temporary Frocemain
- Existing Trunk Sewers
- Existing Pumping Station and Forcemain (To be Decommissioned)
- ⊠ Pump Station

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FIG. 4.1-4

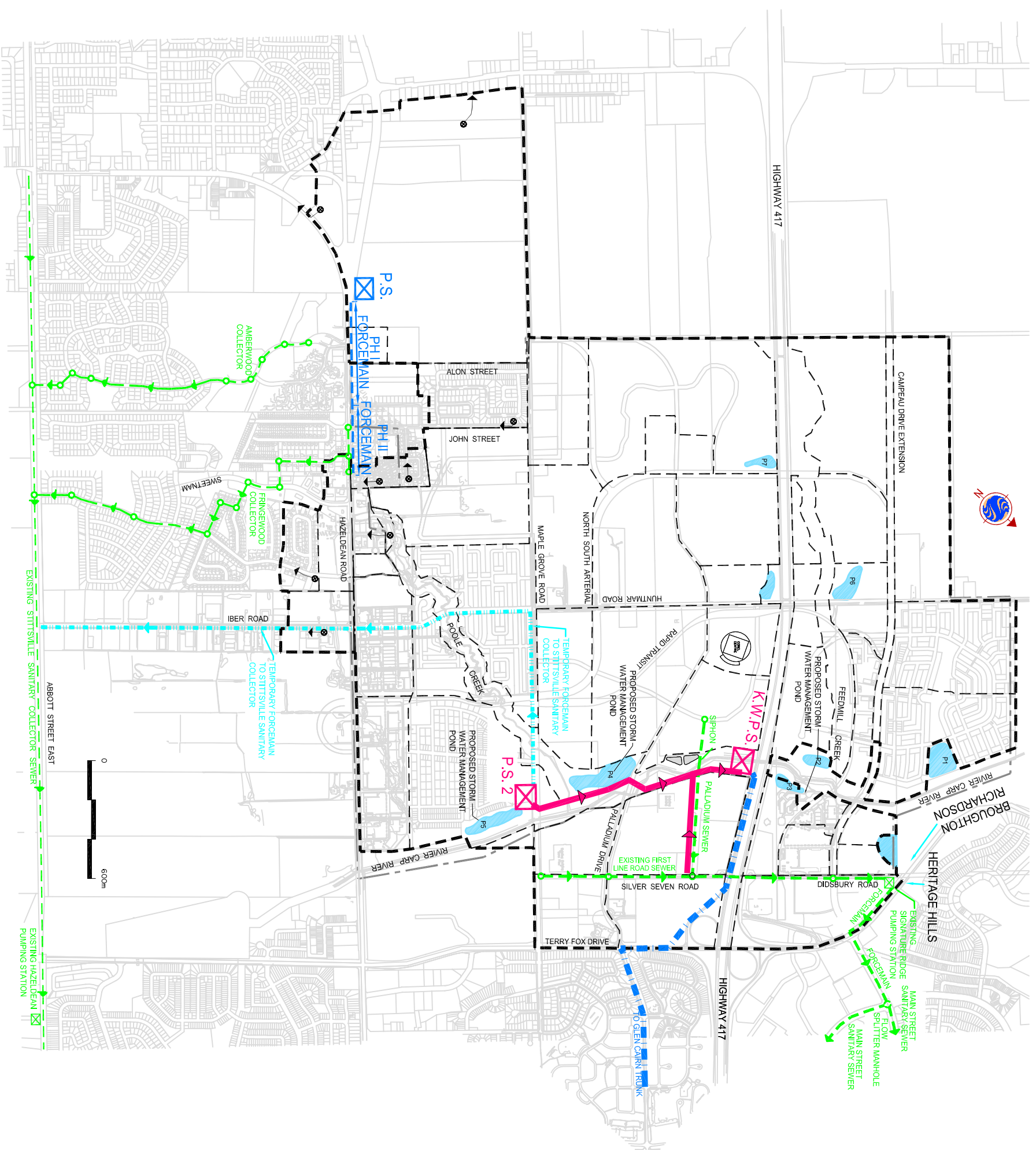


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# INTERNAL SANITARY SERVICING ALTERNATIVE II

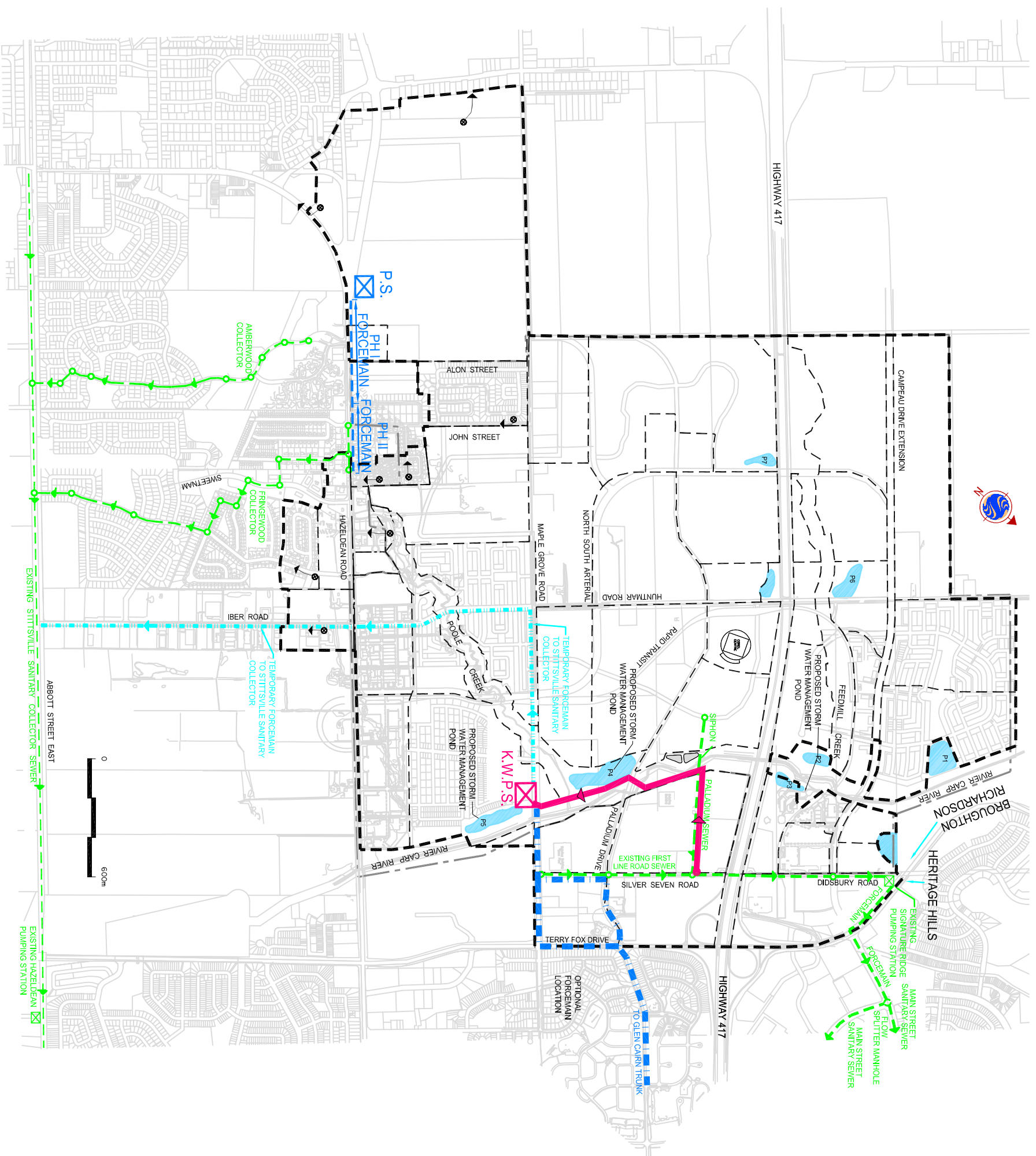


- Ultimate Major Drainage Limit
- Proposed Trunk Sewer
- - - Forcemain
- · - · - Temporary Frocemain
- - - Existing Trunk Sewers
- ⊗ Existing Pumping Station and Forcemain (To be Decommissioned)
- ⊗ Pump Station

FIG. 4.1-5

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# INTERNAL SANITARY SERVICING ALTERNATIVE III



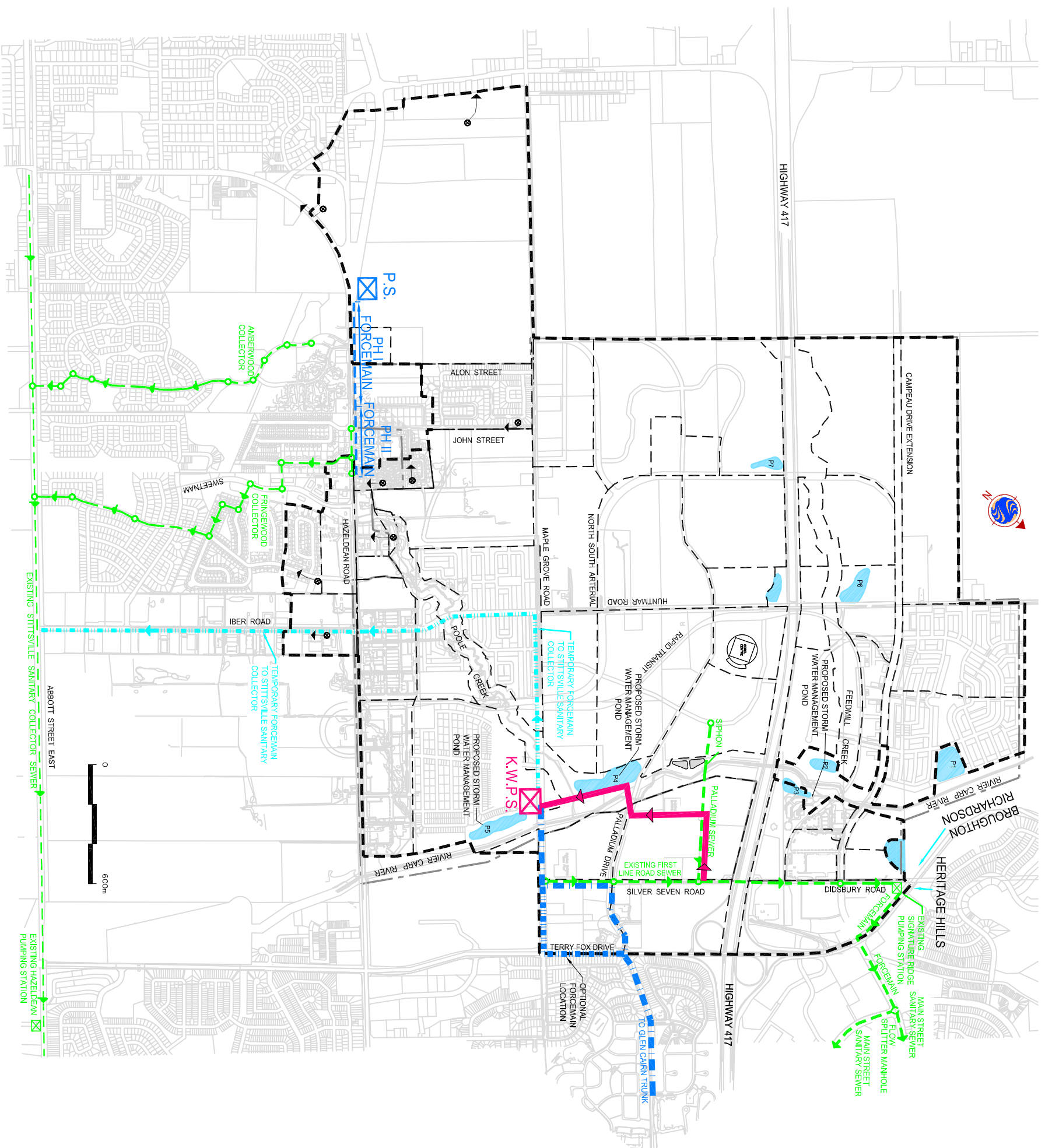
- Ultimate Major Drainage Limit
- Proposed Trunk Sewer
- Forcemain
- ⋯ Temporary Frocemain
- - - Existing Trunk Sewers
- ⊙→ Existing Pumping Station and Forcemain (To be Decommissioned)
- ⊠ Pump Station

FIG. 4.1-6

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# INTERNAL SANITARY SERVICING ALTERNATIVE IIIA (PREFERRED OPTION)



- Ultimate Major Drainage Limit
- Proposed Trunk Sewer
- - - Forcemain
- · · Temporary Frocemain
- - - Existing Trunk Sewers
- · · Existing Pumping Station and Forcemain (To be Decommissioned)
- ⊗ Pump Station

FIG. 4.1-6A

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#### **Natural Environment (NE) (14%)**

All four servicing options have a similar level of impact on the natural environment. Alternatives III and IIIA use a gravity sewer and a single pump station, thereby using less energy to discharge the sanitary flow from the KWCP. Alternatives II & III require the greatest amount of construction within the Carp River corridor.

Alternatives I and II both require two pumping stations. This increases the potential for impacts over the remaining options from the use of the emergency diesel generators and construction and construction.

#### **4.2.3.3 Selection of Preferred Internal Servicing Alternative**

Based on the above evaluation, Alternatives III and IIIA are considered to be the most viable options for the internal wastewater servicing for the KWCP. When comparing the two options, all of the evaluation criteria are similar. However, Alternative III requires the construction of the trunk sanitary sewer within the Carp River corridor. Alternative IIIA utilizes the proposed road allowances for the construction of a portion of the trunk sewer alignment, minimizing the potential for impacts to the Carp River. Based on the evaluation results, Alternative IIIA is selected as the preferred servicing alternative.

#### **4.2.4 Temporary Forcemain Alternatives**

##### **4.2.4.1 Description of Temporary Forcemain Alternatives**

A temporary forcemain will be required to service the initial phases of development within the KWCP. Three potential alignments were selected based on available corridors through the existing community. Each alignment begins at the preferred location of the Kanata West Pump Station, located on Maple Grove Road and west of the Carp River. All three servicing scenarios ultimately discharge to a temporary outlet, the Stittsville Collector Sewer. As illustrated on **Figure 4.1-7** the alternative forcemain alignments are:

- Alternative I – West along Maple Grove Road to Huntmar Road. South along Huntmar Road and Iber Road to the Stittsville Collector Sewer situated along Abbott Street East.
- Alternative II – South, parallel to the west side of the Carp River and through the proposed development lands to the Glen Cairn stormwater pond. East to Terry Fox Drive, then south along Terry Fox Drive to the Stittsville Collector Sewer.
- Alternative III – East on Maple Grove Road to Terry Fox Drive. South on Terry Fox Drive to the Stittsville Collector Sewer.

##### **4.2.4.2 Evaluation of Temporary Forcemain Alternatives**

The temporary forcemain alternatives were evaluated and ranked using the criteria discussed in Section 4.2. The results of the evaluation are summarized in **Table 4.1-4**. An explanation of the category rankings and weightings are provided below.

TABLE 4.1-3



Kanata West Wastewater - Internal Servicing Alternatives

Criteria	Indicators	Weighting	Rationale for Relative Weights	Alternatives			
				Internal Servicing			
				I	II	III	IIIA
<b>CONSTRUCTABILITY/FUNCTIONALITY</b>		<b>36%</b>		<b>14</b>	<b>14</b>	<b>23</b>	<b>22</b>
CO1.1	Geotechnical Issues and Construction Risks	Potential for encountering poor soils and/or elevated groundwater conditions.		3	3	3	3
CO1.2	Infrastructure Requirements	Extent of works required.	Alt. I and II require two pumping stations. Alt. III and IIIA require one pumping station. All Alts. require the same amount of piping.	1	1	3	3
CO1.3	Operational Impacts	Amount of maintenance intensive infrastructure required.	Alt. I and II (with two pumping stations) have more maintenance intensive infrastructure.	1	1	2	2
CO1.4	Construction Scheduling	Impact of construction on development timing.		3	3	3	3
CO1.5	Property Acquisition	Ease of property acquisition. (Depends on status of lands and adjacent lands, i.e. vacant, leased or owner occupied.)	Alt. III requires the least amount of property acquisition with only one pumping station located on active developers lands and using the Carp River corridor for sewer alignment.	2	2	4	3
CO1.6	System Reliability	Proximity of a storm sewer, SWM or other surface water for emergency overflow	Alt. I and II have pumping stations remotely located relative to proposed storm ponds.	2	2	4	4
CO1.7	Servicing Flexibility	Ease of accommodating potential changes in servicing plans.	The more central location of the main pumping station to the tributary area makes Alt. III and IIIA more flexible to change.	2	2	4	4
<b>ECONOMY</b>		<b>25%</b>		<b>11</b>	<b>11</b>	<b>18</b>	<b>18</b>
E1	Potential to Use Combined Service Corridor	Length and area of combined service corridor.	Alt. III and IIIA service the entire area south of Hwy 417 with one pumping station.	2	2	4	4
E2	Efficiency of Use of Existing Infrastructure	Use of existing capacity		4	4	4	4
E3	Energy Consumption	Pumping requirements	Alt. I and II requires double pumping of a significant portion of the service area.	1	1	3	3
E5	Impact on Agriculture	Agriculture area likely to be affected by infrastructure.		3	3	3	3
E9	Capital Cost	Estimated cost of construction.	Alt. I and II are significantly more expensive primarily due to the cost of two pumping stations.	1	1	4	4
<b>CARING AND HEALTHY COMMUNITIES</b>		<b>25%</b>		<b>10</b>	<b>10</b>	<b>10</b>	<b>10</b>
C3	Displacement of Residents, Community/Recreation Features and Institutions.	Affects areas of residence, institutions or businesses.		4	4	4	4
C4	Disruption to Existing Community	Extent of works affecting existing residences and businesses and visibility of additional infrastructure.		3	3	3	3
C9	Consistency with Planned Land Use and Infrastructure	Compatibility with City land use, design guidelines and infrastructure servicing corridor planning (Kanata West Roadwork Environmental Study Report and Storm Sewer and Watermain Needs).		3	3	3	3
<b>NATURAL ENVIRONMENT</b>		<b>14%</b>		<b>13</b>	<b>9</b>	<b>11</b>	<b>14</b>
N1	Impact on Significant Natural Features	Loss of natural area due to installation of works.	Alts. II & III require a significant amount of work inside the Carp River corridor.	4	2	2	3
N3	Impact on Aquatic Systems	Potential impact on fish habitat due to installation of works.	Alts. II & III require a significant amount of work inside the Carp River corridor.	3	2	2	3
N4	Impact on Quality and Quantity of Surface Water and Groundwater	Potential impact on water quality in the Carp River resulting from rare emergency overflows to the SWM pond due to pump station failure.	Alts. I & II require two pumping stations for each alternative. Alts. III & IIIA require only one station each.	2	2	3	3
N5	Impact on Global Warming	Difference in carbon dioxide emissions resulting from occasional use of diesel generator.	Alt. I and II require double pumping where Alt. III and IIIA only require single pumping.	1	1	2	2
N6	Effects on Urban Greenspace, Open Space and Vegetation (i.e. trees, shrubs, etc.)	Disruption to greenspace and trees.	Alt. III requires work within the Carp River Corridor.	3	2	2	3
<b>Total Score</b>		<b>100%</b>		<b>2.39</b>	<b>2.26</b>	<b>3.21</b>	<b>3.29</b>
<b>Ranking</b>				<b>3</b>	<b>4</b>	<b>2</b>	<b>1</b>
<b>Estimated Capital Cost (in \$million)</b>				<b>8.5</b>	<b>8.5</b>	<b>5.5</b>	<b>5.5</b>

**Description of Alternatives**

Internal Servicing Alternative I - Silver Seven Road at HWY 417  
 Internal Servicing Alternative II - HWY 417 East of Carp River  
 Internal Servicing Alternative III - Maple Grove Road West of the Carp River  
 Internal Servicing Alternative IIIA - Maple Grove Road West of the Carp River with an Alternative Sewer Alignment

**Evaluation Ranking**

1 - 2 High or Negative Impact  
 3 Moderate or No Impact  
 4-5 Low or Positive Impact

**Constructability/Functionality (C/F) –36%**

All three alternatives require the construction of a shallow forcemain so geotechnical issues are not considered to be a concern along the selected alignments. However, an assessment of the subsurface conditions indicates that unlike Alternative III, Alternatives I and II will not require rock excavation.

Alternatives I and III are located entirely within existing or proposed road allowances eliminating the need for additional land or easements. A benefit of Alternative II is that the length of the require forcemain is moderately less than Alternatives I and III.

Alternative I is advantageous for routine maintenance operations as the alignment is located within a lower classification of roadway when compared to Alternative III.

**Economy (E) – 25%**

Approximately 50% of the Alternative I forcemain will be installed in conjunction with other development works minimizing the amount of reinstatement required. This reduces the overall cost of Alternative I relative to the other remaining options. A large portion of Alternative II would be constructed in open fields requiring fewer costs for reinstatement when compared to Alternative III.

**Caring and Healthy Community (CHC) – 25%**

All three alternatives present similar impacts to the community. These impacts are limited to the duration of construction and are therefore considered minimal. Alternative I creates the least amount of impact when compared to Alternatives II and III. This is due to the fact that approximately half of the construction of the temporary forcemain will be done with other development works. Alternative II requires construction along major arterials within existing communities east of the KWCP, resulting in the highest level of impact.

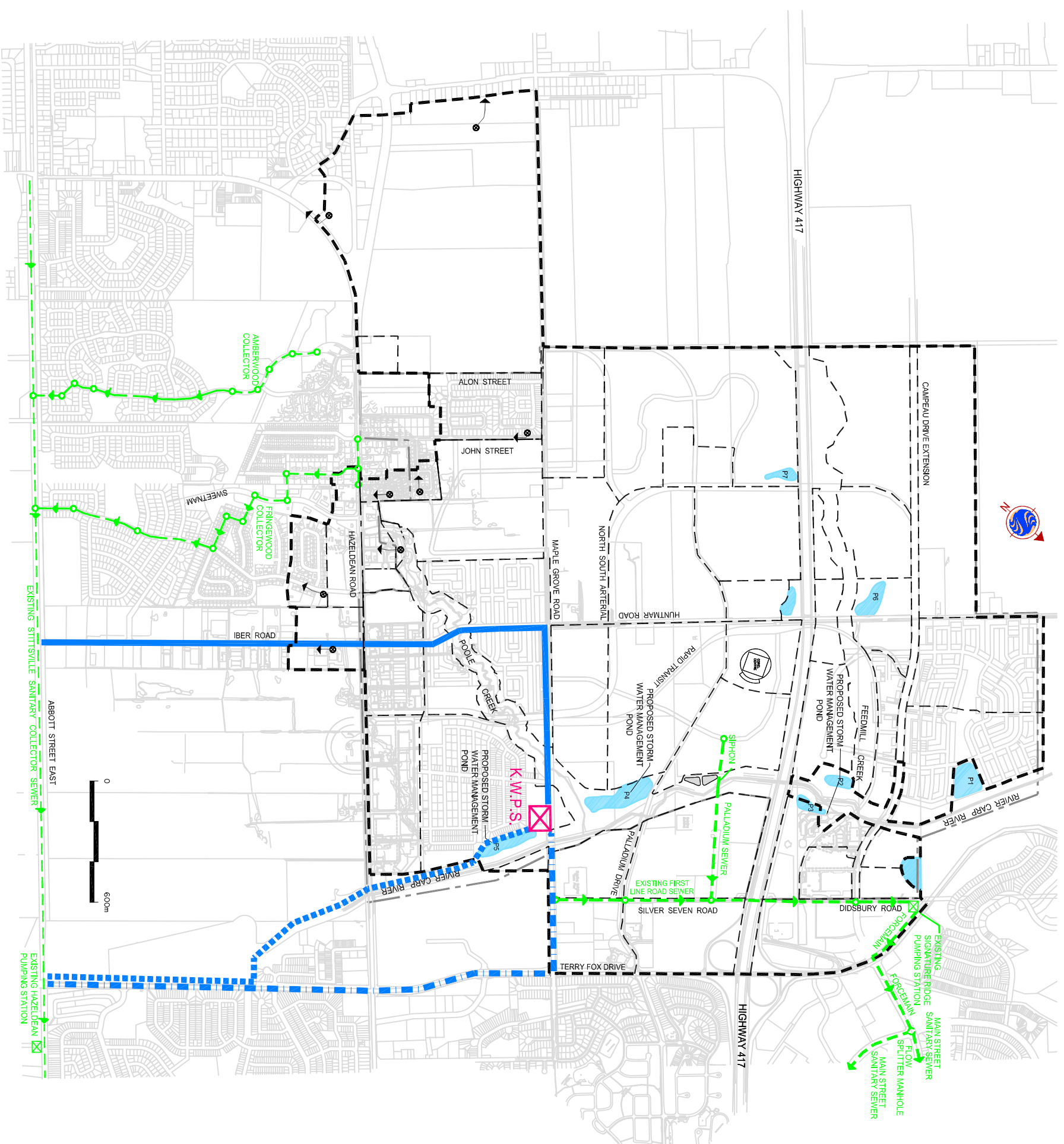
**Natural Environment (NE) – 14%**

Alternatives I and III are entirely contained within existing or proposed road allowances. However, Alternative III would require a crossing at the Carp River. Construction monitoring to detect any required mitigation measures for potential impacts to water quality would be required. A large portion of the Alternative II alignment is within the Carp River corridor and will have the highest impact on existing natural features.

**4.2.4.3 Selection of Preferred Temporary Forcemain Alternative**

Based on the above evaluation, temporary forcemain Alternative I, the Huntmar Road/Iber Road alignment, is selected as the preferred alternative. This alignment facilitates routine maintenance operations, as it is located within a roadway of lower classification when compared to the other alternatives (Terry Fox Drive). This alignment also results in the least amount of impact on the existing natural features. The Alternative I alignment is similar to Alternative II as the most economical options. Over half of the alignment will be constructed in conjunction with other works, unlike Alternative II.

# TEMPORARY SANITARY FORCEMAIN ALTERNATIVES



- Ultimate Major Drainage Limit
- Alternate I (Preferred Option)
- - - Alternate II
- . - . Alternate III
- - - Existing Trunk Sewers
- Existing Pumping Station and Forcemain (To be Decommissioned)
- ⊗ Pump Station

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FIG. 4.1-7

TABLE 4.1-4

Kanata West Wastewater - Temporary Forcemain Alternatives



Criteria	Indicators	Weighting	Rationale for Relative Weights	Temporary Forcemain Alternatives		
				I	II	III
<b>CONSTRUCTABILITY/FUNCTIONALITY</b>			<b>36%</b>	<b>21</b>	<b>18</b>	<b>20</b>
CO1.1	Geotechnical Issues and Construction Risks	Potential for encountering poor soils and/or elevated groundwater conditions.	Alt. III requires crossing of the Carp River through deep clay deposits.	3	3	2
CO1.2	Infrastructure Requirements	Extent of works required.		2	2	2
CO1.3	Operational Impacts	Amount of maintenance intensive infrastructure required.		3	3	3
CO1.4	Construction Scheduling	Impact of construction on development timing.		3	3	3
CO1.5	Property Acquisition	Ease of property acquisition. (Depends on status of lands and adjacent lands, i.e. vacant, leased or owner occupied.)	Alt. II requires property acquisition from private owners.	4	1	4
CO1.6	System Reliability	Proximity of a storm sewer, SWM or other surface water for emergency overflow		3	3	3
CO1.7	Servicing Flexibility	Ease of accommodating potential changes in servicing plans.		3	3	3
<b>ECONOMY</b>			<b>25%</b>	<b>16</b>	<b>14</b>	<b>13</b>
E1	Potential to Use Combined Service Corridor	Length and area of combined service corridor.	Alts. I uses a common corridor with other new works for half of length. Alt. II requires a new single use corridor for 1/3 of its length.	4	1	2
E2	Efficiency of Use of Existing Infrastructure	Use of existing capacity		3	3	3
E3	Energy Consumption	Pumping requirements		3	3	3
E5	Impact on Agriculture	Agriculture area likely to be affected by infrastructure.		3	3	3
E9	Capital Cost	Estimated cost of construction.	Alt. II is the least expensive and Alt. III is the most expensive to install.	3	4	2
<b>CARING AND HEALTHY COMMUNITIES</b>			<b>25%</b>	<b>8</b>	<b>5</b>	<b>7</b>
C3	Displacement of Residents, Community/Recreation Features and Institutions.	Affects areas of residence, institutions or businesses.	Alt. II is adjacent to Carp River corridor.	3	2	3
C4	Disruption to Existing Community	Extent of works affecting existing residences and businesses and visibility of additional infrastructure.	Alt. II and III are along major arterials in existing communities.	3	1	2
O9	Consistency with Planned Land Use and Infrastructure	Competibility with City land use, design guidelines and infrastructure servicing corridor planning (Kanata West Roadwork Environmental Study Report and Storm Sewer and Watermain Needs).		2	2	2
<b>NATURAL ENVIRONMENT</b>			<b>14%</b>	<b>15</b>	<b>9</b>	<b>14</b>
N1	Impact on Significant Natural Features	Loss of natural area due to installation of works.	Alt. II is adjacent to Carp River corridor.	3	1	2
N3	Impact on Aquatic Systems	Potential impact on fish habitat due to installation of works.	Alt. II is adjacent to the Carp River corridor which presents a potential for impacts to aquatic systems..	3	2	3
N4	Impact on Quality and Quantity of Surface Water and Groundwater	Potential impact on water quality in the Carp River resulting from rare emergency overflows to the SWM pond due to pump station failure.	Alt. II requires construction along a significant portion of the Carp River corridor which is currently vegetated.	3	2	3
N5	Impact on Global Warming	Difference in carbon dioxide emissions resulting from occasional use of diesel generator.		3	3	3
N6	Effects on Urban Greenspace, Open Space and Vegetation (i.e.trees,shrubs,etc.)	Disruption to greenspace and trees.	Alt. II is adjacent to Carp River corridor which presents a potential for impacts to aquatic systems..	3	1	3
<b>Total Score</b>			<b>100%</b>	<b>2.93</b>	<b>2.29</b>	<b>2.52</b>
<b>Ranking</b>				<b>1</b>	<b>3</b>	<b>2</b>
<b>Estimated Capital Cost (in \$million)</b>				<b>1.5</b>	<b>1.5</b>	<b>2</b>

Description of Alternatives

Temporary Forcemain Alternative I - Maple Grove/Huntmar/Iber Road to the Stittsville Collector  
 Temporary Forcemain Alternative II- Carp River/Terry Fox to the Stittsville Collector  
 Temporary Forcemain Alternative III- Maple Grove/Terry Fox to the Stittsville Collector

Evaluation Ranking

1 -2 High or Negative Impact  
 3 Moderate or No Impact  
 4-5 Low or Positive Impact

## 4.2.5 Trunk Sewer Alignment Alternatives

### 4.2.5.1 Description of Trunk Sewer Alignment Alternatives

Three potential alignments were considered for the gravity sewer that will service the un-serviced lands on Hazeldean Road. This sewer will also permit the decommissioning of several small existing pumping stations located along the north limit of the Village of Stittsville. As illustrated in **Figure 4.1-8** the alternative alignments considered for this sewer are:

- Alternative I - Maple Grove Road from the proposed pumping station to Huntmar Road, south on Huntmar Road to Hazeldean Road at Iber Road.
- Alternative II - Maple Grove Road to south of Poole Creek, southerly along Poole Creek to the transit corridor, southerly along the transitway to Hazeldean Road at Iber Road.
- Alternative III - South from the Maple Grove Road Pumping Station through the proposed development lands adjacent to the Carp River to Hazeldean Road, west on Hazeldean Road to Iber Road.

### 4.2.5.2 Evaluation of the Trunk Sewer Alignment Alternatives

The alternative sewer alignments were evaluated and ranked using the criteria discussed in Section 4.2. The results of the evaluation are summarized in **Table 4.1-5**. . An explanation of the category rankings and weightings are provided below.

#### **Constructability/Functionality (C/F) – 36%**

All three alternatives require approximately the same depth of excavation and present similar geotechnical issues. A benefit of Alternative I is that at least half of the works will be installed in conjunction with other infrastructure. In addition, the Alternative I alignment will be installed in a corridor that will be part of Phase One of construction providing flexibility in phasing works outside the KWCP area.

Alternatives I and II require the least amount of infrastructure to reach Iber Road.

#### **Economy (E) – 25%**

Alternatives I and II offer the opportunity to use combined service corridors along Maple Grove Road and Huntmar Road (Alternative I) and Hazeldean Road and the transitway (Alternative II). Alternative I would be part of Phase 1 of construction and will ensure that the timing of installation will coincide with other joint use utilities. This ensures that the economies of the combined corridor servicing will materialize for Alternative I.

Alternatives I and II are the least costly to install as they require the least amount of infrastructure.

#### **Caring and Healthy Community (CHC) – 25%**

There are no significant differences between the three alternatives in terms of the impact on the community. The alignment of all three alternatives is primarily confined to within the development area. Impacts will be confined to the period of construction in all cases.

#### **Natural Environment (NE) – 14%**

All three sewer alignment alternatives have a similar impact on the environment. Each alignment is confined to existing right-of-ways or in new right-of-ways proposed within the development area. Alternative I requires crossing Poole Creek that may impact water quality.

##### **4.2.5.3 Selection of Preferred Huntmar Road Sewer Alignment Alternative**

Based on the above evaluation, Huntmar Road sewer Alternative I is selected as the preferred alignment for the gravity sewer. This sewer will service Hazeldean Road and the southern portion of the KWCP. The alignment is preferred because it maximizes the flexibility for development within the KWCP without compromising the surrounding communities or natural environment.

#### **4.2.6 Signature Ridge Pumping Station Alternatives**

##### **4.2.6.1 Description of Signature Ridge Pumping Station Alternatives**

The Signature Ridge Pumping Station is a critical element for providing sanitary service to the KWCP. The present condition of the station is insufficient to provide the necessary level of service required to service the proposed area. To the capacity, two alternatives were considered for the Station. The station can be upgraded (Alternative II) or it can be completely rebuilt (Alternative I), including the construction of a new wet well, pumps and auxiliary power facility. Upgrade recommendations have been described in the “Signature Ridge Pumping Station Feasibility Study” by R.V. Anderson Assoc. Ltd., dated Sept. 2003.

These alternatives were considered because of the significant amount of infrastructure that is currently dependent on the Signature Ridge Pumping Station for an outlet. The station is also located in close proximity to the northeast portion of the KWCP. **Figure 4.1-9** illustrates the location of the Signature Ridge Pumping Station.

##### **4.2.6.2 Evaluation of Signature Ridge Pumping Station Alternatives**

The alternative pumping station alternatives were evaluated and ranked using the criteria discussed in Section 4.2. The results of the evaluation are summarized in **Table 4.1-6**. An explanation of the category rankings and weightings are provided below.

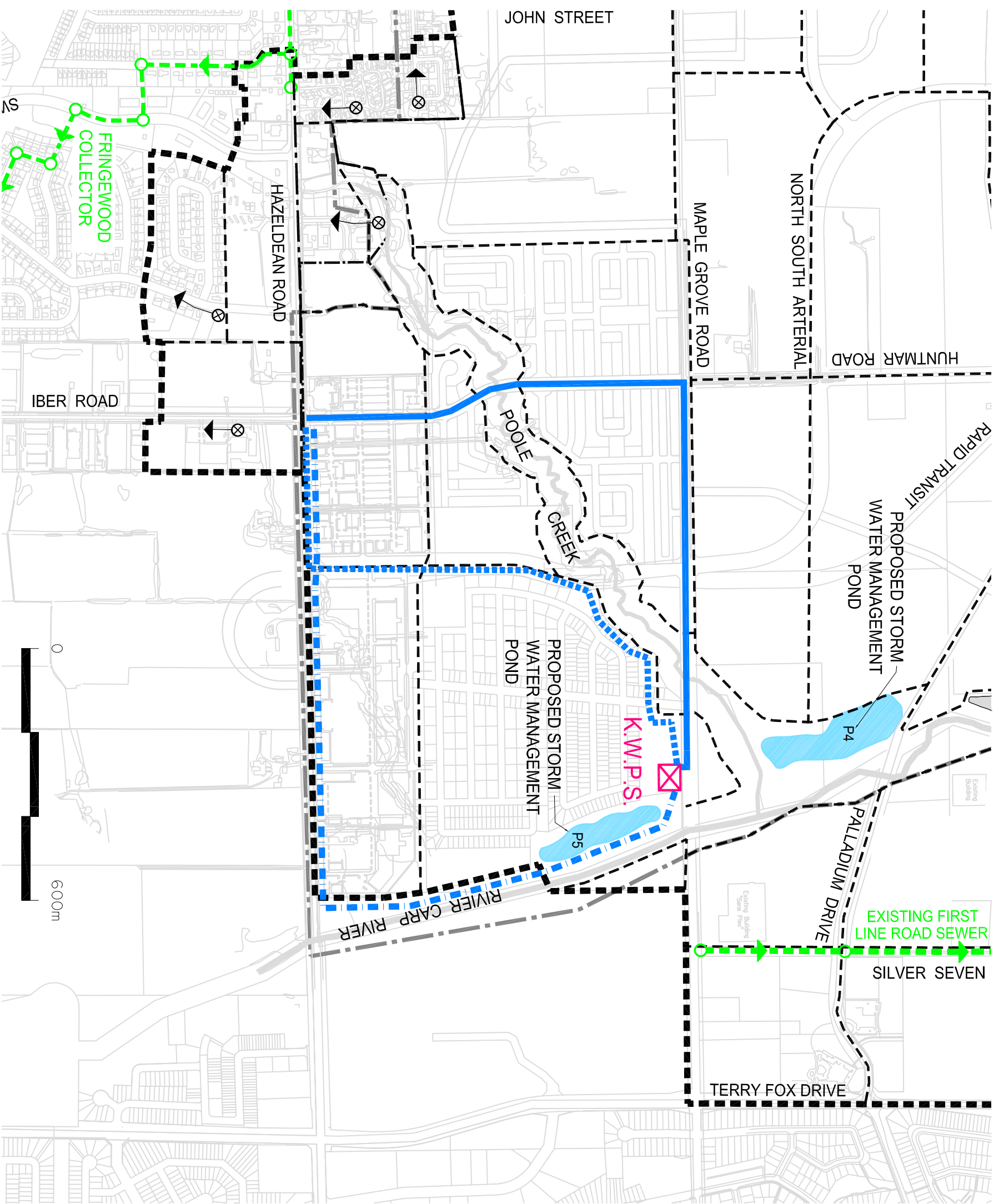
#### **Constructability/Functionality (C/F) 36%**

The Signature Ridge Pumping Station requires only mechanical upgrades to provide the necessary level of service, which can be accomplished through Alternative I (Station up-grade). This eliminates the need to perform deep excavations in soft clays for reconstruction of the wet well. A benefit of constructing a new pumping station would be the ability to increase the pumping capacity to more than that required for the KWCP, increasing the flexibility of the overall wastewater system.

Upgrading the existing station will not require any property acquisition and can be completed in stages to meet the needs of future development over time.



# TRUNK SEWER ALIGNMENT ALTERNATIVES



- Ultimate Major Drainage Limit
- Alternate I (Preferred Option)
- ..... Alternate II
- - - Alternate III
- - - Existing Trunk Sewers
- Existing Pumping Station and Forcemain (To be Decommissioned)
- ⊗ Pump Station

FIG. 4.1-8

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TABLE 4.1-5

Kanata West Wastewater - Trunk Sewer Alternatives



Criteria	Indicators	Weighting	Rationale for Relative Weights	Trunk Sewer Alternatives		
				I	II	III
<b>CONSTRUCTABILITY/FUNCTIONALITY</b>			<b>36%</b>	<b>27</b>	<b>17</b>	<b>19</b>
CO1.1	Geotechnical Issues and Construction Risks	Potential for encountering poor soils and/or elevated groundwater conditions.	7%	3	3	3
CO1.2	Infrastructure Requirements	Extent of works required.	7%	3	3	2
CO1.3	Operational Impacts	Amount of maintenance intensive infrastructure required.	6%	3	3	3
CO1.4	Construction Scheduling	Impact of construction on development timing.	4%	5	2	2
CO1.5	Property Acquisition	Ease of property acquisition. (Depends on status of lands and adjacent lands, i.e. vacant, leased or owner occupied.)	2%	5	2	5
CO1.6	System Reliability	Proximity of a storm sewer, SWM or other surface water for emergency overflow	6%	3	3	3
CO1.7	Servicing Flexibility	Ease of accommodating potential changes in servicing plans.	5%	5	1	1
<b>ECONOMY</b>			<b>25%</b>	<b>17</b>	<b>15</b>	<b>12</b>
E1	Potential to Use Combined Service Corridor	Length and area of combined service corridor.	6%	5	3	2
E2	Efficiency of Use of Existing Infrastructure	Use of existing capacity	5%	3	3	3
E3	Energy Consumption	Pumping requirements	4%	3	3	3
E5	Impact on Agriculture	Agriculture area likely to be affected by infrastructure.	2%	3	3	3
E9	Capital Cost	Estimated cost of construction.	8%	3	3	1
<b>CARING AND HEALTHY COMMUNITIES</b>			<b>25%</b>	<b>9</b>	<b>9</b>	<b>9</b>
C3	Displacement of Residents, Community/Recreation Features and Institutions.	Affects areas of residence, institutions or businesses.	6%	3	3	3
C4	Disruption to Existing Community	Extent of works affecting existing residences and businesses and visibility of additional infrastructure.	11%	3	3	3
C9	Consistency with Planned Land Use and Infrastructure	Compatibility with City land use, design guidelines and infrastructure servicing corridor planning (Kanata West Roadwork Environmental Study Report and Storm Sewer and Watermain Needs).	8%	3	3	3
<b>NATURAL ENVIRONMENT</b>			<b>14%</b>	<b>13</b>	<b>15</b>	<b>15</b>
N1	Impact on Significant Natural Features	Loss of natural area due to installation of works.	3%	2	3	3
N3	Impact on Aquatic Systems	Potential impact on fish habitat due to installation of works.	3%	2	3	3
N4	Impact on Quality and Quantity of Surface Water and Groundwater	Potential impact on water quality in the Carp River resulting from rare emergency overflows to the SWM pond due to pump station failure.	3%	3	3	3
N5	Impact on Global Warming	Difference in carbon dioxide emissions resulting from occasional use of diesel generator.	1%	3	3	3
N6	Effects on Urban Greenspace, Open Space and Vegetation (i.e. trees, shrubs, etc.)	Disruption to greenspace and trees.	5%	3	3	3
<b>Total Score</b>			<b>100%</b>	<b>3.29</b>	<b>2.84</b>	<b>2.61</b>
<b>Ranking</b>				<b>1</b>	<b>2</b>	<b>3</b>
<b>Estimated Capital Cost (in \$million)</b>				<b>1.5</b>	<b>1.5</b>	<b>2.5</b>

**Description of Alternatives**  
 Trunk Sewer Alternative I - Maple Grove/Huntmar/Hazeldean Road  
 Trunk Sewer Alternative II- Maple Grove/Poole Creek/Transitway/Hazeldean Road  
 Trunk Sewer Alternative III - Maple Grove/Hazeldean Road

**Evaluation Ranking**  
 1 -2 High or Negative Impact  
 3 Moderate or No Impact  
 4-5 Low or Positive Impact

#### **Economy (E) 25%**

The reconstruction of the Signature Ridge Pumping Station is significantly more than the costs to upgrade the existing station.

#### **Caring and Healthy Community (CHC) 25%**

In terms of the impact on the Community, there are no significant differences between the two alternatives.

#### **Natural Environment (NE) 14%**

There are no significant differences between the two options with respect to impacts to the natural environment. Both alternatives require the construction of an emergency overflow to the Carp River. Impacts to surface water quality as a result of potential station overflows during an emergency situation are not expected to occur. Should an overflow occur for either alternative, the impacts would be mitigated by a SWM pond. Increases in CO<sub>2</sub> emissions as a result of the use of diesel generators during power failures or maintenance procedures will be negligible and are similar in both alternatives.

#### **4.2.6.3 Selection of Preferred Signature Ridge Pumping Station Alternative**

Based on the above evaluation, the Signature Ridge Pumping Station Alternative I, station upgrade, is selected as the preferred alternative. This alternative maximizes the use of existing infrastructure and offers the most flexibility in phasing of the works with the least amount of capital expenditure or impacts.

#### **4.2.6.4 Summary**

The preferred alternatives selected for the wastewater outlet, the internal servicing system, the temporary forcemain, the trunk sewer alignment, and the Signature Ridge Pumping Station have been used to develop a comprehensive wastewater servicing plan for the KWCP. This servicing plan is discussed in future detail in the following section of this report.

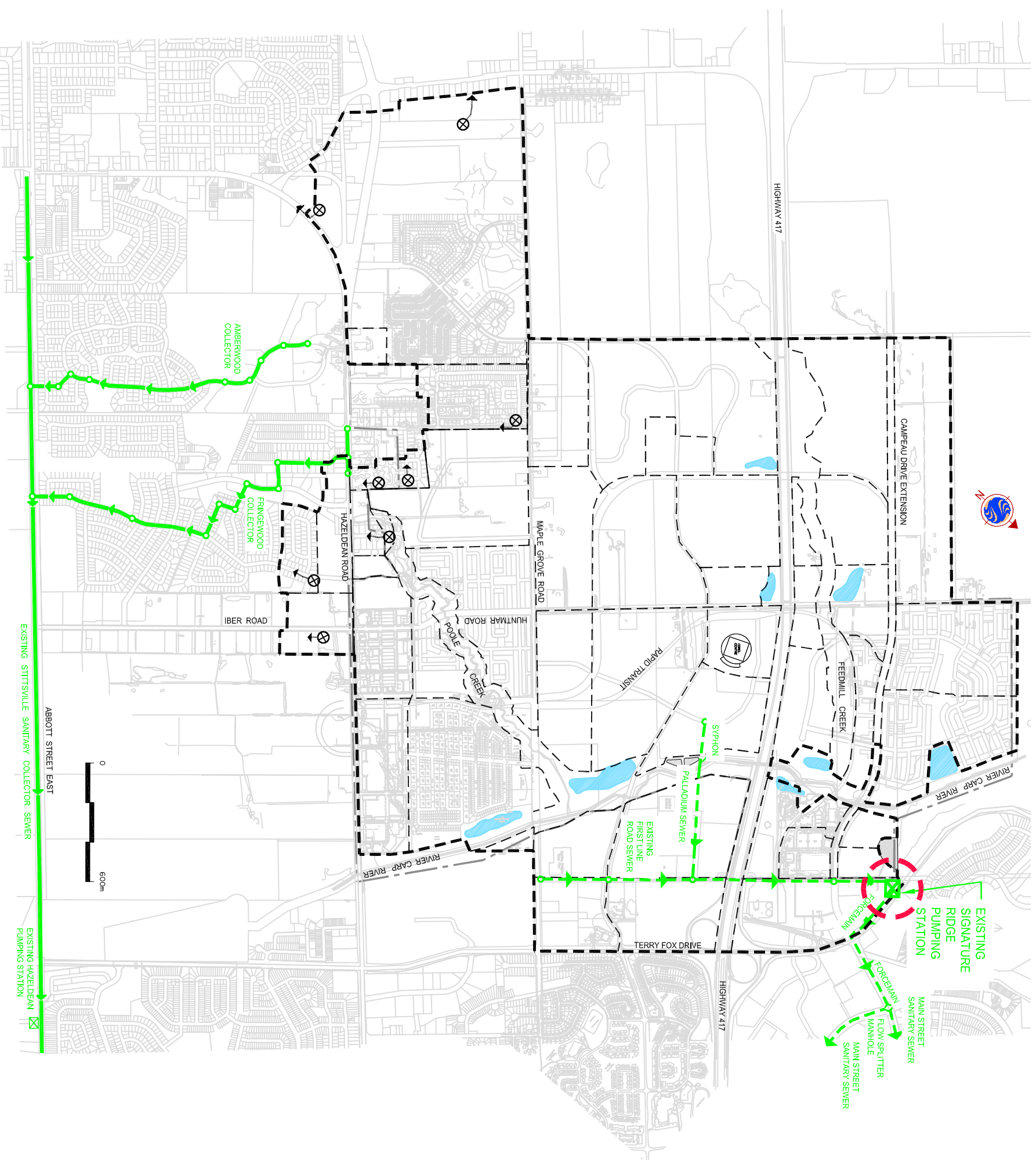
### **4.3 Preferred Sanitary Sewer Servicing Plan**

Section 4.2 has detailed the selection of preferred alternatives for the major infrastructure required to provide sanitary sewer service to the KWCP. These preferred alternatives have been used to develop a Master Sanitary Servicing Plan for the area. This plan is illustrated on **Drawing S-1** (appended to this report). The major features of this plan are:

- (i.) An upgraded Signature Ridge Pumping Station (SRPS) to service all the KWCP lands north of the Queensway, the existing urban area north of the Queensway currently proposed to drain to the SRPS, and the Broughton/Richardson Interstitial lands. A spreadsheet detailing the exact areas and flows tributary to the SRPS is included in **Figure 4.2-1**.

The 400 l/sec peak flow capacity identified in **Figure 4.2-1** for the upgraded SRPS, is consistent with the findings of the R.V. Anderson Report titled "Signature Ridge Pumping Station Upgrades Feasibility Study".

# SIGNATURE RIDGE PUMPING STATION LOCATION



- Legend:**
- Ultimate Drainage Limit
  - Existing Stittsville Sewer
  - - - Existing Trunk Sewer
  - ⊗ → Existing Pumping Station and Forcemain (To be Decommissioned)

MAY 2006

TABLE 4.1-6



Kanata West Wastewater - Temporary Forcemain/Trunk Sewer/Signature Ridge Alternatives

Criteria		Indicators	Weighting	Rationale for Relative Weights	Signature Ridge PS Alternative	
					Upgrade	Rebuild
<b>CONSTRUCTABILITY/FUNCTIONALITY</b>			<b>36%</b>		<b>24</b>	<b>16</b>
CO1.1	Geotechnical Issues and Construction Risks	Potential for encountering poor soils and/or elevated groundwater conditions.	7%	Alt. II requires reconstruction of the pumping station in very soft clays where Alt. I does not require reconstruction of the wet well.	3	1
CO1.2	Infrastructure Requirements	Extent of works required.	7%	Alt. I only requires upgrading of hardware within the existing pumping station.	4	1
CO1.3	Operational Impacts	Amount of maintenance intensive infrastructure required.	6%		3	3
CO1.4	Construction Scheduling	Impact of construction on development timing.	4%	Alt. I can be phased to suit development timing where Alt. II requires a lengthy total reconstruction program.	4	2
CO1.5	Property Acquisition	Ease of property acquisition. (Depends on status of lands and adjacent lands, i.e. vacant, leased or owner occupied.)	2%	Alt. II requires property acquisition for a new station because existing station will have to remain in service during construction.	5	2
CO1.6	System Reliability	Proximity of a storm sewer, SWM or other surface water for emergency overflow	6%		3	3
CO1.7	Servicing Flexibility	Ease of accommodating potential changes in servicing plans.	5%	Alt. II can be built to accommodate changes where Alt. I is designed to the maximum.	2	4
<b>ECONOMY</b>			<b>25%</b>		<b>19</b>	<b>12</b>
E1	Potential to Use Combined Service Corridor	Length and area of combined service corridor.	6%		3	3
E2	Efficiency of Use of Existing Infrastructure	Use of existing capacity	5%	Alt. I maximizes the use of the existing station.	5	2
E3	Energy Consumption	Pumping requirements	4%		3	3
E5	Impact on Agriculture	Agriculture area likely to be affected by infrastructure.	2%		3	3
E9	Capital Cost	Estimated cost of construction.	8%	Alt. II is significantly more expensive to construct.	5	1
<b>CARING AND HEALTHY COMMUNITIES</b>			<b>25%</b>		<b>12</b>	<b>9</b>
C3	Displacement of Residents, Community/Recreation Features and Institutions.	Affects areas of residence, institutions or businesses.	6%		4	4
C4	Disruption to Existing Community	Extent of works affecting existing residences and businesses and visibility of additional infrastructure.	11%	Alt. 1 requires only internal up-grades and will have minimal construction traffic or related impacts.	4	3
G9	Consistency with Planned Land Use and Infrastructure	Compatibility with City land use, design guidelines and infrastructure servicing corridor planning (Kanata West Roadwork Environmental Study Report and Storm Sewer and Watermain Needs).	8%	Alt. I maximizes use of currently planned infrastructure by upgrading existing station to its maximum potential.	4	2
<b>NATURAL ENVIRONMENT</b>			<b>14%</b>		<b>14</b>	<b>14</b>
N1	Impact on Significant Natural Features	Loss of natural area due to installation of works.	3%		3	3
N3	Impact on Aquatic Systems	Potential impact on fish habitat due to installation of works.	3%		3	3
N4	Impact on Quality and Quantity of Surface Water and Groundwater	Potential impact on water quality in the Carp River resulting from rare emergency overflows to the SWM pond due to pump station failure.	3%		3	3
N5	Impact on Global Warming	Difference in carbon dioxide emissions resulting from occasional use of diesel generator.	1%		2	2
N6	Effects on Urban Greenspace, Open Space and Vegetation (i.e.trees,shrubs,etc.)	Disruption to greenspace and trees.	5%		3	3
<b>Total Score</b>			<b>100%</b>		<b>3.60</b>	<b>2.48</b>
<b>Ranking</b>					<b>1</b>	<b>2</b>
<b>Estimated Capital Cost (in \$million)</b>					<b>1</b>	<b>4</b>

**Description of Alternatives**  
 Signature Ridge PS Alternative 1 - Rebuild  
 Signature Ridge PS Alternative II - Upgrade

**Evaluation Ranking**  
 1 -2 High or Negative Impact  
 3 Moderate or No Impact  
 4-5 Low or Positive Impact

The Signature Ridge Pumping Station is currently not equipped with catastrophic failure protection in the form of a gravity overflow. A hydraulic analysis of the proposed sewer system was therefore completed to evaluate the potential for providing a gravity overflow. This analysis demonstrates that catastrophic protection can be provided by gravity. The analysis is included in **Appendix 4.2** and demonstrates that overflows to the existing stormwater management pond on First Line Road and to Pond I can provide the necessary level of protection.

- (ii.) A single new pumping station and forcemain located south of Maple Grove Road and west of the Carp River.

This new pumping station ultimately services all the KWCP south of Highway 417, the lands south of the 417 originally tributary to the SRPS, and the lands in the Village of Stittsville, along Hazeldean Road which are currently unserviceable by gravity to the Stittsville Sanitary Sewer System. This new pumping station has also been designed to accommodate the decommissioning of up to eight small public and private pumping stations along Hazeldean Road without deepening the Kanata West system. **Figure 4.2-1** details the exact areas and flows from Stittsville which will ultimately be tributary to the new pumping station. The areas are also illustrated on **Drawing S-1**.

**Figures 4.2-3 and 4.2-4** illustrate a conceptual layout and cross-section for the new pumping station and **Appendix 4.3** details the conceptual design of the pumping station.

The new pumping station will temporarily outlet to the Stittsville Collector Sewer via a temporary forcemain in Huntmar Road and Iber Road. This temporary forcemain is designed to accommodate a flow of 190 l/sec (approximately 3,000 units). The temporary outlet will be located entirely within a public right-of-way. The single 405 mm diameter forcemain used for the initial outlet can be kept in service for long-term use as an emergency back up outlet. Rationale on the availability of capacity in the Stittsville Collector Sewer is attached as **Appendix 4.1**.

The permanent outlet for the new pumping station consists of a forcemain leading from the pumping station to the Glen Cairn Collector Sewer east of Eagleson Road. The preferred route for this forcemain is along Maple Grove Road to Silver Seven Road; along the east side of Silver Seven Road, in an easement, in the undeveloped lands between Maple Grove Road and Palladium Drive; easterly along Palladium Drive to Katimavik Road; and easterly along the north side of Katimavik Road, in the corridor for the unbuilt westbound lanes of Katimavik Road, to Eagleson Road and the Glen Cairn Collector Sewer. The location of the new pumping station is in close proximity to Stormwater Management Ponds 4 and 5. This provides catastrophic failure protection to the new pumping station in the form of a gravity overflow. The hydraulic analysis of this overflow system is attached as **Appendix 4.2**.

The preferred sanitary sewer system also includes a gravity sewer, which collects flow from several minor internal sanitary sewers and directs this flow to the new pumping station location. As illustrated on **Drawing S-1** this minor collector sewer runs parallel to the west side of the Carp River corridor between Maple Grove Road and Palladium Drive, crossing under the Carp River by boring beneath the river. The sewer extends northerly to intercept flows from Silver Seven Road and diverts them from the Signature Ridge Pumping Station. The inclusion of this north south sewer is a key element in eliminating the need for double pumping within Kanata



**SANITARY SEWER DESIGN SHEET**  
**PROJECT :** Kanata West Servicibility Stury  
**LOCATION :** CITY OF OTTAWA

PAGE 1 OF 1  
 PROJECT: 3598-LD-03  
 DATE: Apr 2005  
 DESIGN: JIM  
 FILE: 3598LD.sewers.XLS

**PHASE 1 SIGNATURE RIDGE (population based criteria..ICI simultaneous peaking)**

STREET	LOCATION		TOTAL AREA (Ha)	RESIDENTIAL							EMPLOYMENT/RETAIL/BUSINESS PARK/OPEN SPACES							INFILTRATION			TOTAL FLOW (l/s)	PROPOSED SEWER												
	FROM MH	TO MH		APPLIC AREA (Ha)	UNIT/Ha	TOTAL UNITS	POPULATION		PEAK FACTOR	PEAK FLOW (l/s)	APPLIC AREA (Ha)	ACCUM AREA (Ha)	TOTAL AREA (Ha)	FLOW RATE (l/Ha/d)	PEAK FLOW			AREA (Ha)		PEAK FLOW (l/s)		CAPACITY l/s	VELOCITY (ful) m/s	LGTH. (m)	PIPE (mm)	GRADE %	AVAIL. CAP. (%)							
							INDIV	ACCUM							INDIV	ACCUM	TOTAL	INDIV	CUMUL									TOTAL CUMUL						
Campeau Drive Trunk Sewer	1	2	0.00							0.00	0.00		35000	0.00	0.00		0.00	0.00																
			0.00							0.00	0.00		35000	0.00	0.00		0.00	0.00																
			0.00							0.00	0.00		50000	0.00	0.00		0.00	0.00																
			0.00							0.00	0.00	0.00	50000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	283.79	1.27	500.0	525	0.40	100.00%			
	2	3	29.19	29.19	19	555	1664	1664	3.65	24.58							0.00	29.19	29.19															
			0.00							24.58				0.00	0.00		50000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	29.19	8.17	32.75	286.61	0.98	700.0	600	0.20	88.57%	
	14	3	0.00										50000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00										
			0.00										50000	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00										
	3	4					1664		3.65	24.58				0.00	0.00		50000	0.00	0.00	29.19	8.17	32.75												
	4A	4	27.86	27.86	19	529	1588	1588	3.66	23.55								27.86	27.86	27.86	7.80	31.36												
	4	5	4.13	1.76	50	88	263	3515	3.38	48.17	2.37	2.37	123.33	35000	1.44	1.44	1.44	4.13	4.13	61.18	17.13	66.74												
Corel Centre Etc. (Existing Sewer)		15	6.05								6.05	6.05		30000	3.15	3.15		6.05																
																					30.00													
			20.15								20.15	26.20	26.20	14400	5.04	8.19	8.19	20.15	26.20	26.20	7.34	45.52												
First Line Road Sewer		15	14.59								14.59	14.59		35000	8.87	8.87		14.59	14.59															
			11.97								11.97	26.56		35000	7.27	16.14		11.97	26.56															
			20.66								20.66	47.22		35000	12.55	28.69		20.66	47.22															
			28.89								28.89	76.11	76.11	35000	17.55	46.25	46.25	28.89	76.11	76.11	21.31	67.56					100.21	0.88	694.0	375	0.30	32.59%		
<b>Totals South Of Queensway To SRPS</b>	<b>15</b>	<b>5A</b>	<b>102.31</b>	<b>0.00</b>	<b>0</b>	<b>0</b>	<b>0</b>	<b>0</b>	<b>0.00</b>	<b>0.00</b>	<b>102.31</b>						<b>54.44</b>	<b>102.31</b>	<b>102.31</b>	<b>58.65</b>	<b>113.08</b>	<b>113.08</b>				<b>203.90</b>	<b>1.24</b>	<b>230.0</b>	<b>450</b>	<b>0.47</b>	<b>44.54%</b>			
	Queensway	5	6.35								6.35	108.66		35000	3.86	58.29		6.35	6.35															
			11.80	5.02	50	251	752	752	3.88	11.81	6.79	115.45	115.45	35000	4.12	62.42	62.42	11.80	18.15	120.46	63.73	137.96				203.90	1.24	420.0	450	0.47	32.34%			
	5	5A	3.88								3.88	119.33		35000	2.36	64.77		3.88	124.34															
			25.54								25.54	144.87	268.20	35000	15.52	81.73	81.73	25.54	149.88	211.06	89.10	230.81				519.43	1.14	300.0	750	0.20	55.56%			
			149.88																		63.73	63.73												
Heritage Hills		5A	90.20	90.20	19	1714	5141	5141	3.23	67.35	0.00							90.20																
Heritage Hills		5A	4.88								4.88	4.88	4.88	50000	4.24	4.24	4.24	4.88	95.08	95.08	26.62	98.21												
Broughton-Richardson / Interstitial		5A																				65.00												
<b>Total To SRPS</b>	<b>5A</b>	<b>SRPS</b>	<b>306.14</b>	<b>154.03</b>	<b>3136</b>	<b>9409</b>	<b>9409</b>	<b>9409</b>	<b>3.23</b>	<b>127.33</b>	<b>152.12</b>						<b>85.97</b>		<b>306.14</b>	<b>115.72</b>	<b>394.02</b>					<b>625.68</b>	<b>1.37</b>	<b>30.0</b>	<b>750</b>	<b>0.29</b>	<b>37.03%</b>			

Average Daily Per capita Flow Rate = 350 l/cap/d  
 Infiltration Allowance Flow Rate = 0.28 l/sec/Ha  
 Residential Peaking Factor = 1+(14/(4+(P^0.5))), P=Pop. in 1000's, Max of 4  
 Population density per unit = 3.00  
 P. F. For Employment/Retail/Business Park = 1.50

**Note: Sewer from node 5 to SRPS is existing and is to be replaced.**

Revision No. 1: April 11, 2005  
 Revision No. 2: April 20, 2005  
 Revision No. 3: June 07, 2005  
 Revision No. 4: Oct. 14, 2005  
 Revision No. 5: Feb. 15, 2006



**FIG. 4.2-2**



**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**Appendix C    STORMWATER MANAGEMENT**

**C.1    STORM SEWER DESIGN SHEET AND ROOF STORAGE  
CALCULATIONS**



Wellings of Stittsville Ph 2- 20 Cedarow Court

**STORM SEWER  
DESIGN SHEET  
(City of Ottawa)**

**DESIGN PARAMETERS**

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

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

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

DATE: 2021-09-01  
 REVISION: 1  
 DESIGNED BY: TR  
 CHECKED BY: -

FILE NUMBER: 160401511

LOCATION			DRAINAGE AREA																PIPE SELECTION																				
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR)	AREA (5-YEAR)	AREA (10-YEAR)	AREA (100-YEAR)	AREA (ROOF)	C (2-YEAR)	C (5-YEAR)	C (10-YEAR)	C (100-YEAR)	A x C (2-YEAR)	ACCUM Ax C (2YR)	A x C (5-YEAR)	ACCUM Ax C (5YR)	A x C (10-YEAR)	ACCUM Ax C (10YR)	A x C (100-YEAR)	ACCUM Ax C (100YR)	T of C (min)	I <sub>2</sub> YEAR (mm/h)	I <sub>5</sub> YEAR (mm/h)	I <sub>10</sub> YEAR (mm/h)	I <sub>100</sub> YEAR (mm/h)	Q <sub>CONTROL</sub> (L/s)	ACCUM Q <sub>CONTROL</sub> (L/s)	Q <sub>ACT</sub> (CIA/360) (L/s)	LENGTH (m)	PIPE WIDTH OR DIAMETE (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE (%)	Q <sub>cap</sub> (FULL) (L/s)	% FULL (-)	VEL (FULL) (m/s)	VEL (ACT) (m/s)	TIME OF FLOW (min)
ROOF 1-ROOF 12. UGPK 1 TO UGPK - 7	BLDG STM STC 300	STM STC 300	0.81	0.00	0.00	0.00	0.76	0.90	0.00	0.00	0.00	0.729	0.729	0.000	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	61.0	61.0	216.5	2.4	450	450	CIRCULAR	CONCRETE	-	1.00	297.4	72.80%	1.81	1.73	0.02
	STM STC 300	STM 100	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.729	0.000	0.000	0.000	0.000	0.000	0.000	10.02	76.72	104.07	122.00	178.35	0.0	61.0	216.4	2.8	450	450	CIRCULAR	CONCRETE	-	1.00	297.4	72.74%	1.81	1.73	0.03
	STM 100	TANK	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.729	0.000	0.000	0.000	0.000	0.000	0.000	10.05	76.61	103.93	121.83	178.10	0.0	61.0	216.1	8.7	450	450	CIRCULAR	CONCRETE	-	1.00	297.4	72.67%	1.81	1.73	0.08
			<b>10.13</b>																																				
	TANK STM 101	STM 101	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.729	0.000	0.000	0.000	0.000	0.000	0.000	10.13	76.29	103.49	121.32	177.34	0.0	0.0	125.0	1.5	525	525	CIRCULAR	CONCRETE	-	0.20	200.6	62.31%	0.90	0.82	0.03
	STM 101	EX STM	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.000	0.729	0.000	0.000	0.000	0.000	0.000	0.000	10.16	76.18	103.33	121.13	177.07	0.0	0.0	154.3	20.8	525	525	CIRCULAR	CONCRETE	-	0.20	200.6	76.88%	0.90	0.87	0.40
			<b>10.56</b>																																				
																			675 675																				

Note:  
 ICD and weir are proposed to be constructed in STM 101 prior to flows discharging to approved outlet, therefore a 450mm diameter pipe is sufficient as flows will be restricted.

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF1 and 2  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0022	0	0.025	21	0	0	0.025
0.050	0.0006	0.0044	1	0.050	85	1	1	0.050
0.075	0.0008	0.0055	5	0.075	191	3	5	0.075
0.100	0.0009	0.0066	11	0.100	339	7	11	0.100
0.125	0.0011	0.0077	22	0.125	530	11	22	0.125
0.150	0.0013	0.0088	38	0.150	763	16	38	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.2	280.0	1.2	0.07778
4.6	608.0	3.4	0.24669
11.1	986.7	6.5	0.52078
21.9	1394.4	10.8	0.90812
38.0	1820.1	16.1	1.41371

**Rooftop Storage Summary**

Total Building Area (sq.m)		954	
Assume Available Roof Area (sq. 80%)		763.2	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		7	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		38	
Estimated 100 Year Drawdown Time (h)		1.1	

**From Watts Drain Catalogue**

Head (m)	L/s					
	Open	75%	50%	25%	Closed	
0.025	0.3155	0.3155	<b>0.3155</b>	0.3155	0.3155	
0.050	0.6309	0.6309	<b>0.6309</b>	0.6309	0.6309	
0.075	0.9464	0.8675	<b>0.7886</b>	0.7098	0.6309	
0.100	1.2618	1.1041	<b>0.9464</b>	0.7886	0.6309	
0.125	1.5773	1.3407	<b>1.1041</b>	0.8675	0.6309	
0.150	1.8927	1.5773	<b>1.2618</b>	0.9464	0.6309	

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.006	0.008	-
Depth (m)	0.077	0.136	0.150
Volume (cu.m)	5.3	29.1	38.2
Draintime (hrs)	0.3	1.1	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF3  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.00126	0	0.025	24	0	0	0.025
0.050	0.0006	0.00252	2	0.050	98	1	2	0.050
0.075	0.0009	0.00379	5	0.075	220	4	5	0.075
0.100	0.0013	0.00505	13	0.100	390	8	13	0.100
0.125	0.0016	0.00631	25	0.125	610	12	25	0.125
0.150	0.0019	0.00757	44	0.150	878	19	44	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.4	564.0	1.4	0.15667
5.3	1020.6	3.9	0.44016
12.8	1490.6	7.5	0.85422
25.2	1966.0	12.4	1.40032
43.7	2444.0	18.5	2.07922

**Rooftop Storage Summary**

Total Building Area (sq.m)		1098	
Assume Available Roof Area (sq. 80%)	80%	878.4	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		4	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		44	
Estimated 100 Year Drawdown Time (h)		1.9	

**From Watts Drain Catalogue**

Head (m)	L/s				
	<b>Open</b>	75%	50%	25%	Closed
0.025	<b>0.3155</b>	0.3155	0.3155	0.3155	0.3155
0.050	<b>0.6309</b>	0.6309	0.6309	0.6309	0.6309
0.075	<b>0.9464</b>	0.8675	0.7886	0.7098	0.6309
0.100	<b>1.2618</b>	1.1041	0.9464	0.7886	0.6309
0.125	<b>1.5773</b>	1.3407	1.1041	0.8675	0.6309
0.150	<b>1.8927</b>	1.5773	1.2618	0.9464	0.6309

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.004	0.007	-
Depth (m)	0.080	0.142	0.150
Volume (cu.m)	7.0	37.9	43.9
Draintime (hrs)	0.5	1.9	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF4  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0016	0	0.025	8	0	0	0.025
0.050	0.0006	0.0032	1	0.050	31	0	1	0.050
0.075	0.0006	0.0032	2	0.075	70	1	2	0.075
0.100	0.0006	0.0032	4	0.100	125	2	4	0.100
0.125	0.0006	0.0032	8	0.125	196	4	8	0.125
0.150	0.0006	0.0032	14	0.150	282	6	14	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.5	144.6	0.5	0.04018
1.7	392.6	1.2	0.14924
4.1	764.6	2.4	0.36162
8.1	1260.5	4.0	0.71176
14.0	1880.4	5.9	1.23411

**Rooftop Storage Summary**

Total Building Area (sq.m)		352	
Assume Available Roof Area (sq. 80%)		281.6	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		5	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		14	
Estimated 100 Year Drawdown Time (h)		0.9	

**From Watts Drain Catalogue**

Head (m)	L/s				
	Open	75%	50%	25%	Closed
0.025	0.3155	0.3155	0.3155	0.3155	<b>0.3155</b>
0.050	0.6309	0.6309	0.6309	0.6309	<b>0.6309</b>
0.075	0.9464	0.8675	0.7886	0.7098	<b>0.6309</b>
0.100	1.2618	1.1041	0.9464	0.7886	<b>0.6309</b>
0.125	1.5773	1.3407	1.1041	0.8675	<b>0.6309</b>
0.150	1.8927	1.5773	1.2618	0.9464	<b>0.6309</b>

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.002	0.003	-
Depth (m)	0.029	0.135	0.150
Volume (cu.m)	0.1	10.5	14.1
Draintime (hrs)	0.0	0.9	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF5  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.00000	0	0.000	0	0	0	0.000
0.025	0.0003	0.00126	0	0.025	24	0	0	0.025
0.050	0.0006	0.00252	2	0.050	98	1	2	0.050
0.075	0.0009	0.00379	5	0.075	220	4	5	0.075
0.100	0.0013	0.00505	13	0.100	390	8	13	0.100
0.125	0.0016	0.00631	25	0.125	610	12	25	0.125
0.150	0.0019	0.00757	44	0.150	878	19	44	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.4	564.0	1.4	0.15667
5.3	1020.6	3.9	0.44016
12.8	1490.6	7.5	0.85422
25.2	1966.0	12.4	1.40032
43.7	2444.0	18.5	2.07922

**Rooftop Storage Summary**

Total Building Area (sq.m)		1098	
Assume Available Roof Area (sq.	80%	878.4	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		4	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		44	
Estimated 100 Year Drawdown Time (h)		1.9	

**From Watts Drain Catalogue**

Head (m)	L/s	Open	75%	50%	25%	Closed
0.025	<b>0.3155</b>	0.3155	0.3155	0.3155	0.3155	0.3155
0.050	<b>0.6309</b>	0.6309	0.6309	0.6309	0.6309	0.6309
0.075	<b>0.9464</b>	0.8675	0.7886	0.7098	0.6309	0.6309
0.100	<b>1.2618</b>	1.1041	0.9464	0.7886	0.6309	0.6309
0.125	<b>1.5773</b>	1.3407	1.1041	0.8675	0.6309	0.6309
0.150	<b>1.8927</b>	1.5773	1.2618	0.9464	0.6309	0.6309

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.004	0.007	-
Depth (m)	0.080	0.142	0.150
Volume (cu.m)	7.0	37.9	43.9
Drainetime (hrs)	0.5	1.9	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF6 and 7  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0022	0	0.025	21	0	0	0.025
0.050	0.0006	0.0044	1	0.050	85	1	1	0.050
0.075	0.0008	0.0055	5	0.075	191	3	5	0.075
0.100	0.0009	0.0066	11	0.100	340	7	11	0.100
0.125	0.0011	0.0077	22	0.125	531	11	22	0.125
0.150	0.0013	0.0088	38	0.150	764	16	38	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
1.2	280.3	1.2	0.07787
4.6	608.7	3.4	0.24694
11.1	987.8	6.5	0.52133
21.9	1395.9	10.8	0.90907
38.0	1822.1	16.1	1.41519

**Rooftop Storage Summary**

Total Building Area (sq.m)		955	
Assume Available Roof Area (sq. 80%)		764	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		7	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		38	
Estimated 100 Year Drawdown Time (h)		1.1	

**From Watts Drain Catalogue**

Head (m)	L/s					
	Open	75%	50%	25%	Closed	
0.025	0.3155	0.3155	<b>0.3155</b>	0.3155	0.3155	
0.050	0.6309	0.6309	<b>0.6309</b>	0.6309	0.6309	
0.075	0.9464	0.8675	<b>0.7886</b>	0.7098	0.6309	
0.100	1.2618	1.1041	<b>0.9464</b>	0.7886	0.6309	
0.125	1.5773	1.3407	<b>1.1041</b>	0.8675	0.6309	
0.150	1.8927	1.5773	<b>1.2618</b>	0.9464	0.6309	

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.006	0.008	-
Depth (m)	0.077	0.136	0.150
Volume (cu.m)	5.3	29.1	38.2
Drain time (hrs)	0.3	1.1	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF8  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0006	0	0.025	2	0	0	0.025
0.050	0.0006	0.0013	0	0.050	9	0	0	0.050
0.075	0.0007	0.0014	0	0.075	19	0	0	0.075
0.100	0.0008	0.0016	1	0.100	34	1	1	0.100
0.125	0.0009	0.0017	2	0.125	54	1	2	0.125
0.150	0.0009	0.0019	4	0.150	78	2	4	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.1	99.7	0.1	0.02768
0.5	240.4	0.3	0.09447
1.1	421.4	0.7	0.21152
2.2	631.6	1.1	0.38695
3.9	863.6	1.6	0.62685

**Rooftop Storage Summary**

Total Building Area (sq.m)		97	
Assume Available Roof Area (sq. 80%)		77.6	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		2	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		4	
Estimated 100 Year Drawdown Time (h)		0.3	

**From Watts Drain Catalogue**

Head (m)	L/s			
	Open	75%	50%	25% Closed
0.025	0.3155	0.3155	0.3155	<b>0.3155</b> 0.3155
0.050	0.6309	0.6309	0.6309	<b>0.6309</b> 0.6309
0.075	0.9464	0.8675	0.7886	<b>0.7098</b> 0.6309
0.100	1.2618	1.1041	0.9464	<b>0.7886</b> 0.6309
0.125	1.5773	1.3407	1.1041	<b>0.8675</b> 0.6309
0.150	1.8927	1.5773	1.2618	<b>0.9464</b> 0.6309

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.000	0.002	-
Depth (m)	0.000	0.117	0.150
Volume (cu.m)	0.0	1.9	3.9
Draintime (hrs)	0.0	0.3	



Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF9  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0006	0	0.025	1	0	0	0.025
0.050	0.0006	0.0013	0	0.050	5	0	0	0.050
0.075	0.0007	0.0014	0	0.075	11	0	0	0.075
0.100	0.0008	0.0016	1	0.100	19	0	1	0.100
0.125	0.0009	0.0017	1	0.125	29	1	1	0.125
0.150	0.0009	0.0019	2	0.150	42	1	2	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.1	54.4	0.1	0.01512
0.3	131.4	0.2	0.05162
0.6	230.2	0.4	0.11557
1.2	345.1	0.6	0.21143
2.1	471.9	0.9	0.34251

**Rooftop Storage Summary**

Total Building Area (sq.m)		53	
Assume Available Roof Area (sq. 80%)	80%	42.4	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		2	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		2	
Estimated 100 Year Drawdown Time (h)		0.1	

**From Watts Drain Catalogue**

Head (m)	L/s			
	Open	75%	50%	25% Closed
0.025	0.3155	0.3155	0.3155	<b>0.3155</b>
0.050	0.6309	0.6309	0.6309	<b>0.6309</b>
0.075	0.9464	0.8675	0.7886	<b>0.7098</b>
0.100	1.2618	1.1041	0.9464	<b>0.7886</b>
0.125	1.5773	1.3407	1.1041	<b>0.8675</b>
0.150	1.8927	1.5773	1.2618	<b>0.9464</b>

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.000	0.002	-
Depth (m)	0.000	0.100	0.150
Volume (cu.m)	0.0	0.6	2.1
Draintime (hrs)	0.0	0.1	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF10  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0066	1	0.025	62	1	1	0.025
0.050	0.0006	0.0132	4	0.050	250	4	4	0.050
0.075	0.0007	0.0149	14	0.075	562	10	14	0.075
0.100	0.0008	0.0166	33	0.100	999	19	33	0.100
0.125	0.0009	0.0182	65	0.125	1561	32	65	0.125
0.150	0.0009	0.0199	112	0.150	2248	47	112	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
3.6	274.9	3.6	0.07637
13.5	663.3	9.9	0.26063
32.8	1162.6	19.3	0.58357
64.5	1742.4	31.7	1.06758
111.9	2382.8	47.4	1.72946

**Rooftop Storage Summary**

Total Building Area (sq.m)		2810	
Assume Available Roof Area (sq. 80%)		2248	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		21	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		112	
Estimated 100 Year Drawdown Time (h)		1.5	

**From Watts Drain Catalogue**

Head (m)	L/s			
	Open	75%	50%	25% Closed
0.025	0.3155	0.3155	0.3155	<b>0.3155</b> 0.3155
0.050	0.6309	0.6309	0.6309	<b>0.6309</b> 0.6309
0.075	0.9464	0.8675	0.7886	<b>0.7098</b> 0.6309
0.100	1.2618	1.1041	0.9464	<b>0.7886</b> 0.6309
0.125	1.5773	1.3407	1.1041	<b>0.8675</b> 0.6309
0.150	1.8927	1.5773	1.2618	<b>0.9464</b> 0.6309

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.017	0.019	-
Depth (m)	0.100	0.141	0.150
Volume (cu.m)	33.1	94.7	112.4
Draintime (hrs)	0.6	1.5	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF11  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0006	0	0.025	2	0	0	0.025
0.050	0.0006	0.0013	0	0.050	9	0	0	0.050
0.075	0.0007	0.0014	1	0.075	21	0	1	0.075
0.100	0.0008	0.0016	1	0.100	38	1	1	0.100
0.125	0.0009	0.0017	2	0.125	59	1	2	0.125
0.150	0.0009	0.0019	4	0.150	85	2	4	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.1	108.9	0.1	0.03025
0.5	262.7	0.4	0.10323
1.2	460.5	0.7	0.23114
2.4	690.2	1.2	0.42285
4.2	943.8	1.8	0.68501

**Rooftop Storage Summary**

Total Building Area (sq.m)		106	
Assume Available Roof Area (sq. 80%)		84.8	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		2	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		4	
Estimated 100 Year Drawdown Time (h)		0.4	

**From Watts Drain Catalogue**

Head (m)	L/s			
	Open	75%	50%	25% Closed
0.025	0.3155	0.3155	0.3155	<b>0.3155</b> 0.3155
0.050	0.6309	0.6309	0.6309	<b>0.6309</b> 0.6309
0.075	0.9464	0.8675	0.7886	<b>0.7098</b> 0.6309
0.100	1.2618	1.1041	0.9464	<b>0.7886</b> 0.6309
0.125	1.5773	1.3407	1.1041	<b>0.8675</b> 0.6309
0.150	1.8927	1.5773	1.2618	<b>0.9464</b> 0.6309

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.000	0.002	-
Depth (m)	0.000	0.120	0.150
Volume (cu.m)	0.0	2.2	4.2
Draintime (hrs)	0.0	0.4	

Roof Drain Design Calculation Sheet

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court  
Roof Drain Design Sheet, Area ROOF12  
Standard Watts Model R1100 Accutrol Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0003	0.0006	0	0.025	2	0	0	0.025
0.050	0.0006	0.0013	0	0.050	9	0	0	0.050
0.075	0.0007	0.0014	1	0.075	20	0	1	0.075
0.100	0.0008	0.0016	1	0.100	36	1	1	0.100
0.125	0.0009	0.0017	2	0.125	56	1	2	0.125
0.150	0.0009	0.0019	4	0.150	80	2	4	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.1	102.7	0.1	0.02854
0.5	247.9	0.4	0.09739
1.2	434.4	0.7	0.21806
2.3	651.1	1.1	0.39892
4.0	890.4	1.7	0.64624

**Rooftop Storage Summary**

Total Building Area (sq.m)		100	
Assume Available Roof Area (sq. 80%)		80	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		2	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		4	
Estimated 100 Year Drawdown Time (h)		0.3	

**From Watts Drain Catalogue**

Head (m)	L/s			
	Open	75%	50%	25% Closed
0.025	0.3155	0.3155	0.3155	<b>0.3155</b> 0.3155
0.050	0.6309	0.6309	0.6309	<b>0.6309</b> 0.6309
0.075	0.9464	0.8675	0.7886	<b>0.7098</b> 0.6309
0.100	1.2618	1.1041	0.9464	<b>0.7886</b> 0.6309
0.125	1.5773	1.3407	1.1041	<b>0.8675</b> 0.6309
0.150	1.8927	1.5773	1.2618	<b>0.9464</b> 0.6309

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.001	0.002	-
Depth (m)	0.073	0.118	0.150
Volume (cu.m)	0.5	2.0	4.0
Drain time (hrs)	0.1	0.3	

# Stormwater Management Calculations

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court**  
**Modified Rational Method Calculatons for Storage**

100 yr Intensity City of Ottawa	$I = a/(t + b)^c$		a =	1735.688	t (min)	I (mm/hr)
			b =	6.014	10	178.56
			c =	0.820	20	119.95
					30	91.87
				40	75.15	
				50	63.95	
				60	55.89	
				70	49.79	
				80	44.99	
				90	41.11	
				100	37.90	
				110	35.20	
				120	32.89	

### 100 YEAR Modified Rational Method for Entire Site

**Subdrainage Area:** ROOF12 Roof  
**Area (ha):** 0.01 Maximum Storage Depth: 150 mm  
**C:** 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m <sup>3</sup> )	Depth (mm)	
10	178.56	4.96	1.69	3.28	1.97	117.3	0.00
20	119.95	3.33	1.69	1.65	<b>1.98</b>	117.5	0.00
30	91.87	2.55	1.64	0.91	1.64	110.1	0.00
40	75.15	2.09	1.58	0.51	1.22	100.7	0.00
50	63.95	1.78	1.50	0.28	0.84	87.4	0.00
60	55.89	1.55	1.42	0.14	0.49	74.6	0.00
70	49.79	1.38	1.32	0.07	0.28	59.0	0.00
80	44.99	1.25	1.22	0.03	0.14	48.4	0.00
90	41.11	1.14	1.12	0.02	0.12	44.4	0.00
100	37.90	1.05	1.04	0.02	0.10	41.1	0.00
110	35.20	0.98	0.97	0.01	0.09	38.3	0.00
120	32.89	0.91	0.90	0.01	0.07	35.8	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	117.51	0.12	1.69	1.98	4.00	0.00

**Subdrainage Area:** ROOF11 Roof  
**Area (ha):** 0.01 Maximum Storage Depth: 150 mm  
**C:** 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m <sup>3</sup> )	Depth (mm)	
10	178.56	5.26	1.69	3.57	2.14	118.5	0.00
20	119.95	3.53	1.70	1.83	<b>2.20</b>	119.7	0.00
30	91.87	2.71	1.66	1.05	1.88	113.1	0.00
40	75.15	2.21	1.60	0.61	1.46	104.3	0.00
50	63.95	1.88	1.53	0.35	1.05	93.0	0.00
60	55.89	1.65	1.45	0.19	0.69	80.6	0.00
70	49.79	1.47	1.37	0.10	0.41	67.0	0.00
80	44.99	1.33	1.28	0.04	0.21	53.3	0.00
90	41.11	1.21	1.19	0.03	0.14	47.0	0.00
100	37.90	1.12	1.10	0.02	0.12	43.5	0.00
110	35.20	1.04	1.02	0.02	0.10	40.5	0.00
120	32.89	0.97	0.96	0.01	0.09	37.9	0.00

# Stormwater Management Calculations

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court**  
**Modified Rational Method Calculatons for Storage**

Storage: Roof Storage								
	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check		
100-year Water Level	119.70	0.12	1.70	2.20	4.24	0.00		
<hr/>								
<b>Subdrainage Area:</b> ROOF10					Roof			
<b>Area (ha):</b> 0.28		Maximum Storage Depth:			150 mm			
<b>C:</b> 1.00								
	<b>tc (min)</b>	<b>I (100 yr) (mm/hr)</b>	<b>Qactual (L/s)</b>	<b>Qrelease (L/s)</b>	<b>Qstored (L/s)</b>	<b>Vstored (m<sup>3</sup>)</b>	<b>Depth (mm)</b>	
	10	178.56	139.49	18.48	121.01	72.60	129.0	0.00
	20	119.95	93.70	19.07	74.63	89.55	137.9	0.00
	30	91.87	71.77	19.25	52.52	94.53	140.6	0.00
	40	75.15	58.70	19.25	39.45	<b>94.68</b>	140.6	0.00
	50	63.95	49.96	19.17	30.79	92.36	139.4	0.00
	60	55.89	43.66	19.04	24.62	88.64	137.5	0.00
	70	49.79	38.89	18.88	20.01	84.05	135.0	0.00
	80	44.99	35.15	18.70	16.44	78.93	132.3	0.00
	90	41.11	32.12	18.51	13.60	73.46	129.4	0.00
	100	37.90	29.61	18.31	11.30	67.78	126.4	0.00
	110	35.20	27.50	18.07	9.43	62.23	122.8	0.00
	120	32.89	25.70	17.79	7.90	56.91	118.6	0.00
Storage: Roof Storage								
	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check		
100-year Water Level	140.64	0.14	19.25	94.68	112.40	0.00		
<hr/>								
<b>Subdrainage Area:</b> ROOF9					Roof			
<b>Area (ha):</b> 0.01		Maximum Storage Depth:			150 mm			
<b>C:</b> 1.00								
	<b>tc (min)</b>	<b>I (100 yr) (mm/hr)</b>	<b>Qactual (L/s)</b>	<b>Qrelease (L/s)</b>	<b>Qstored (L/s)</b>	<b>Vstored (m<sup>3</sup>)</b>	<b>Depth (mm)</b>	
	10	178.56	2.63	1.58	1.05	<b>0.63</b>	100.1	0.00
	20	119.95	1.77	1.46	0.30	0.37	81.9	0.00
	30	91.87	1.35	1.29	0.06	0.11	54.6	0.00
	40	75.15	1.11	1.08	0.02	0.06	42.9	0.00
	50	63.95	0.94	0.93	0.01	0.04	36.8	0.00
	60	55.89	0.82	0.82	0.01	0.03	32.3	0.00
	70	49.79	0.73	0.73	0.00	0.02	28.9	0.00
	80	44.99	0.66	0.66	0.00	0.01	26.2	0.00
	90	41.11	0.61	0.60	0.00	0.01	23.9	0.00
	100	37.90	0.56	0.56	0.00	0.01	22.1	0.00
	110	35.20	0.52	0.52	0.00	0.01	20.5	0.00
	120	32.89	0.48	0.48	0.00	0.01	19.2	0.00
Storage: Roof Storage								
	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check		
100-year Water Level	100.15	0.10	1.58	0.63	2.12	0.00		
<hr/>								
<b>Subdrainage Area:</b> ROOF8					Roof			
<b>Area (ha):</b> 0.01		Maximum Storage Depth:			150 mm			
<b>C:</b> 1.00								
	<b>tc (min)</b>	<b>I (100 yr) (mm/hr)</b>	<b>Qactual (L/s)</b>	<b>Qrelease (L/s)</b>	<b>Qstored (L/s)</b>	<b>Vstored (m<sup>3</sup>)</b>	<b>Depth (mm)</b>	

# Stormwater Management Calculations

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court**  
**Modified Rational Method Calculatons for Storage**

	(min)	(mm/hr)	(L/s)	(L/s)	(L/s)	(m <sup>3</sup> )	(mm)	
	10	178.56	4.82	1.68	3.13	<b>1.88</b>	116.7	0.00
	20	119.95	3.23	1.68	1.55	1.87	116.3	0.00
	30	91.87	2.48	1.63	0.85	1.52	108.5	0.00
	40	75.15	2.03	1.57	0.46	1.10	98.3	0.00
	50	63.95	1.72	1.48	0.25	0.74	84.5	0.00
	60	55.89	1.51	1.39	0.12	0.42	70.3	0.00
	70	49.79	1.34	1.29	0.05	0.21	54.9	0.00
	80	44.99	1.21	1.19	0.03	0.13	47.0	0.00
	90	41.11	1.11	1.09	0.02	0.11	43.1	0.00
	100	37.90	1.02	1.01	0.02	0.09	39.9	0.00
	110	35.20	0.95	0.94	0.01	0.08	37.1	0.00
	120	32.89	0.89	0.88	0.01	0.07	34.8	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	116.66	0.12	1.68	1.88	3.88	0.00

**Subdrainage Area:** ROOF6 and 7  
**Area (ha):** 0.10  
**C:** 1.00

Roof  
 Maximum Storage Depth: 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m <sup>3</sup> )	Depth (mm)	
10	178.56	47.41	7.84	39.57	23.74	127.5	0.00
20	119.95	31.85	8.16	23.69	28.42	134.8	0.00
30	91.87	24.39	8.21	16.18	<b>29.12</b>	135.9	0.00
40	75.15	19.95	8.15	11.80	28.32	134.7	0.00
50	63.95	16.98	8.05	8.93	26.79	132.3	0.00
60	55.89	14.84	7.92	6.92	24.91	129.4	0.00
70	49.79	13.22	7.78	5.44	22.84	126.1	0.00
80	44.99	11.94	7.60	4.34	20.85	122.1	0.00
90	41.11	10.91	7.40	3.51	18.96	117.7	0.00
100	37.90	10.06	7.21	2.85	17.09	113.4	0.00
110	35.20	9.35	7.03	2.32	15.28	109.2	0.00
120	32.89	8.73	6.85	1.88	13.56	105.2	0.00

Storage: Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	135.90	0.14	8.21	29.12	38.20	0.00

**Subdrainage Area:** ROOF5  
**Area (ha):** 0.11  
**C:** 1.00

Roof  
 Maximum Storage Depth: 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m <sup>3</sup> )	Depth (mm)	
10	178.56	54.50	6.54	47.96	28.78	129.5	0.00
20	119.95	36.61	7.00	29.61	35.54	138.7	0.00
30	91.87	28.04	7.14	20.90	37.62	141.5	0.00
40	75.15	22.94	7.16	15.78	<b>37.87</b>	141.8	0.00
50	63.95	19.52	7.11	12.41	37.23	141.0	0.00
60	55.89	17.06	7.04	10.02	36.08	139.4	0.00
70	49.79	15.20	6.94	8.26	34.68	137.5	0.00
80	44.99	13.73	6.83	6.90	33.12	135.4	0.00
90	41.11	12.55	6.72	5.83	31.48	133.2	0.00
100	37.90	11.57	6.61	4.96	29.76	130.9	0.00
110	35.20	10.75	6.49	4.26	28.09	128.6	0.00

# Stormwater Management Calculations

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court**  
**Modified Rational Method Calculatons for Storage**

	120	32.89	10.04	6.37	3.67	26.40	126.3	0.00
Storage:	Roof Storage							
	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check		
100-year Water Level	141.83	0.14	7.16	37.87	43.92	0.00		

**Subdrainage Area:** ROOF4 Roof  
**Area (ha):** 0.04 Maximum Storage Depth: 150 mm  
**C:** 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.56	17.47	3.15	14.32	8.59	126.9	0.00
20	119.95	11.74	3.15	8.59	10.31	134.1	0.00
30	91.87	8.99	3.15	5.84	<b>10.51</b>	135.0	0.00
40	75.15	7.35	3.15	4.20	10.09	133.2	0.00
50	63.95	6.26	3.15	3.11	9.32	130.0	0.00
60	55.89	5.47	3.15	2.32	8.35	125.9	0.00
70	49.79	4.87	3.15	1.72	7.23	119.2	0.00
80	44.99	4.40	3.15	1.25	6.01	111.6	0.00
90	41.11	4.02	3.15	0.87	4.71	103.4	0.00
100	37.90	3.71	3.15	0.56	3.35	91.5	0.00
110	35.20	3.44	3.15	0.29	1.95	76.9	0.00
120	32.89	3.22	3.15	0.07	0.50	48.6	-0.08

Storage: Roof Storage

	134.96	0.13	3.15	10.51	14.08	0.00	
100-year Water Level	134.96	0.13	3.15	10.51	14.08	0.00	

**Subdrainage Area:** ROOF3 Roof  
**Area (ha):** 0.11 Maximum Storage Depth: 150 mm  
**C:** 1.00

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m^3)	Depth (mm)	
10	178.56	54.50	6.54	47.96	28.78	129.5	0.00
20	119.95	36.61	7.00	29.61	35.54	138.7	0.00
30	91.87	28.04	7.14	20.90	37.62	141.5	0.00
40	75.15	22.94	7.16	15.78	<b>37.87</b>	141.8	0.00
50	63.95	19.52	7.11	12.41	37.23	141.0	0.00
60	55.89	17.06	7.04	10.02	36.08	139.4	0.00
70	49.79	15.20	6.94	8.26	34.68	137.5	0.00
80	44.99	13.73	6.83	6.90	33.12	135.4	0.00
90	41.11	12.55	6.72	5.83	31.48	133.2	0.00
100	37.90	11.57	6.61	4.96	29.76	130.9	0.00
110	35.20	10.75	6.49	4.26	28.09	128.6	0.00
120	32.89	10.04	6.37	3.67	26.40	126.3	0.00

Storage: Roof Storage

	141.83	0.14	7.16	37.87	43.92	0.00	
100-year Water Level	141.83	0.14	7.16	37.87	43.92	0.00	

**Subdrainage Area:** ROOF1 and 2 Roof  
**Area (ha):** 0.10 Maximum Storage Depth: 150 mm  
**C:** 1.00



# Stormwater Management Calculations

**Project #160401511, Wellings of Stittsville Phase 2, 20 Cedarow Court**  
**Modified Rational Method Calculatons for Storage**

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m <sup>3</sup> )	Depth (mm)	
10	178.56	47.36	7.84	39.52	23.71	127.5	0.00
20	119.95	31.81	8.16	23.65	28.38	134.8	0.00
30	91.87	24.36	8.21	16.16	<b>29.08</b>	135.9	0.00
40	75.15	19.93	8.15	11.78	28.27	134.6	0.00
50	63.95	16.96	8.05	8.91	26.73	132.2	0.00
60	55.89	14.82	7.92	6.90	24.85	129.3	0.00
70	49.79	13.20	7.78	5.42	22.78	126.1	0.00
80	44.99	11.93	7.60	4.33	20.79	122.0	0.00
90	41.11	10.90	7.40	3.50	18.92	117.7	0.00
100	37.90	10.05	7.21	2.84	17.05	113.3	0.00
110	35.20	9.34	7.03	2.31	15.22	109.1	0.00
120	32.89	8.72	6.85	1.87	13.49	105.1	0.00

Storage:    Roof Storage

	Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check
100-year Water Level	135.88	0.14	8.21	29.08	38.16	0.00



Adjustable Accutrol Weir

Tag: \_\_\_\_\_

Adjustable Flow Control  
for Roof Drains

**ADJUSTABLE ACCUTROL(for Large Sump Roof Drains only)**

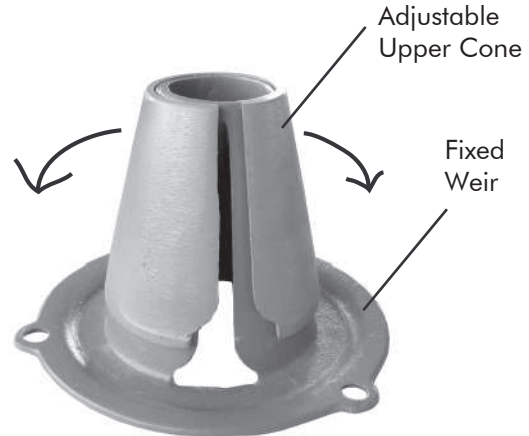
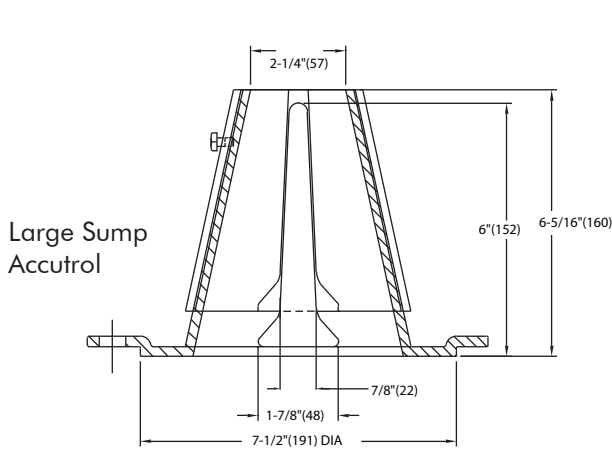
For more flexibility in controlling flow with heads deeper than 2", Watts Drainage offers the Adjustable Accutrol. The Adjustable Accutrol Weir is designed with a single parabolic opening that can be covered to restrict flow above 2" of head to less than 5 gpm per inch, up to 6" of head. To adjust the flow rate for depths over 2" of head, set the slot in the adjustable upper cone according to the flow rate required. Refer to Table 1 below.

Note: Flow rates are directly proportional to the amount of weir opening that is exposed.

**EXAMPLE:**

For example, if the adjustable upper cone is set to cover 1/2 of the weir opening, flow rates above 2" of head will be restricted to 2-1/2 gpm per inch of head.

Therefore, at 3" of head, the flow rate through the Accutrol Weir that has 1/2 the slot exposed will be:  
[ 5 gpm(per inch of head) x 2 inches of head ] + 2-1/2 gpm(for the third inch of head) = 12-1/2 gpm.



1/2 Weir Opening Exposed Shown Above

TABLE 1. Adjustable Accutrol Flow Rate Settings

Weir Opening Exposed	Head of Water					
	1"	2"	3"	4"	5"	6"
	Flow Rate (gallons per minute)					
Fully Exposed	5	10	15	20	25	30
3/4	5	10	13.75	17.5	21.25	25
1/2	5	10	12.5	15	17.5	20
1/4	5	10	11.25	12.5	13.75	15
Closed	5	10	10	10	10	10

Job Name \_\_\_\_\_ Contractor \_\_\_\_\_

Job Location \_\_\_\_\_ Contractor's P.O. No. \_\_\_\_\_

Engineer \_\_\_\_\_ Representative \_\_\_\_\_

WATTS Drainage reserves the right to modify or change product design or construction without prior notice and without incurring any obligation to make similar changes and modifications to products previously or subsequently sold. See your WATTS Drainage representative for any clarification. Dimensions are subject to manufacturing tolerances.



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**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**C.2 SAMPLE PCSWMM MODEL INPUT (12HR 100YR SCS)**

```

[TITLE]
;;Project Title/Notes

[OPTIONS]
;;Option          Value
FLOW_UNITS        LPS
INFILTRATION      HORTON
FLOW_ROUTING      DYNWAVE
LINK_OFFSETS      ELEVATION
MIN_SLOPE         0
ALLOW_PONDING     YES
SKIP_STEADY_STATE NO

START_DATE        07/23/2009
START_TIME        00:00:00
REPORT_START_DATE 07/23/2009
REPORT_START_TIME 00:00:00
END_DATE          07/24/2009
END_TIME          00:00:00
SWEEP_START       01/01
SWEEP_END         12/31
DRY_DAYS          0
REPORT_STEP       00:05:00
WET_STEP          00:05:00
DRY_STEP          00:05:00
ROUTING_STEP      1
RULE_STEP         00:00:00

INERTIAL_DAMPING  PARTIAL
NORMAL_FLOW_LIMITED BOTH
FORCE_MAIN_EQUATION H-W
VARIABLE_STEP     0
LENGTHENING_STEP 0
MIN_SURFAREA      0
MAX_TRIALS        8
HEAD_TOLERANCE    0.0015
SYS_FLOW_TOL      5
LAT_FLOW_TOL      5
MINIMUM_STEP      0.5

```

```

THREADS          4

```

```

[EVAPORATION]
;;Data Source    Parameters
;;-----
CONSTANT         0.0
DRY_ONLY         NO

```

```

[RAINGAGES]
;;Name           Format   Interval SCF   Source
;;-----
RG1              INTENSITY 0:15   1.0   TIMESERIES 100SCS

```

```

[SUBCATCHMENTS]
;;Name           Rain Gage      Outlet          Area    %Imperv  Width  %Slope  CurbLen  SnowPack
;;-----
EXT-1            RG1            CB507-S         0.068859 38.571  95     1.5     0
ROOF_10          RG1            ROOF-10-S      0.281012 100     136    1.5     0
ROOF_11          RG1            ROOF-11-S      0.010607 100     21     1.5     0
ROOF_12          RG1            ROOF-12-S      0.01253  100     15.6   1.5     0
ROOF_3           RG1            ROOF-3-S       0.109818 100     130    1.5     0
ROOF_4           RG1            ROOF-4-S       0.035229 100     46     1.5     0
ROOF_5           RG1            ROOF-5-S       0.109819 100     130    1.5     0
ROOF_8           RG1            ROOF-8-S       0.009743 100     21     1.5     0
ROOF_9           RG1            ROOF-9-S       0.005311 100     15     1.5     0
ROOF1_2          RG1            ROOF-1-2-S     0.0954   100     95     1.5     0
ROOF6_7          RG1            ROOF-6-7-S     0.0955   100     95     1.5     0

```

UGPK_1	RG1	TANKS	0.144022	77.143	115	2	0
UGPK_2	RG1	TANKS	0.152475	80	122	2	0
UGPK_3	RG1	TANKS	0.059673	58.571	60	2	0
UGPK_4	RG1	TANKS	0.119964	70	95	2	0
UGPK_5	RG1	TANKS	0.110163	70	85	2	0
UGPK_6	RG1	TANKS	0.021989	100	60	15	0
UGPK_7	RG1	TANKS	0.112091	78.571	78	2	0
UGPK_8	RG1	TANKS	0.061679	75.714	42	2	0
UGPK_9	RG1	TANKS	0.032467	100	42	2	0
UNC-1	RG1	OF1	0.078091	41.429	78	2	0
UNC-2	RG1	OF2	0.515043	8.571	25	1	0
UNC-3	RG1	OF3	0.069306	61.429	122	2	0
UNC-4	RG1	CB507-S	0.051524	37.143	90	2	0

[SUBAREAS]

;;Subcatchment	N-Imperv	N-Perv	S-Imperv	S-Perv	PctZero	RouteTo	PctRouted
EXT-1	0.013	0.2	1.57	4.67	0	PERVIOUS	100
ROOF_10	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF_11	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF_12	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF_3	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF_4	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF_5	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF_8	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100

ROOF_9	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF1_2	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ROOF6_7	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_1	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_2	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_3	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_4	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_5	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_6	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_7	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_8	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UGPK_9	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
UNC-1	0.013	0.2	1.57	4.67	0	PERVIOUS	100
UNC-2	0.013	0.2	1.57	4.67	0	PERVIOUS	100
UNC-3	0.013	0.2	1.57	4.67	0	PERVIOUS	100
UNC-4	0.013	0.2	1.57	4.67	0	PERVIOUS	100

[INFILTRATION]

;;Subcatchment	Param1	Param2	Param3	Param4	Param5
EXT-1	76.2	13.2	4.14	7	0
ROOF_10	76.2	13.2	4.14	7	0
ROOF_11	76.2	13.2	4.14	7	0
ROOF_12	76.2	13.2	4.14	7	0
ROOF_3	76.2	13.2	4.14	7	0
ROOF_4	76.2	13.2	4.14	7	0
ROOF_5	76.2	13.2	4.14	7	0
ROOF_8	76.2	13.2	4.14	7	0
ROOF_9	76.2	13.2	4.14	7	0
ROOF1_2	76.2	13.2	4.14	7	0
ROOF6_7	76.2	13.2	4.14	7	0
UGPK_1	76.2	13.2	4.14	7	0
UGPK_2	76.2	13.2	4.14	7	0
UGPK_3	76.2	13.2	4.14	7	0
UGPK_4	76.2	13.2	4.14	7	0
UGPK_5	76.2	13.2	4.14	7	0
UGPK_6	76.2	13.2	4.14	7	0
UGPK_7	76.2	13.2	4.14	7	0
UGPK_8	76.2	13.2	4.14	7	0

UGPK_9	76.2	13.2	4.14	7	0
UNC-1	76.2	13.2	4.14	7	0
UNC-2	76.2	13.2	4.14	7	0
UNC-3	76.2	13.2	4.14	7	0
UNC-4	76.2	13.2	4.14	7	0

[JUNCTIONS]

;;Name	Elevation	MaxDepth	InitDepth	SurDepth	Aponded
100	99.4	2.735	0	0	0

[OUTFALLS]

;;Name	Elevation	Type	Stage Data	Gated	Route To
HEADWALL	98.7	FREE		NO	
OF1	0	FREE		NO	
OF2	0	FREE		NO	
OF3	0	FREE		NO	
OF4	101.87	FIXED	102.17	NO	

[STORAGE]

;;Name	Elev.	MaxDepth	InitDepth	Shape	Curve Name/Params	N/A	Fevap
Psi	Ksat	IMD					
1000	99.92	4.18	0	FUNCTIONAL	1.13 0 0	0	0
CB507-S	102.56	2.23	0	FUNCTIONAL	0 0 0	0	0
ROOF-10-S	114	0.15	0	TABULAR	ROOF10	0	0
ROOF-11-S	114	0.15	0	TABULAR	ROOF11	0	0
ROOF-12-S	114	0.15	0	TABULAR	ROOF12	0	0
ROOF-1-2-S	114	0.15	0	TABULAR	ROOF1and2	0	0
ROOF-3-S	114	0.15	0	TABULAR	ROOF3	0	0
ROOF-4-S	114	0.15	0	TABULAR	ROOF4	0	0
ROOF-5-S	114	0.15	0	TABULAR	ROOF5	0	0
ROOF-6-7-S	114	0.15	0	TABULAR	ROOF6and7	0	0
ROOF-8-S	114	0.15	0	TABULAR	ROOF8	0	0
ROOF-9-S	114	0.15	0	TABULAR	ROOF9	0	0
TANKS	99.7	3.61	0	FUNCTIONAL	0 0 222	0	0

[CONDUITS]

;;Name	From Node	To Node	Length	Roughness	InOffset	OutOffset	InitFlow
C1	CB507-S	OF4	21.3	0.013	102.56	102.45	0
C2	1000	100	20.8	0.013	99.87	99.83	0
Pipe_13	100	HEADWALL	11.135	0.013	99.548	99.52	0

[ORIFICES]

;;Name	From Node	To Node	Type	Offset	Qcoeff	Gated	CloseTime
CISTERN-0	TANKS	1000	SIDE	99.95	0.61	NO	0

[WEIRS]

;;Name	From Node	To Node	Type	CrestHt	Qcoeff	Gated	EndCon
EndCoeff	Surcharge	RoadWidth	RoadSurf	Coeff.	Curve		
W1	TANKS	1000	TRANSVERSE	102.25	1.67	NO	0
YES							

[OUTLETS]

;;Name	From Node	To Node	Offset	Type	QTable/Qcoeff	Qexpon
ROOF10-0	ROOF-10-S	TANKS	114	TABULAR/HEAD	ROOF-10-0	
NO						
ROOF11-0	ROOF-11-S	TANKS	114	TABULAR/HEAD	ROOF-11-0	
NO						
ROOF12-0	ROOF-12-S	TANKS	114	TABULAR/HEAD	ROOF-12-0	
NO						
ROOF1-2-0	ROOF-1-2-S	TANKS	114	TABULAR/HEAD	ROOF-1-2-0	
NO						
ROOF3-0	ROOF-3-S	TANKS	114	TABULAR/HEAD	ROOF-3-0	

```

NO
ROOF4-0      ROOF-4-S      TANKS      114      TABULAR/HEAD      ROOF-4-0
NO
ROOF5-0      ROOF-5-S      TANKS      114      TABULAR/HEAD      ROOF-5-0
NO
ROOF6-7-0    ROOF-6-7-S    TANKS      114      TABULAR/HEAD      ROOF-6-7-0
NO
ROOF8-0      ROOF-8-S      TANKS      114      TABULAR/HEAD      ROOF-8-0
NO
ROOF9-0      ROOF-9-S      TANKS      114      TABULAR/HEAD      ROOF-9-0
NO

```

```

[XSECTIONS]
;;Link      Shape      Geom1      Geom2      Geom3      Geom4      Barrels      Culvert
;;-----
C1          CIRCULAR    0.25      0          0          0          1
C2          CIRCULAR    0.45      0          0          0          1
Pipe_13     CIRCULAR    0.9       0          0          0          1
CISTERN-0   CIRCULAR    0.075     0          0          0
W1          RECT_OPEN   1         0.5        0          0

```

```

[TRANSECTS]
;;Transect Data in HEC-2 format
;
NC 0.013    0.013    0.013
X1 Overland 5         0.15     6.85     0.0       0.0       0.0       0.0       0.0
GR 0.15     0         0         0.15     0         6.85     0.15     7         0.15     7
;
[LE: 0][RE: 7]
NC 0.013    0.013    0.013
X1 Overland(orig) 4         0.15     6.85     0.0       0.0       0.0       0.0       0.0
GR 0.15     0         0         0.15     0         6.85     0.15     7

```

```

[LOSSES]
;;Link      Kentry      Kexit      Kavg      Flap Gate      Seepage
;;-----
C2          0           0.14       0         NO             0

```

```
[INFLOWS]
```

```

;;Node      Constituent      Time Series      Type      Mfactor      Sfactor      Baseline Pattern
;;-----
100         FLOW             100yrHydrograph  FLOW      1.0          1           0

```

```

[CURVES]
;;Name      Type      X-Value      Y-Value
;;-----
BIOSWALE_BASEFLOW Rating    0           0
BIOSWALE_BASEFLOW      0.01      0.3
BIOSWALE_BASEFLOW      10        0.3

ROOF-10-0      Rating    0           0
ROOF-10-0      0.025    6.624
ROOF-10-0      0.05     13.249
ROOF-10-0      0.075    14.905
ROOF-10-0      0.1      16.561
ROOF-10-0      0.125    18.217
ROOF-10-0      0.15     19.873

ROOF-11-0      Rating    0           0
ROOF-11-0      0.025    0.631
ROOF-11-0      0.05     1.262
ROOF-11-0      0.075    1.42
ROOF-11-0      0.1      1.577
ROOF-11-0      0.125    1.735
ROOF-11-0      0.15     1.893

ROOF-12-0      Rating    0           0
ROOF-12-0      0.025    0.631
ROOF-12-0      0.05     1.262
ROOF-12-0      0.075    1.42
ROOF-12-0      0.1      1.577
ROOF-12-0      0.125    1.735
ROOF-12-0      0.15     1.893

ROOF-1-2-0      Rating    0           0
ROOF-1-2-0      0.025    2.21
ROOF-1-2-0      0.05     4.42
ROOF-1-2-0      0.075    5.52

```

ROOF-1-2-0		0.1	6.62
ROOF-1-2-0		0.125	7.73
ROOF-1-2-0		0.15	8.83
ROOF-3-0	Rating	0	0
ROOF-3-0		0.025	1.26
ROOF-3-0		0.05	2.52
ROOF-3-0		0.075	3.79
ROOF-3-0		0.1	5.05
ROOF-3-0		0.125	6.31
ROOF-3-0		0.15	7.57
ROOF-4-0	Rating	0	0
ROOF-4-0		0.025	1.58
ROOF-4-0		0.05	3.15
ROOF-4-0		0.075	3.15
ROOF-4-0		0.1	3.15
ROOF-4-0		0.125	3.15
ROOF-4-0		0.15	3.15
ROOF-5-0	Rating	0	0
ROOF-5-0		0.025	1.26
ROOF-5-0		0.05	2.52
ROOF-5-0		0.075	3.79
ROOF-5-0		0.1	5.05
ROOF-5-0		0.125	6.31
ROOF-5-0		0.15	7.57
ROOF-6-7-0	Rating	0	0
ROOF-6-7-0		0.025	2.21
ROOF-6-7-0		0.05	4.42
ROOF-6-7-0		0.075	5.52
ROOF-6-7-0		0.1	6.62
ROOF-6-7-0		0.125	7.73
ROOF-6-7-0		0.15	8.83
ROOF-8-0	Rating	0	0
ROOF-8-0		0.025	0.63
ROOF-8-0		0.05	1.26

ROOF-8-0		0.075	1.42
ROOF-8-0		0.1	1.58
ROOF-8-0		0.125	1.73
ROOF-8-0		0.15	1.89
ROOF-9-0	Rating	0	0
ROOF-9-0		0.025	0.631
ROOF-9-0		0.05	1.262
ROOF-9-0		0.075	1.42
ROOF-9-0		0.1	1.577
ROOF-9-0		0.125	1.735
ROOF-9-0		0.15	1.893
ROOF10	Storage	0	0
ROOF10		0.025	62.44
ROOF10		0.05	249.78
ROOF10		0.075	562
ROOF10		0.1	999.11
ROOF10		0.125	1561.11
ROOF10		0.15	2248
ROOF11	Storage	0	0
ROOF11		0.025	2.36
ROOF11		0.05	9.42
ROOF11		0.075	21.2
ROOF11		0.1	37.69
ROOF11		0.125	58.89
ROOF11		0.15	84.8
ROOF12	Storage	0	0
ROOF12		0.025	2.22
ROOF12		0.05	8.89
ROOF12		0.075	20
ROOF12		0.1	35.56
ROOF12		0.125	55.56
ROOF12		0.15	80
ROOF1and2	Storage	0	0
ROOF1and2		0.025	21.2



ROOF1and2		0.05	84.8
ROOF1and2		0.075	190.8
ROOF1and2		0.1	339.2
ROOF1and2		0.125	530
ROOF1and2		0.15	763.2
ROOF3	Storage	0	0
ROOF3		0.025	24.4
ROOF3		0.05	97.6
ROOF3		0.075	219.6
ROOF3		0.1	390.4
ROOF3		0.125	610
ROOF3		0.15	878.4
ROOF4	Storage	0	0
ROOF4		0.025	7.822222222
ROOF4		0.05	31.28888889
ROOF4		0.075	70.4
ROOF4		0.1	125.1555556
ROOF4		0.125	195.5555556
ROOF4		0.15	281.6
ROOF5	Storage	0	0
ROOF5		0.025	24.4
ROOF5		0.05	97.6
ROOF5		0.075	219.6
ROOF5		0.1	390.4
ROOF5		0.125	610
ROOF5		0.15	878.4
ROOF6and7	Storage	0	0
ROOF6and7		0.025	21.22
ROOF6and7		0.05	84.89
ROOF6and7		0.075	191
ROOF6and7		0.1	339.56
ROOF6and7		0.125	530.56
ROOF6and7		0.15	764
ROOF8	Storage	0	0

ROOF8		0.025	2.16
ROOF8		0.05	8.62
ROOF8		0.075	19.4
ROOF8		0.1	34.49
ROOF8		0.125	53.89
ROOF8		0.15	77.6
ROOF9	Storage	0	0
ROOF9		0.025	1.18
ROOF9		0.05	4.71
ROOF9		0.075	10.6
ROOF9		0.1	18.84
ROOF9		0.125	29.44
ROOF9		0.15	42.4
TANK	Storage	0	560.7
TANK		0.026	560.7
TANK		0.051	560.7
TANK		0.077	560.7
TANK		0.102	559.44
TANK		0.127	559.44
TANK		0.153	558.18
TANK		0.178	556.92
TANK		0.204	555.66
TANK		0.229	554.4
TANK		0.254	551.88
TANK		0.28	549.36
TANK		0.305	546.84
TANK		0.331	543.06
TANK		0.356	539.28
TANK		0.381	534.24
TANK		0.407	527.94
TANK		0.432	521.64
TANK		0.458	514.08
TANK		0.483	505.26
TANK		0.508	495.18
TANK		0.534	483.84
TANK		0.559	478.8
TANK		0.585	464.94

TANK	0.61	449.82
TANK	0.635	434.7
TANK	0.661	419.58
TANK	0.686	403.2
TANK	0.712	383.04
TANK	0.737	360.36
TANK	0.762	347.76
TANK	0.796	335.16
TANK	0.813	320.04
TANK	0.839	304.92
TANK	0.864	289.8
TANK	0.889	272.16
TANK	0.915	258.3
TANK	0.94	244.44
TANK	0.965	233.1
TANK	0.991	221.76
TANK	1.016	211.68
TANK	1.041	201.6
TANK	1.067	192.78
TANK	1.092	185.22
TANK	1.118	180.18
TANK	1.143	176.4
TANK	1.168	172.62
TANK	1.194	170.1
TANK	1.219	167.58
TANK	1.245	165.06
TANK	1.27	163.8
TANK	1.295	162.54
TANK	1.321	162.54
TANK	1.346	162.54
TANK	1.372	161.28
TANK	1.397	161.28
TANK	1.422	161.28
TANK	1.448	161.28
TANK	1.473	161.28
TANK	1.499	161.28
TANK	1.524	161.28
TANK	1.549	161.28
TANK	1.575	161.28

TANK	1.6	161.28
TANK	1.626	161.28
TANK	1.651	161.28
TANK	1.676	161.28
TANK	1.702	161.28
TANK	1.727	161.28
TANK	1.753	161.28
TANK	1.778	161.28
TANK	1.803	161.28
TANK	1.829	161.28
TANK	1.83	0
TANK	5	0

```

[TIMESERIES]
;;Name      Date      Time      Value
;;-----
;MTO Distribution, 15min intervals
002SCS      0:00      0
002SCS      0:15      1.08
002SCS      0:30      1.08
002SCS      0:45      1.08
002SCS      1:00      1.08
002SCS      1:15      1.08
002SCS      1:30      1.08
002SCS      1:45      1.08
002SCS      2:00      1.296
002SCS      2:15      1.296
002SCS      2:30      1.296
002SCS      2:45      1.296
002SCS      3:00      1.728
002SCS      3:15      1.728
002SCS      3:30      1.728
002SCS      3:45      1.728
002SCS      4:00      2.592
002SCS      4:15      2.592
002SCS      4:30      3.456
002SCS      4:45      3.456
002SCS      5:00      5.184
002SCS      5:15      5.184

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002SCS	5:30	20.736
002SCS	5:45	57.024
002SCS	6:00	7.776
002SCS	6:15	7.776
002SCS	6:30	3.456
002SCS	6:45	3.456
002SCS	7:00	2.592
002SCS	7:15	2.592
002SCS	7:30	2.592
002SCS	7:45	2.592
002SCS	8:00	1.512
002SCS	8:15	1.512
002SCS	8:30	1.512
002SCS	8:45	1.512
002SCS	9:00	1.512
002SCS	9:15	1.512
002SCS	9:30	1.512
002SCS	9:45	1.512
002SCS	10:00	0.864
002SCS	10:15	0.864
002SCS	10:30	0.864
002SCS	10:45	0.864
002SCS	11:00	0.864
002SCS	11:15	0.864
002SCS	11:30	0.864
002SCS	11:45	0.864
002SCS	12:00	0
005SCS	0:00:00	0
005SCS	0:15:00	1.44
005SCS	0:30:00	1.44
005SCS	0:45:00	1.44
005SCS	1:00:00	1.44
005SCS	1:15:00	1.44
005SCS	1:30:00	1.44
005SCS	1:45:00	1.44
005SCS	2:00:00	1.728
005SCS	2:15:00	1.728
005SCS	2:30:00	1.728

005SCS	2:45:00	1.728
005SCS	3:00:00	2.304
005SCS	3:15:00	2.304
005SCS	3:30:00	2.304
005SCS	3:45:00	2.304
005SCS	4:00:00	3.456
005SCS	4:15:00	3.456
005SCS	4:30:00	4.608
005SCS	4:45:00	4.608
005SCS	5:00:00	6.912
005SCS	5:15:00	6.912
005SCS	5:30:00	27.648
005SCS	5:45:00	76.032
005SCS	6:00:00	10.368
005SCS	6:15:00	10.368
005SCS	6:30:00	4.608
005SCS	6:45:00	4.608
005SCS	7:00:00	3.456
005SCS	7:15:00	3.456
005SCS	7:30:00	3.456
005SCS	7:45:00	3.456
005SCS	8:00:00	2.016
005SCS	8:15:00	2.016
005SCS	8:30:00	2.016
005SCS	8:45:00	2.016
005SCS	9:00:00	2.016
005SCS	9:15:00	2.016
005SCS	9:30:00	2.016
005SCS	9:45:00	2.016
005SCS	10:00:00	1.152
005SCS	10:15:00	1.152
005SCS	10:30:00	1.152
005SCS	10:45:00	1.152
005SCS	11:00:00	1.152
005SCS	11:15:00	1.152
005SCS	11:30:00	1.152
005SCS	11:45:00	1.152
005SCS	12:00:00	0

010SCS	0:00:00	0
010SCS	0:15:00	1.68
010SCS	0:30:00	1.68
010SCS	0:45:00	1.68
010SCS	1:00:00	1.68
010SCS	1:15:00	1.68
010SCS	1:30:00	1.68
010SCS	1:45:00	1.68
010SCS	2:00:00	2.02
010SCS	2:15:00	2.02
010SCS	2:30:00	2.02
010SCS	2:45:00	2.02
010SCS	3:00:00	2.69
010SCS	3:15:00	2.69
010SCS	3:30:00	2.69
010SCS	3:45:00	2.69
010SCS	4:00:00	4.03
010SCS	4:15:00	4.03
010SCS	4:30:00	5.38
010SCS	4:45:00	5.38
010SCS	5:00:00	8.06
010SCS	5:15:00	8.06
010SCS	5:30:00	32.26
010SCS	5:45:00	88.7
010SCS	6:00:00	12.1
010SCS	6:15:00	12.1
010SCS	6:30:00	5.38
010SCS	6:45:00	5.38
010SCS	7:00:00	4.03
010SCS	7:15:00	4.03
010SCS	7:30:00	4.03
010SCS	7:45:00	4.03
010SCS	8:00:00	2.35
010SCS	8:15:00	2.35
010SCS	8:30:00	2.35
010SCS	8:45:00	2.35
010SCS	9:00:00	2.35
010SCS	9:15:00	2.35
010SCS	9:30:00	2.35

010SCS	9:45:00	2.35
010SCS	10:00:00	1.34
010SCS	10:15:00	1.34
010SCS	10:30:00	1.34
010SCS	10:45:00	1.34
010SCS	11:00:00	1.34
010SCS	11:15:00	1.34
010SCS	11:30:00	1.34
010SCS	11:45:00	1.34
010SCS	12:00:00	0
025SCS	0:00:00	0
025SCS	0:15:00	1.98
025SCS	0:30:00	1.98
025SCS	0:45:00	1.98
025SCS	1:00:00	1.98
025SCS	1:15:00	1.98
025SCS	1:30:00	1.98
025SCS	1:45:00	1.98
025SCS	2:00:00	2.376
025SCS	2:15:00	2.376
025SCS	2:30:00	2.376
025SCS	2:45:00	2.376
025SCS	3:00:00	3.168
025SCS	3:15:00	3.168
025SCS	3:30:00	3.168
025SCS	3:45:00	3.168
025SCS	4:00:00	4.752
025SCS	4:15:00	4.752
025SCS	4:30:00	6.336
025SCS	4:45:00	6.336
025SCS	5:00:00	9.504
025SCS	5:15:00	9.504
025SCS	5:30:00	38.016
025SCS	5:45:00	104.544
025SCS	6:00:00	14.256
025SCS	6:15:00	14.256
025SCS	6:30:00	6.336
025SCS	6:45:00	6.336

025SCS	7:00:00	4.752
025SCS	7:15:00	4.752
025SCS	7:30:00	4.752
025SCS	7:45:00	4.752
025SCS	8:00:00	2.772
025SCS	8:15:00	2.772
025SCS	8:30:00	2.772
025SCS	8:45:00	2.772
025SCS	9:00:00	2.772
025SCS	9:15:00	2.772
025SCS	9:30:00	2.772
025SCS	9:45:00	2.772
025SCS	10:00:00	1.584
025SCS	10:15:00	1.584
025SCS	10:30:00	1.584
025SCS	10:45:00	1.584
025SCS	11:00:00	1.584
025SCS	11:15:00	1.584
025SCS	11:30:00	1.584
025SCS	11:45:00	1.584
025SCS	12:00:00	0
050SCS	0:00:00	0
050SCS	0:15:00	2.19
050SCS	0:30:00	2.19
050SCS	0:45:00	2.19
050SCS	1:00:00	2.19
050SCS	1:15:00	2.19
050SCS	1:30:00	2.19
050SCS	1:45:00	2.19
050SCS	2:00:00	2.628
050SCS	2:15:00	2.628
050SCS	2:30:00	2.628
050SCS	2:45:00	2.628
050SCS	3:00:00	3.504
050SCS	3:15:00	3.504
050SCS	3:30:00	3.504
050SCS	3:45:00	3.504
050SCS	4:00:00	5.256

050SCS	4:15:00	5.256
050SCS	4:30:00	7.008
050SCS	4:45:00	7.008
050SCS	5:00:00	10.512
050SCS	5:15:00	10.512
050SCS	5:30:00	42.048
050SCS	5:45:00	115.632
050SCS	6:00:00	15.768
050SCS	6:15:00	15.768
050SCS	6:30:00	7.008
050SCS	6:45:00	7.008
050SCS	7:00:00	5.256
050SCS	7:15:00	5.256
050SCS	7:30:00	5.256
050SCS	7:45:00	5.256
050SCS	8:00:00	3.066
050SCS	8:15:00	3.066
050SCS	8:30:00	3.066
050SCS	8:45:00	3.066
050SCS	9:00:00	3.066
050SCS	9:15:00	3.066
050SCS	9:30:00	3.066
050SCS	9:45:00	3.066
050SCS	10:00:00	1.752
050SCS	10:15:00	1.752
050SCS	10:30:00	1.752
050SCS	10:45:00	1.752
050SCS	11:00:00	1.752
050SCS	11:15:00	1.752
050SCS	11:30:00	1.752
050SCS	11:45:00	1.752
050SCS	12:00:00	0

;MTO Distribution, 15min intervals

100SCS	0:00	0
100SCS	0:15	2.4
100SCS	0:30	2.4
100SCS	0:45	2.4
100SCS	1:00	2.4

100SCS	1:15	2.4
100SCS	1:30	2.4
100SCS	1:45	2.4
100SCS	2:00	2.4
100SCS	2:15	2.88
100SCS	2:30	2.88
100SCS	2:45	2.88
100SCS	3:00	2.88
100SCS	3:15	3.84
100SCS	3:30	3.84
100SCS	3:45	3.84
100SCS	4:00	3.84
100SCS	4:15	5.76
100SCS	4:30	5.76
100SCS	4:45	7.68
100SCS	5:00	7.68
100SCS	5:15	11.52
100SCS	5:30	11.52
100SCS	5:45	46.08
100SCS	6:00	126.72
100SCS	6:15	17.28
100SCS	6:30	17.28
100SCS	6:45	7.68
100SCS	7:00	7.68
100SCS	7:15	5.76
100SCS	7:30	5.76
100SCS	7:45	5.76
100SCS	8:00	5.76
100SCS	8:15	3.36
100SCS	8:30	3.36
100SCS	8:45	3.36
100SCS	9:00	3.36
100SCS	9:15	3.36
100SCS	9:30	3.36
100SCS	9:45	3.36
100SCS	10:00	3.36
100SCS	10:15	1.92
100SCS	10:30	1.92
100SCS	10:45	1.92

100SCS	11:00	1.92
100SCS	11:15	1.92
100SCS	11:30	1.92
100SCS	11:45	1.92
100SCS	12:00	0
100yrHydrograph	0:05	0
100yrHydrograph	0:10	0
100yrHydrograph	0:15	0
100yrHydrograph	0:20	0
100yrHydrograph	0:25	0
100yrHydrograph	0:30	0
100yrHydrograph	0:35	0
100yrHydrograph	0:40	0
100yrHydrograph	0:45	0
100yrHydrograph	0:50	0
100yrHydrograph	0:55	0
100yrHydrograph	1:00	0
100yrHydrograph	1:05	0
100yrHydrograph	1:10	0
100yrHydrograph	1:15	0
100yrHydrograph	1:20	0
100yrHydrograph	1:25	0
100yrHydrograph	1:30	0
100yrHydrograph	1:35	0
100yrHydrograph	1:40	0
100yrHydrograph	1:45	0
100yrHydrograph	1:50	0
100yrHydrograph	1:55	0
100yrHydrograph	2:00	0
100yrHydrograph	2:05	0.03368589
100yrHydrograph	2:10	0.400265
100yrHydrograph	2:15	0.6780789
100yrHydrograph	2:20	0.8096212
100yrHydrograph	2:25	0.9188437
100yrHydrograph	2:30	1.041047
100yrHydrograph	2:35	1.160273
100yrHydrograph	2:40	1.279933
100yrHydrograph	2:45	1.400491

100yrHydrograph	2:50	1.521803
100yrHydrograph	2:55	1.6431
100yrHydrograph	3:00	1.770213
100yrHydrograph	3:05	1.89238
100yrHydrograph	3:10	2.011746
100yrHydrograph	3:15	2.129715
100yrHydrograph	3:20	2.24681
100yrHydrograph	3:25	2.372673
100yrHydrograph	3:30	2.52147
100yrHydrograph	3:35	2.690327
100yrHydrograph	3:40	2.870473
100yrHydrograph	3:45	3.049115
100yrHydrograph	3:50	3.225464
100yrHydrograph	3:55	3.398892
100yrHydrograph	4:00	3.569112
100yrHydrograph	4:05	3.736014
100yrHydrograph	4:10	3.899172
100yrHydrograph	4:15	4.062318
100yrHydrograph	4:20	4.221156
100yrHydrograph	4:25	4.414243
100yrHydrograph	4:30	4.674353
100yrHydrograph	4:35	4.971942
100yrHydrograph	4:40	5.279111
100yrHydrograph	4:45	6.154143
100yrHydrograph	4:50	6.507444
100yrHydrograph	4:55	6.698976
100yrHydrograph	5:00	6.914319
100yrHydrograph	5:05	7.139013
100yrHydrograph	5:10	7.358335
100yrHydrograph	5:15	7.568453
100yrHydrograph	5:20	7.776987
100yrHydrograph	5:25	8.043731
100yrHydrograph	5:30	8.375088
100yrHydrograph	5:35	8.720076
100yrHydrograph	5:40	9.065854
100yrHydrograph	5:45	9.411835
100yrHydrograph	5:50	9.817601
100yrHydrograph	5:55	10.7888
100yrHydrograph	6:00	12.18063

100yrHydrograph	6:05	13.55768
100yrHydrograph	6:10	15.79406
100yrHydrograph	6:15	18.70004
100yrHydrograph	6:20	42.87194
100yrHydrograph	6:25	89.41938
100yrHydrograph	6:30	110.0801
100yrHydrograph	6:35	120.6727
100yrHydrograph	6:40	126.8955
100yrHydrograph	6:45	131.1839
100yrHydrograph	6:50	135.1019
100yrHydrograph	6:55	136.1944
100yrHydrograph	7:00	134.3278
100yrHydrograph	7:05	133.2315
100yrHydrograph	7:10	131.9975
100yrHydrograph	7:15	130.5499
100yrHydrograph	7:20	128.9494
100yrHydrograph	7:25	127.0917
100yrHydrograph	7:30	124.968
100yrHydrograph	7:35	122.8077
100yrHydrograph	7:40	120.8129
100yrHydrograph	7:45	118.6745
100yrHydrograph	7:50	116.2747
100yrHydrograph	7:55	113.7336
100yrHydrograph	8:00	111.03
100yrHydrograph	8:05	107.4665
100yrHydrograph	8:10	103.368
100yrHydrograph	8:15	99.26289
100yrHydrograph	8:20	95.15655
100yrHydrograph	8:25	90.6512
100yrHydrograph	8:30	85.96979
100yrHydrograph	8:35	81.40397
100yrHydrograph	8:40	77.20897
100yrHydrograph	8:45	73.056
100yrHydrograph	8:50	68.54881
100yrHydrograph	8:55	63.82666
100yrHydrograph	9:00	59.70413
100yrHydrograph	9:05	56.16789
100yrHydrograph	9:10	53.06845
100yrHydrograph	9:15	50.40911

100yrHydrograph	9:20	48.21593
100yrHydrograph	9:25	46.66951
100yrHydrograph	9:30	45.38166
100yrHydrograph	9:35	44.14623
100yrHydrograph	9:40	42.96396
100yrHydrograph	9:45	41.84916
100yrHydrograph	9:50	40.78232
100yrHydrograph	9:55	39.74205
100yrHydrograph	10:00	38.7661
100yrHydrograph	10:05	37.83093
100yrHydrograph	10:10	36.92952
100yrHydrograph	10:15	36.06311
100yrHydrograph	10:20	35.16625
100yrHydrograph	10:25	33.88804
100yrHydrograph	10:30	32.4535
100yrHydrograph	10:35	31.05881
100yrHydrograph	10:40	29.78304
100yrHydrograph	10:45	28.63077
100yrHydrograph	10:50	27.58944
100yrHydrograph	10:55	26.653
100yrHydrograph	11:00	25.66802
100yrHydrograph	11:05	24.91771
100yrHydrograph	11:10	24.2445
100yrHydrograph	11:15	23.63535
100yrHydrograph	11:20	23.08379
100yrHydrograph	11:25	22.58303
100yrHydrograph	11:30	22.12786
100yrHydrograph	11:35	21.71402
100yrHydrograph	11:40	21.33782
100yrHydrograph	11:45	20.99621
100yrHydrograph	11:50	20.68643
100yrHydrograph	11:55	20.40586
100yrHydrograph	12:00	20.1529
100yrHydrograph	12:05	19.91762
100yrHydrograph	12:10	19.63007
100yrHydrograph	12:15	19.3239
100yrHydrograph	12:20	19.12781
100yrHydrograph	12:25	18.97093
100yrHydrograph	12:30	18.80998

100yrHydrograph	12:35	18.64461
100yrHydrograph	12:40	18.47622
100yrHydrograph	12:45	18.30533
100yrHydrograph	12:50	18.13219
100yrHydrograph	12:55	17.95722
100yrHydrograph	13:00	17.74718
100yrHydrograph	13:05	17.56915
100yrHydrograph	13:10	17.39227
100yrHydrograph	13:15	17.21504
100yrHydrograph	13:20	17.03772
100yrHydrograph	13:25	16.86038
100yrHydrograph	13:30	16.68295
100yrHydrograph	13:35	16.50546
100yrHydrograph	13:40	16.32839
100yrHydrograph	13:45	16.15076
100yrHydrograph	13:50	15.97309
100yrHydrograph	13:55	15.79539
100yrHydrograph	14:00	15.61763
100yrHydrograph	14:05	15.44018
100yrHydrograph	14:10	15.26246
100yrHydrograph	14:15	15.08446
100yrHydrograph	14:20	14.90619
100yrHydrograph	14:25	14.72721
100yrHydrograph	14:30	14.54815
100yrHydrograph	14:35	14.36913
100yrHydrograph	14:40	14.19005
100yrHydrograph	14:45	14.011
100yrHydrograph	14:50	13.83076
100yrHydrograph	14:55	13.65002
100yrHydrograph	15:00	13.46919
100yrHydrograph	15:05	13.27224
100yrHydrograph	15:10	13.09266
100yrHydrograph	15:15	12.91431
100yrHydrograph	15:20	12.73593
100yrHydrograph	15:25	12.55792
100yrHydrograph	15:30	12.37996
100yrHydrograph	15:35	12.2021
100yrHydrograph	15:40	12.02427
100yrHydrograph	15:45	11.84651



100yrHydrograph	15:50	11.66898
100yrHydrograph	15:55	11.49166
100yrHydrograph	16:00	11.3145
100yrHydrograph	16:05	11.13787
100yrHydrograph	16:10	10.96224
100yrHydrograph	16:15	10.78573
100yrHydrograph	16:20	10.60909
100yrHydrograph	16:25	10.43269
100yrHydrograph	16:30	10.25677
100yrHydrograph	16:35	10.08101
100yrHydrograph	16:40	9.904943
100yrHydrograph	16:45	9.728643
100yrHydrograph	16:50	9.552533
100yrHydrograph	16:55	9.376752
100yrHydrograph	17:00	9.201365
100yrHydrograph	17:05	9.026419
100yrHydrograph	17:10	8.854236
100yrHydrograph	17:15	8.68074
100yrHydrograph	17:20	8.507633
100yrHydrograph	17:25	8.335072
100yrHydrograph	17:30	8.163301
100yrHydrograph	17:35	7.992058
100yrHydrograph	17:40	7.821409
100yrHydrograph	17:45	7.651455
100yrHydrograph	17:50	7.482112
100yrHydrograph	17:55	7.313401
100yrHydrograph	18:00	7.145336
100yrHydrograph	18:05	6.979621
100yrHydrograph	18:10	6.813243
100yrHydrograph	18:15	6.647324
100yrHydrograph	18:20	6.482216
100yrHydrograph	18:25	5.824821
100yrHydrograph	18:30	5.295127
100yrHydrograph	18:35	5.060137
100yrHydrograph	18:40	4.84829
100yrHydrograph	18:45	4.645963
100yrHydrograph	18:50	4.453671
100yrHydrograph	18:55	4.270811
100yrHydrograph	19:00	4.09688

100yrHydrograph	19:05	3.932506
100yrHydrograph	19:10	3.778864
100yrHydrograph	19:15	3.629502
100yrHydrograph	19:20	3.486742
100yrHydrograph	19:25	3.350802
100yrHydrograph	19:30	3.221082
100yrHydrograph	19:35	3.097033
100yrHydrograph	19:40	2.978455
100yrHydrograph	19:45	2.864869
100yrHydrograph	19:50	2.756282
100yrHydrograph	19:55	2.654007
100yrHydrograph	20:00	2.556878
100yrHydrograph	20:05	2.461823
100yrHydrograph	20:10	2.370567
100yrHydrograph	20:15	2.283155
100yrHydrograph	20:20	2.199545
100yrHydrograph	20:25	2.119371
100yrHydrograph	20:30	2.0424
100yrHydrograph	20:35	1.968455
100yrHydrograph	20:40	1.897445
100yrHydrograph	20:45	1.82913
100yrHydrograph	20:50	1.763424
100yrHydrograph	20:55	1.701357
100yrHydrograph	21:00	1.642923
100yrHydrograph	21:05	1.586518
100yrHydrograph	21:10	1.530593
100yrHydrograph	21:15	1.476459
100yrHydrograph	21:20	1.424285
100yrHydrograph	21:25	1.373979
100yrHydrograph	21:30	1.325569
100yrHydrograph	21:35	1.278883
100yrHydrograph	21:40	1.233917
100yrHydrograph	21:45	1.190506
100yrHydrograph	21:50	1.148588
100yrHydrograph	21:55	1.108119
100yrHydrograph	22:00	1.069048
100yrHydrograph	22:05	1.031321
100yrHydrograph	22:10	0.9948828
100yrHydrograph	22:15	0.9598325

100yrHydrograph	22:20	0.9265975
100yrHydrograph	22:25	0.895036
100yrHydrograph	22:30	0.8651206
100yrHydrograph	22:35	0.8349182
100yrHydrograph	22:40	0.8052853
100yrHydrograph	22:45	0.7765169
100yrHydrograph	22:50	0.7486013
100yrHydrograph	22:55	0.721274
100yrHydrograph	23:00	0.6946917
100yrHydrograph	23:05	0.6689637
100yrHydrograph	23:10	0.6440647
100yrHydrograph	23:15	0.619979
100yrHydrograph	23:20	0.5967006
100yrHydrograph	23:25	0.5741587
100yrHydrograph	23:30	0.5523183
100yrHydrograph	23:35	0.5311568
100yrHydrograph	23:40	0.5106529
100yrHydrograph	23:45	0.4907854
100yrHydrograph	23:50	0.4715335
100yrHydrograph	23:55	0.4528767

120SCS	0:00	0
120SCS	0:15	2.88
120SCS	0:30	2.88
120SCS	0:45	2.88
120SCS	1:00	2.88
120SCS	1:15	2.88
120SCS	1:30	2.88
120SCS	1:45	2.88
120SCS	2:00	2.88
120SCS	2:15	3.456
120SCS	2:30	3.456
120SCS	2:45	3.456
120SCS	3:00	3.456
120SCS	3:15	4.608
120SCS	3:30	4.608
120SCS	3:45	4.608
120SCS	4:00	4.608
120SCS	4:15	6.912

120SCS	4:30	6.912
120SCS	4:45	9.216
120SCS	5:00	9.216
120SCS	5:15	13.824
120SCS	5:30	13.824
120SCS	5:45	55.296
120SCS	6:00	152.064
120SCS	6:15	20.736
120SCS	6:30	20.736
120SCS	6:45	9.216
120SCS	7:00	9.216
120SCS	7:15	6.912
120SCS	7:30	6.912
120SCS	7:45	6.912
120SCS	8:00	6.912
120SCS	8:15	4.032
120SCS	8:30	4.032
120SCS	8:45	4.032
120SCS	9:00	4.032
120SCS	9:15	4.032
120SCS	9:30	4.032
120SCS	9:45	4.032
120SCS	10:00	4.032
120SCS	10:15	2.304
120SCS	10:30	2.304
120SCS	10:45	2.304
120SCS	11:00	2.304
120SCS	11:15	2.304
120SCS	11:30	2.304
120SCS	11:45	2.304
120SCS	12:00	0

```
[REPORT]
;Reporting Options
INPUT      YES
CONTROLS   NO
SUBCATCHMENTS ALL
NODES      ALL
LINKS      ALL
```

[TAGS]

Node 1000 MN

[MAP]

DIMENSIONS 350504.706880014 5015809.99134237 350738.979598863 5016090.18291699  
UNITS Meters

[COORDINATES]

```
;;Node X-Coord Y-Coord
;;-----
100 350580.8 5016032
HEADWALL 350565.848 5016038.828
OF1 350702.274 5015973.798
OF2 350547.792 5015987.469
OF3 350691.907 5015873.685
OF4 350536.617 5015877.154
1000 350599.591 5016022.489
CB507-S 350565.233 5015895.797
ROOF-10-S 350597.664 5015958.821
ROOF-11-S 350644.317 5015987.934
ROOF-12-S 350612.609 5016011.36
ROOF-1-2-S 350676.499 5015951.256
ROOF-3-S 350694.246 5015914.342
ROOF-4-S 350662.511 5015888.115
ROOF-5-S 350638.638 5015863.467
ROOF-6-7-S 350600.067 5015880.741
ROOF-8-S 350571.435 5015920.731
ROOF-9-S 350589.892 5015966.401
TANKS 350611.749 5016033.953
```

[VERTICES]

```
;;Link X-Coord Y-Coord
;;-----
W1 350597.646 5016038.74
```

[POLYGONS]

```
;;Subcatchment X-Coord Y-Coord
;;-----
```

[SYMBOLS]

```
;;Gage X-Coord Y-Coord
;;-----
```

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**C.3 SAMPLE PCSWMM MODEL OUTPUT (12HR 100YR SCS)**

WARNING 03: negative offset ignored for Link C2

\*\*\*\*\*

Element Count

\*\*\*\*\*

Number of rain gages ..... 1  
 Number of subcatchments ... 24  
 Number of nodes ..... 19  
 Number of links ..... 15  
 Number of pollutants ..... 0  
 Number of land uses ..... 0

\*\*\*\*\*

Raingage Summary

\*\*\*\*\*

Name	Data Source	Data Type	Recording Interval
RG1	100SCS	INTENSITY	15 min.

\*\*\*\*\*

Subcatchment Summary

\*\*\*\*\*

Name	Area	Width	%Imperv	%Slope	Rain Gage	Outlet
EXT-1	0.07	95.00	38.57	1.5000	RG1	CB507-S
ROOF_10	0.28	136.00	100.00	1.5000	RG1	ROOF-10-S
ROOF_11	0.01	21.00	100.00	1.5000	RG1	ROOF-11-S
ROOF_12	0.01	15.60	100.00	1.5000	RG1	ROOF-12-S
ROOF_3	0.11	130.00	100.00	1.5000	RG1	ROOF-3-S
ROOF_4	0.04	46.00	100.00	1.5000	RG1	ROOF-4-S
ROOF_5	0.11	130.00	100.00	1.5000	RG1	ROOF-5-S

ROOF_8	0.01	21.00	100.00	1.5000	RG1	ROOF-8-S
ROOF_9	0.01	15.00	100.00	1.5000	RG1	ROOF-9-S
ROOF1_2	0.10	95.00	100.00	1.5000	RG1	ROOF-1-2-S
ROOF6_7	0.10	95.00	100.00	1.5000	RG1	ROOF-6-7-S
UGPK_1	0.14	115.00	77.14	2.0000	RG1	TANKS
UGPK_2	0.15	122.00	80.00	2.0000	RG1	TANKS
UGPK_3	0.06	60.00	58.57	2.0000	RG1	TANKS
UGPK_4	0.12	95.00	70.00	2.0000	RG1	TANKS
UGPK_5	0.11	85.00	70.00	2.0000	RG1	TANKS
UGPK_6	0.02	60.00	100.00	15.0000	RG1	TANKS
UGPK_7	0.11	78.00	78.57	2.0000	RG1	TANKS
UGPK_8	0.06	42.00	75.71	2.0000	RG1	TANKS
UGPK_9	0.03	42.00	100.00	2.0000	RG1	TANKS
UNC-1	0.08	78.00	41.43	2.0000	RG1	OF1
UNC-2	0.52	25.00	8.57	1.0000	RG1	OF2
UNC-3	0.07	122.00	61.43	2.0000	RG1	OF3
UNC-4	0.05	90.00	37.14	2.0000	RG1	CB507-S

\*\*\*\*\*

Node Summary

\*\*\*\*\*

Name	Type	Invert Elev.	Max. Depth	Ponded Area	External Inflow
100	JUNCTION	99.40	2.73	0.0	Yes
HEADWALL	OUTFALL	98.70	1.72	0.0	
OF1	OUTFALL	0.00	0.00	0.0	
OF2	OUTFALL	0.00	0.00	0.0	
OF3	OUTFALL	0.00	0.00	0.0	
OF4	OUTFALL	101.87	0.83	0.0	
1000	STORAGE	99.92	4.18	0.0	
CB507-S	STORAGE	102.56	2.23	0.0	
ROOF-10-S	STORAGE	114.00	0.15	0.0	
ROOF-11-S	STORAGE	114.00	0.15	0.0	
ROOF-12-S	STORAGE	114.00	0.15	0.0	
ROOF-1-2-S	STORAGE	114.00	0.15	0.0	
ROOF-3-S	STORAGE	114.00	0.15	0.0	
ROOF-4-S	STORAGE	114.00	0.15	0.0	

ROOF-5-S	STORAGE	114.00	0.15	0.0
ROOF-6-7-S	STORAGE	114.00	0.15	0.0
ROOF-8-S	STORAGE	114.00	0.15	0.0
ROOF-9-S	STORAGE	114.00	0.15	0.0
TANKS	STORAGE	99.70	3.61	0.0

\*\*\*\*\*  
Link Summary  
\*\*\*\*\*

Name	From Node	To Node	Type	Length	%Slope	Roughness
C1	CB507-S	OF4	CONDUIT	21.3	0.5164	0.0130
C2	1000	100	CONDUIT	20.8	0.4327	0.0130
Pipe_13	100	HEADWALL	CONDUIT	11.1	0.2515	0.0130
CISTERN-0	TANKS	1000	ORIFICE			
W1	TANKS	1000	WEIR			
ROOF10-0	ROOF-10-S	TANKS	OUTLET			
ROOF11-0	ROOF-11-S	TANKS	OUTLET			
ROOF12-0	ROOF-12-S	TANKS	OUTLET			
ROOF1-2-0	ROOF-1-2-S	TANKS	OUTLET			
ROOF3-0	ROOF-3-S	TANKS	OUTLET			
ROOF4-0	ROOF-4-S	TANKS	OUTLET			
ROOF5-0	ROOF-5-S	TANKS	OUTLET			
ROOF6-7-0	ROOF-6-7-S	TANKS	OUTLET			
ROOF8-0	ROOF-8-S	TANKS	OUTLET			
ROOF9-0	ROOF-9-S	TANKS	OUTLET			

\*\*\*\*\*  
Cross Section Summary  
\*\*\*\*\*

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full Flow
C1	CIRCULAR	0.25	0.05	0.06	0.25	1	42.74
C2	CIRCULAR	0.45	0.16	0.11	0.45	1	187.55
Pipe_13	CIRCULAR	0.90	0.64	0.23	0.90	1	907.85

\*\*\*\*\*  
Transect Summary  
\*\*\*\*\*

Transect Overland  
Area:

	0.0196	0.0392	0.0588	0.0784	0.0980
	0.1177	0.1374	0.1571	0.1768	0.1965
	0.2162	0.2360	0.2558	0.2756	0.2954
	0.3152	0.3351	0.3550	0.3748	0.3947
	0.4147	0.4346	0.4546	0.4745	0.4945
	0.5145	0.5346	0.5546	0.5747	0.5947
	0.6148	0.6350	0.6551	0.6752	0.6954
	0.7156	0.7358	0.7560	0.7762	0.7965
	0.8168	0.8371	0.8574	0.8777	0.8980
	0.9184	0.9388	0.9592	0.9796	1.0000
Hrad:					
	0.0208	0.0415	0.0622	0.0829	0.1036
	0.1242	0.1448	0.1653	0.1858	0.2063
	0.2268	0.2472	0.2676	0.2879	0.3083
	0.3285	0.3488	0.3690	0.3892	0.4094
	0.4295	0.4496	0.4697	0.4897	0.5097
	0.5297	0.5496	0.5695	0.5894	0.6093
	0.6291	0.6489	0.6686	0.6884	0.7081
	0.7277	0.7474	0.7670	0.7865	0.8061
	0.8256	0.8451	0.8646	0.8840	0.9034
	0.9228	0.9421	0.9614	0.9807	1.0000
Width:					
	0.9580	0.9589	0.9597	0.9606	0.9614
	0.9623	0.9631	0.9640	0.9649	0.9657
	0.9666	0.9674	0.9683	0.9691	0.9700
	0.9709	0.9717	0.9726	0.9734	0.9743
	0.9751	0.9760	0.9769	0.9777	0.9786
	0.9794	0.9803	0.9811	0.9820	0.9829
	0.9837	0.9846	0.9854	0.9863	0.9871
	0.9880	0.9889	0.9897	0.9906	0.9914
	0.9923	0.9931	0.9940	0.9949	0.9957

	0.9966	0.9974	0.9983	0.9991	1.0000
Transect Overland(orig)					
Area:					
	0.0196	0.0392	0.0588	0.0784	0.0980
	0.1177	0.1374	0.1571	0.1768	0.1965
	0.2162	0.2360	0.2558	0.2756	0.2954
	0.3152	0.3351	0.3550	0.3748	0.3947
	0.4147	0.4346	0.4546	0.4745	0.4945
	0.5145	0.5346	0.5546	0.5747	0.5947
	0.6148	0.6350	0.6551	0.6752	0.6954
	0.7156	0.7358	0.7560	0.7762	0.7965
	0.8168	0.8371	0.8574	0.8777	0.8980
	0.9184	0.9388	0.9592	0.9796	1.0000
Hrad:					
	0.0208	0.0415	0.0622	0.0829	0.1036
	0.1242	0.1448	0.1653	0.1858	0.2063
	0.2268	0.2472	0.2676	0.2879	0.3083
	0.3285	0.3488	0.3690	0.3892	0.4094
	0.4295	0.4496	0.4697	0.4897	0.5097
	0.5297	0.5496	0.5695	0.5894	0.6093
	0.6291	0.6489	0.6686	0.6884	0.7081
	0.7277	0.7474	0.7670	0.7865	0.8061
	0.8256	0.8451	0.8646	0.8840	0.9034
	0.9228	0.9421	0.9614	0.9807	1.0000
Width:					
	0.9580	0.9589	0.9597	0.9606	0.9614
	0.9623	0.9631	0.9640	0.9649	0.9657
	0.9666	0.9674	0.9683	0.9691	0.9700
	0.9709	0.9717	0.9726	0.9734	0.9743
	0.9751	0.9760	0.9769	0.9777	0.9786
	0.9794	0.9803	0.9811	0.9820	0.9829
	0.9837	0.9846	0.9854	0.9863	0.9871
	0.9880	0.9889	0.9897	0.9906	0.9914
	0.9923	0.9931	0.9940	0.9949	0.9957
	0.9966	0.9974	0.9983	0.9991	1.0000

\*\*\*\*\*

NOTE: The summary statistics displayed in this report are based on results found at every computational time step, not just on results from each reporting time step.

\*\*\*\*\*

\*\*\*\*\*

Analysis Options

\*\*\*\*\*

Flow Units ..... LPS

Process Models:

Rainfall/Runoff ..... YES

RDII ..... NO

Snowmelt ..... NO

Groundwater ..... NO

Flow Routing ..... YES

Ponding Allowed ..... YES

Water Quality ..... NO

Infiltration Method ..... HORTON

Flow Routing Method ..... DYNWAVE

Surcharge Method ..... EXTRAN

Starting Date ..... 07/23/2009 00:00:00

Ending Date ..... 07/24/2009 00:00:00

Antecedent Dry Days ..... 0.0

Report Time Step ..... 00:05:00

Wet Time Step ..... 00:05:00

Dry Time Step ..... 00:05:00

Routing Time Step ..... 1.00 sec

Variable Time Step ..... NO

Maximum Trials ..... 8

Number of Threads ..... 1

Head Tolerance ..... 0.001500 m

*****	Volume	Depth
Runoff Quantity Continuity	hectare-m	mm
*****	-----	-----
Total Precipitation .....	0.226	95.520
Evaporation Loss .....	0.000	0.000
Infiltration Loss .....	0.063	26.653

```

Surface Runoff ..... 0.161 68.040
Final Storage ..... 0.002 1.029
Continuity Error (%) ..... -0.212

```

```

*****
Flow Routing Continuity      Volume      Volume
                             hectare-m   10^6 ltr
*****
Dry Weather Inflow ..... 0.000 0.000
Wet Weather Inflow ..... 0.161 1.607
Groundwater Inflow ..... 0.000 0.000
RDII Inflow ..... 0.000 0.000
External Inflow ..... 0.183 1.828
External Outflow ..... 0.336 3.358
Flooding Loss ..... 0.000 0.000
Evaporation Loss ..... 0.000 0.000
Exfiltration Loss ..... 0.000 0.000
Initial Stored Volume .... 0.000 0.000
Final Stored Volume ..... 0.008 0.077
Continuity Error (%) ..... 0.005

```

```

*****
Highest Flow Instability Indexes
*****
All links are stable.

```

```

*****
Routing Time Step Summary
*****
Minimum Time Step      : 1.00 sec
Average Time Step      : 1.00 sec
Maximum Time Step      : 1.00 sec
Percent in Steady State : 0.00
Average Iterations per Step : 2.00
Percent Not Converging  : 0.00

```

```

*****
Subcatchment Runoff Summary
*****

```

-----			Total	Total	Total	Total	Imperv	Perv	Total	
Total	Peak	Runoff	Total	Total	Total	Total	Imperv	Perv	Total	
Runoff	Runoff	Coeff	Precip	Runon	Evap	Infil	Runoff	Runoff	Runoff	
Subcatchment	Subcatchment		mm	mm	mm	mm	mm	mm	mm	10^6
ltr	LPS									
-----										
EXT-1			95.52	0.00	0.00	52.83	36.25	42.60	42.60	
0.03	22.66	0.446								
ROOF_10			95.52	0.00	0.00	0.00	94.22	0.00	94.22	
0.26	98.91	0.986								
ROOF_11			95.52	0.00	0.00	0.00	93.99	0.00	93.99	
0.01	3.73	0.984								
ROOF_12			95.52	0.00	0.00	0.00	94.03	0.00	94.03	
0.01	4.41	0.984								
ROOF_3			95.52	0.00	0.00	0.00	94.04	0.00	94.04	
0.10	38.65	0.985								
ROOF_4			95.52	0.00	0.00	0.00	94.03	0.00	94.03	
0.03	12.40	0.984								
ROOF_5			95.52	0.00	0.00	0.00	94.04	0.00	94.04	
0.10	38.66	0.985								
ROOF_8			95.52	0.00	0.00	0.00	93.99	0.00	93.99	
0.01	3.43	0.984								
ROOF_9			95.52	0.00	0.00	0.00	93.98	0.00	93.98	
0.00	1.87	0.984								
ROOF1_2			95.52	0.00	0.00	0.00	94.07	0.00	94.07	
0.09	33.58	0.985								
ROOF6_7			95.52	0.00	0.00	0.00	94.07	0.00	94.07	
0.09	33.62	0.985								
UGPK_1			95.52	0.00	0.00	14.36	80.16	7.64	80.16	



0.12	49.41	0.839							
UGPK_2			95.52	0.00	0.00	12.55	81.90	6.69	81.90
0.12	52.49	0.857							
UGPK_3			95.52	0.00	0.00	26.08	68.85	13.80	68.85
0.04	20.01	0.721							
UGPK_4			95.52	0.00	0.00	18.87	75.81	10.00	75.81
0.09	40.79	0.794							
UGPK_5			95.52	0.00	0.00	18.87	75.81	10.00	75.81
0.08	37.46	0.794							
UGPK_6			95.52	0.00	0.00	0.00	93.99	0.00	93.99
0.02	7.74	0.984							
UGPK_7			95.52	0.00	0.00	13.46	81.04	7.16	81.04
0.09	38.52	0.848							
UGPK_8			95.52	0.00	0.00	15.27	79.30	8.10	79.30
0.05	21.12	0.830							
UGPK_9			95.52	0.00	0.00	0.00	94.01	0.00	94.01
0.03	11.43	0.984							
UNC-1			95.52	0.00	0.00	51.92	38.94	43.47	43.47
0.03	25.77	0.455							
UNC-2			95.52	0.00	0.00	72.25	8.08	23.25	23.25
0.12	39.00	0.243							
UNC-3			95.52	0.00	0.00	43.07	57.73	51.73	51.73
0.04	23.41	0.542							
UNC-4			95.52	0.00	0.00	53.22	34.91	42.20	42.20
0.02	16.94	0.442							

\*\*\*\*\*  
Node Depth Summary  
\*\*\*\*\*

Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
100	JUNCTION	0.23	0.43	99.83	0 06:46	0.43
HEADWALL	OUTFALL	0.00	0.00	98.70	0 00:00	0.00
OF1	OUTFALL	0.00	0.00	0.00	0 00:00	0.00

OF2	OUTFALL	0.00	0.00	0.00	0 00:00	0.00
OF3	OUTFALL	0.00	0.00	0.00	0 00:00	0.00
OF4	OUTFALL	0.30	0.30	102.17	0 00:00	0.30
1000	STORAGE	0.07	0.25	100.17	0 06:23	0.25
CB507-S	STORAGE	0.00	0.18	102.74	0 06:15	0.18
ROOF-10-S	STORAGE	0.02	0.14	114.14	0 06:19	0.14
ROOF-11-S	STORAGE	0.01	0.12	114.12	0 06:18	0.12
ROOF-12-S	STORAGE	0.01	0.13	114.13	0 06:18	0.13
ROOF-1-2-S	STORAGE	0.01	0.14	114.14	0 06:19	0.14
ROOF-3-S	STORAGE	0.02	0.15	114.15	0 06:19	0.15
ROOF-4-S	STORAGE	0.01	0.13	114.13	0 06:19	0.13
ROOF-5-S	STORAGE	0.02	0.15	114.15	0 06:19	0.15
ROOF-6-7-S	STORAGE	0.01	0.14	114.14	0 06:19	0.14
ROOF-8-S	STORAGE	0.01	0.11	114.11	0 06:18	0.11
ROOF-9-S	STORAGE	0.00	0.09	114.09	0 06:16	0.08
TANKS	STORAGE	1.26	2.78	102.48	0 06:23	2.78

\*\*\*\*\*  
Node Inflow Summary  
\*\*\*\*\*

Node	Type	Maximum Lateral Inflow LPS	Maximum Total Inflow LPS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
100	JUNCTION	136.19	222.37	0 06:46	1.83	3.12	0.008
HEADWALL	OUTFALL	0.00	222.38	0 06:46	0	3.12	0.000
OF1	OUTFALL	25.77	25.77	0 06:15	0.0339	0.0339	0.000
OF2	OUTFALL	39.00	39.00	0 06:15	0.12	0.12	0.000
OF3	OUTFALL	23.41	23.41	0 06:15	0.0359	0.0359	0.000
OF4	OUTFALL	0.00	39.58	0 06:15	0	0.0511	0.000
1000	STORAGE	0.00	111.32	0 06:23	0	1.29	0.010
CB507-S	STORAGE	39.61	39.61	0 06:15	0.0511	0.0511	-0.001
ROOF-10-S	STORAGE	98.91	98.91	0 06:10	0.265	0.265	-0.001
ROOF-11-S	STORAGE	3.73	3.73	0 06:05	0.00997	0.00997	-0.001
ROOF-12-S	STORAGE	4.41	4.41	0 06:10	0.0118	0.0118	-0.001

ROOF-1-2-S	STORAGE	33.58	33.58	0	06:10	0.0897	0.0897	-0.001
ROOF-3-S	STORAGE	38.65	38.65	0	06:10	0.103	0.103	-0.001
ROOF-4-S	STORAGE	12.40	12.40	0	06:15	0.0331	0.0331	-0.001
ROOF-5-S	STORAGE	38.66	38.66	0	06:10	0.103	0.103	-0.001
ROOF-6-7-S	STORAGE	33.62	33.62	0	06:10	0.0898	0.0898	-0.001
ROOF-8-S	STORAGE	3.43	3.43	0	06:05	0.00916	0.00916	-0.001
ROOF-9-S	STORAGE	1.87	1.87	0	06:10	0.00499	0.00499	-0.001
TANKS	STORAGE	278.96	337.67	0	06:15	0.647	1.37	0.001

\*\*\*\*\*  
Node Surcharge Summary  
\*\*\*\*\*

No nodes were surcharged.

\*\*\*\*\*  
Node Flooding Summary  
\*\*\*\*\*

No nodes were flooded.

\*\*\*\*\*  
Storage Volume Summary  
\*\*\*\*\*

Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow LPS
1000	0.000	2	0	0	0.000	6	0 06:23	111.34
CB507-S	0.000	0	0	0	0.000	0	0 00:00	39.58
ROOF-10-S	0.006	5	0	0	0.093	82	0 06:19	19.23
ROOF-11-S	0.000	1	0	0	0.002	48	0 06:18	1.68
ROOF-12-S	0.000	2	0	0	0.003	67	0 06:18	1.77
ROOF-1-2-S	0.002	4	0	0	0.030	78	0 06:19	8.29

ROOF-3-S	0.003	7	0	0	0.041	91	0 06:19	7.34
ROOF-4-S	0.000	3	0	0	0.010	67	0 06:19	3.15
ROOF-5-S	0.003	7	0	0	0.041	91	0 06:19	7.34
ROOF-6-7-S	0.002	4	0	0	0.030	78	0 06:19	8.29
ROOF-8-S	0.000	1	0	0	0.002	44	0 06:18	1.66
ROOF-9-S	0.000	0	0	0	0.000	19	0 06:16	1.48
TANKS	0.281	35	0	0	0.618	77	0 06:23	111.32

\*\*\*\*\*  
Outfall Loading Summary  
\*\*\*\*\*

Outfall Node	Flow Freq Pcnt	Avg Flow LPS	Max Flow LPS	Total Volume 10^6 ltr
HEADWALL	90.72	39.77	222.38	3.117
OF1	6.58	5.97	25.77	0.034
OF2	11.58	11.97	39.00	0.120
OF3	12.63	3.29	23.41	0.036
OF4	6.36	9.30	39.58	0.051
System	25.57	70.29	253.17	3.358

\*\*\*\*\*  
Link Flow Summary  
\*\*\*\*\*

Link	Type	Maximum  Flow  LPS	Time of Max Occurrence days hr:min	Maximum  Veloc  m/sec	Max/ Full Flow	Max/ Full Depth
C1	CONDUIT	39.58	0 06:15	1.09	0.93	0.69
C2	CONDUIT	111.34	0 06:24	1.28	0.59	0.54
Pipe_13	CONDUIT	222.38	0 06:46	1.35	0.24	0.31

CISTERN-0	ORIFICE	18.15	0	06:23	1.00
W1	WEIR	93.17	0	06:23	0.23
ROOF10-0	DUMMY	19.23	0	06:19	
ROOF11-0	DUMMY	1.68	0	06:18	
ROOF12-0	DUMMY	1.77	0	06:18	
ROOF1-2-0	DUMMY	8.29	0	06:19	
ROOF3-0	DUMMY	7.34	0	06:19	
ROOF4-0	DUMMY	3.15	0	05:52	
ROOF5-0	DUMMY	7.34	0	06:19	
ROOF6-7-0	DUMMY	8.29	0	06:19	
ROOF8-0	DUMMY	1.66	0	06:18	
ROOF9-0	DUMMY	1.48	0	06:16	

\*\*\*\*\*  
Flow Classification Summary  
\*\*\*\*\*

Conduit	Adjusted /Actual Length	----- Fraction of Time in Flow Class -----									
		Up Dry	Down Dry	Sub Dry	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl		
C1	1.00	0.24	0.00	0.00	0.00	0.00	0.76	0.00	0.00		
C2	1.00	0.11	0.00	0.00	0.00	0.00	0.89	0.00	0.00		
Pipe_13	1.00	0.09	0.00	0.00	0.00	0.00	0.91	0.00	0.00		

\*\*\*\*\*  
Conduit Surcharge Summary  
\*\*\*\*\*

No conduits were surcharged.

Analysis begun on: Tue Mar 29 13:35:49 2022  
Analysis ended on: Tue Mar 29 13:35:50 2022  
Total elapsed time: 00:00:01

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**C.4 OIL/GRIT SEPARATOR SIZING CALCULATIONS**

## Detailed Stormceptor Sizing Report – WOS PH2 20 Cedarow Crt

Project Information & Location			
<b>Project Name</b>	WOS PH2	<b>Project Number</b>	20349
<b>City</b>	Ottawa	<b>State/ Province</b>	Ontario
<b>Country</b>	Canada	<b>Date</b>	11/4/2019
Designer Information		EOR Information (optional)	
<b>Name</b>	thakshika rathnasooriya	<b>Name</b>	
<b>Company</b>	stantec	<b>Company</b>	
<b>Phone #</b>	613-724-4081	<b>Phone #</b>	
<b>Email</b>	thakshika.rathnasooriya@stantec.com	<b>Email</b>	

### Stormwater Treatment Recommendation

The recommended Stormceptor Model(s) which achieve or exceed the user defined water quality objective for each site within the project are listed in the below Sizing Summary table.

<b>Site Name</b>	WOS PH2 20 Cedarow Crt
<b>Recommended Stormceptor Model</b>	STC 300
<b>Target TSS Removal (%)</b>	80.0
<b>TSS Removal (%) Provided</b>	80
<b>PSD</b>	Fine Distribution
<b>Rainfall Station</b>	OTTAWA MACDONALD-CARTIER INT'L A

The recommended Stormceptor model achieves the water quality objectives based on the selected inputs, historical rainfall records and selected particle size distribution.

Stormceptor Sizing Summary	
Stormceptor Model	% TSS Removal Provided
STC 300	80
STC 750	85
STC 1000	85
STC 1500	85
STC 2000	86
STC 3000	87
STC 4000	88
STC 5000	89
STC 6000	90
STC 9000	92
STC 10000	92
STC 14000	94
StormceptorMAX	Custom

**Stormceptor**

The Stormceptor oil and sediment separator is sized to treat stormwater runoff by removing pollutants through gravity separation and flotation. Stormceptor’s patented design generates positive TSS removal for each rainfall event, including large storms. Significant levels of pollutants such as heavy metals, free oils and nutrients are prevented from entering natural water resources and the re-suspension of previously captured sediment (scour) does not occur. Stormceptor provides a high level of TSS removal for small frequent storm events that represent the majority of annual rainfall volume and pollutant load. Positive treatment continues for large infrequent events, however, such events have little impact on the average annual TSS removal as they represent a small percentage of the total runoff volume and pollutant load.

**Design Methodology**

Stormceptor is sized using PCSWMM for Stormceptor, a continuous simulation model based on US EPA SWMM. The program calculates hydrology using local historical rainfall data and specified site parameters. With US EPA SWMM’s precision, every Stormceptor unit is designed to achieve a defined water quality objective. The TSS removal data presented follows US EPA guidelines to reduce the average annual TSS load. The Stormceptor’s unit process for TSS removal is settling. The settling model calculates TSS removal by analyzing:

- Site parameters
- Continuous historical rainfall data, including duration, distribution, peaks & inter-event dry periods
- Particle size distribution, and associated settling velocities (Stokes Law, corrected for drag)
- TSS load
- Detention time of the system

Hydrology Analysis	
PCSWMM for Stormceptor calculates annual hydrology with the US EPA SWMM and local continuous historical rainfall data. Performance calculations of Stormceptor are based on the average annual removal of TSS for the selected site parameters. The Stormceptor is engineered to capture sediment particles by treating the required average annual runoff volume, ensuring positive removal efficiency is maintained during each rainfall event, and preventing negative removal efficiency (scour). Smaller recurring storms account for the majority of rainfall events and average annual runoff volume, as observed in the historical rainfall data analyses presented in this section.	

Rainfall Station			
<b>State/Province</b>	Ontario	<b>Total Number of Rainfall Events</b>	4093
<b>Rainfall Station Name</b>	OTTAWA MACDONALD-CARTIER INT’L A	<b>Total Rainfall (mm)</b>	20978.1
<b>Station ID #</b>	6000	<b>Average Annual Rainfall (mm)</b>	567.0
<b>Coordinates</b>	45°19’N, 75°40’W	<b>Total Evaporation (mm)</b>	982.0
<b>Elevation (ft)</b>	370	<b>Total Infiltration (mm)</b>	10341.2
<b>Years of Rainfall Data</b>	37	<b>Total Rainfall that is Runoff (mm)</b>	9654.9

Notes	
<ul style="list-style-type: none"> <li>• Stormceptor performance estimates are based on simulations using PCSWMM for Stormceptor, which uses the EPA Rainfall and Runoff modules.</li> <li>• Design estimates listed are only representative of specific project requirements based on total suspended solids (TSS) removal defined by the selected PSD, and based on stable site conditions only, after construction is completed.</li> <li>• For submerged applications or sites specific to spill control, please contact your local Stormceptor representative for further design assistance.</li> </ul>	

Drainage Area	
Total Area (ha)	1.60
Imperviousness %	50.60

Up Stream Storage	
Storage (ha-m)	Discharge (cms)
0.000	0.000
0.030	0.007
0.060	0.015
0.090	0.022

Water Quality Objective	
TSS Removal (%)	80.0
Runoff Volume Capture (%)	
Oil Spill Capture Volume (L)	
Peak Conveyed Flow Rate (L/s)	126.00
Water Quality Flow Rate (L/s)	

Up Stream Flow Diversion	
Max. Flow to Stormceptor (cms)	

Design Details	
Stormceptor Inlet Invert Elev (m)	
Stormceptor Outlet Invert Elev (m)	
Stormceptor Rim Elev (m)	
Normal Water Level Elevation (m)	
Pipe Diameter (mm)	
Pipe Material	
Multiple Inlets (Y/N)	No
Grate Inlet (Y/N)	No

Particle Size Distribution (PSD)		
Removing the smallest fraction of particulates from runoff ensures the majority of pollutants, such as metals, hydrocarbons and nutrients are captured. The table below identifies the Particle Size Distribution (PSD) that was selected to define TSS removal for the Stormceptor design.		
Fine Distribution		
Particle Diameter (microns)	Distribution %	Specific Gravity
20.0	20.0	1.30
60.0	20.0	1.80
150.0	20.0	2.20
400.0	20.0	2.65
2000.0	20.0	2.65

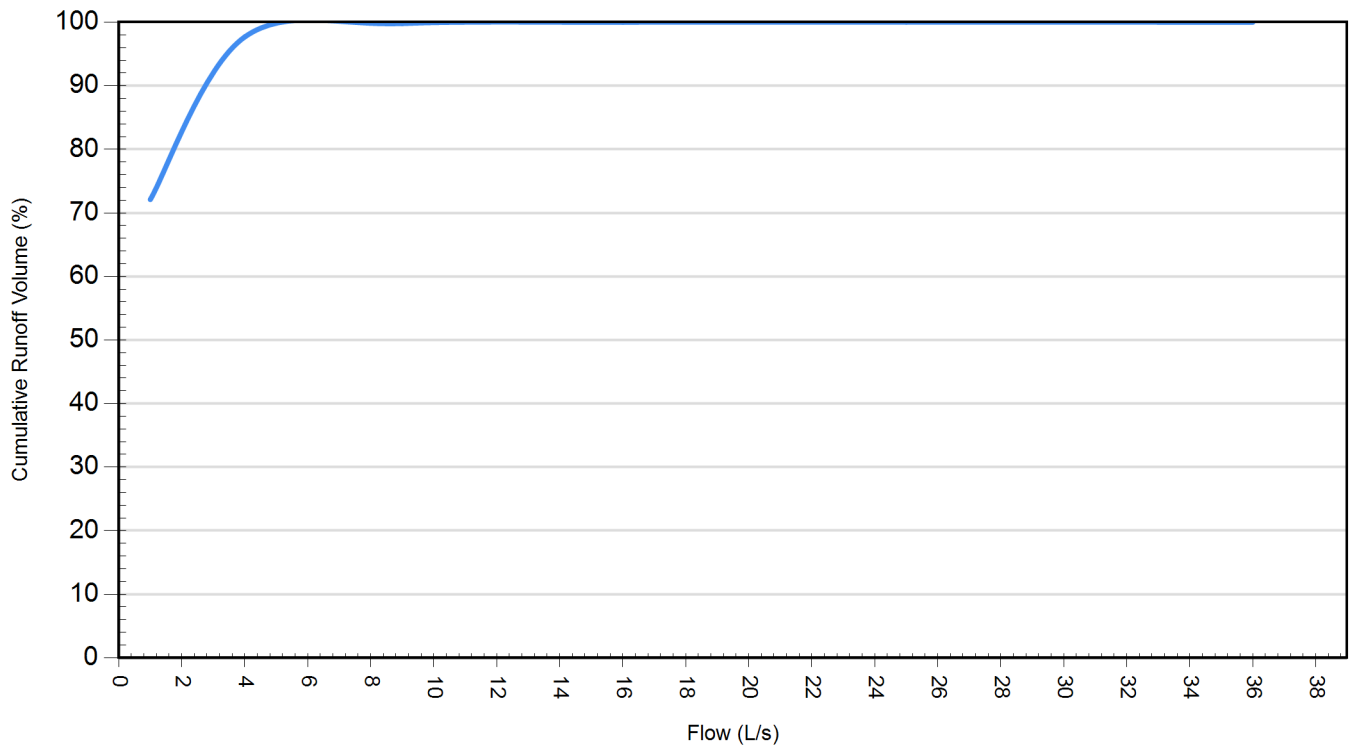
Site Name		WOS PH2 20 Cedarow Crt	
<b>Site Details</b>			
<b>Drainage Area</b>		<b>Infiltration Parameters</b>	
Total Area (ha)	1.60	Horton's equation is used to estimate infiltration	
Imperviousness %	50.60	Max. Infiltration Rate (mm/hr)	61.98
<b>Surface Characteristics</b>		Min. Infiltration Rate (mm/hr)	10.16
Width (m)	253.00	Decay Rate (1/sec)	0.00055
Slope %	2	Regeneration Rate (1/sec)	0.01
Impervious Depression Storage (mm)	0.508	<b>Evaporation</b>	
Pervious Depression Storage (mm)	5.08	Daily Evaporation Rate (mm/day)	2.54
Impervious Manning's n	0.015	<b>Dry Weather Flow</b>	
Pervious Manning's n	0.25	Dry Weather Flow (lps)	0
<b>Maintenance Frequency</b>		<b>Winter Months</b>	
Maintenance Frequency (months) >	12	Winter Infiltration	0
<b>TSS Loading Parameters</b>			
TSS Loading Function			
<b>Buildup/Wash-off Parameters</b>		<b>TSS Availability Parameters</b>	
Target Event Mean Conc. (EMC) mg/L		Availability Constant A	
Exponential Buildup Power		Availability Factor B	
Exponential Washoff Exponent		Availability Exponent C	
		Min. Particle Size Affected by Availability (micron)	



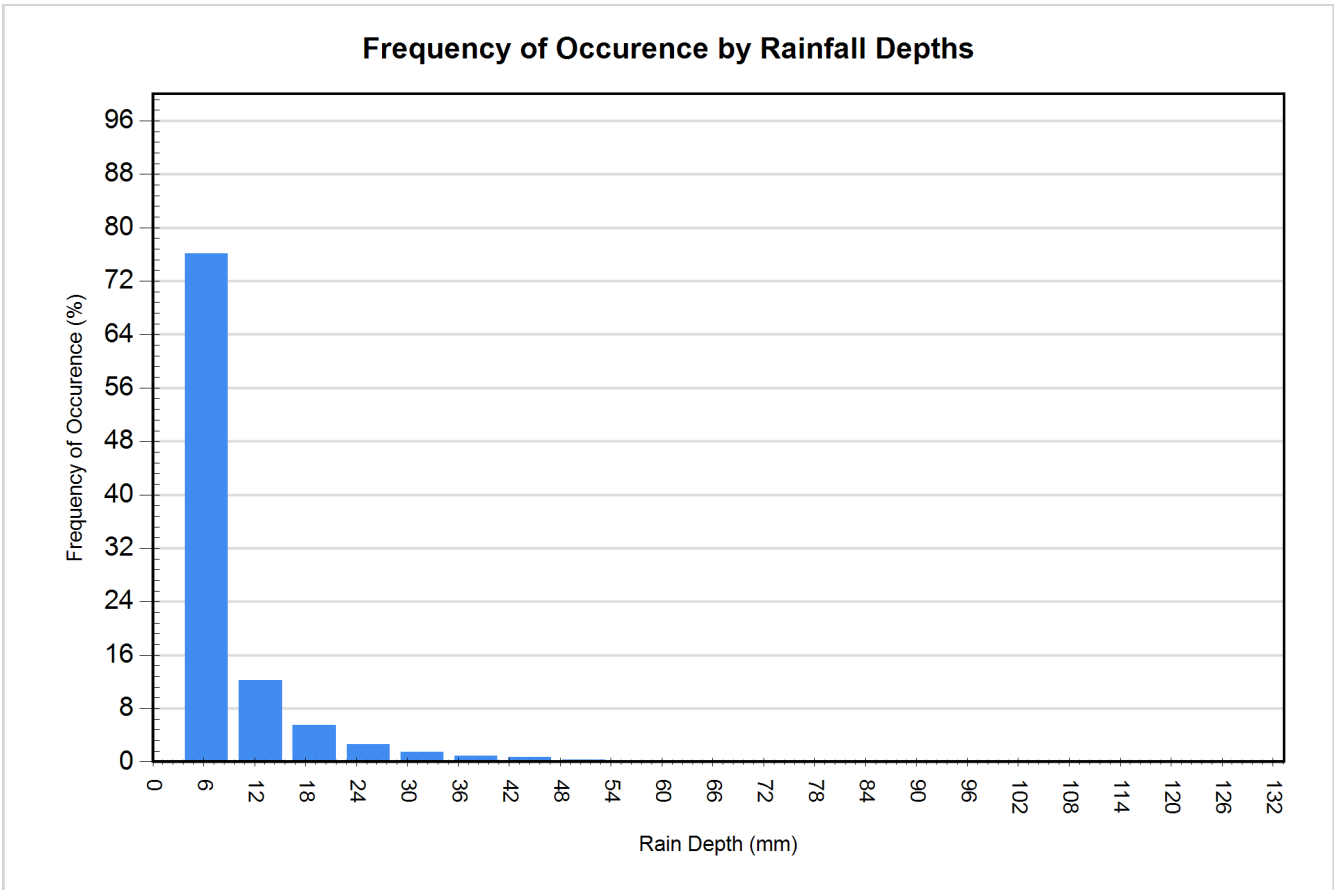
Cumulative Runoff Volume by Runoff Rate			
Runoff Rate (L/s)	Runoff Volume (m³)	Volume Over (m³)	Cumulative Runoff Volume (%)
1	111983	43648	72.1
4	151637	3654	97.7
9	154907	370	99.8
16	155201	73	100.0
25	155273	0	100.0
36	155273	0	100.0

**Cumulative Runoff Volume by Runoff Rate**

For area: 1.60(ha), imperviousness: 50.60%, rainfall station: OTTAWA MACDONALD-CARTIER INT'L A



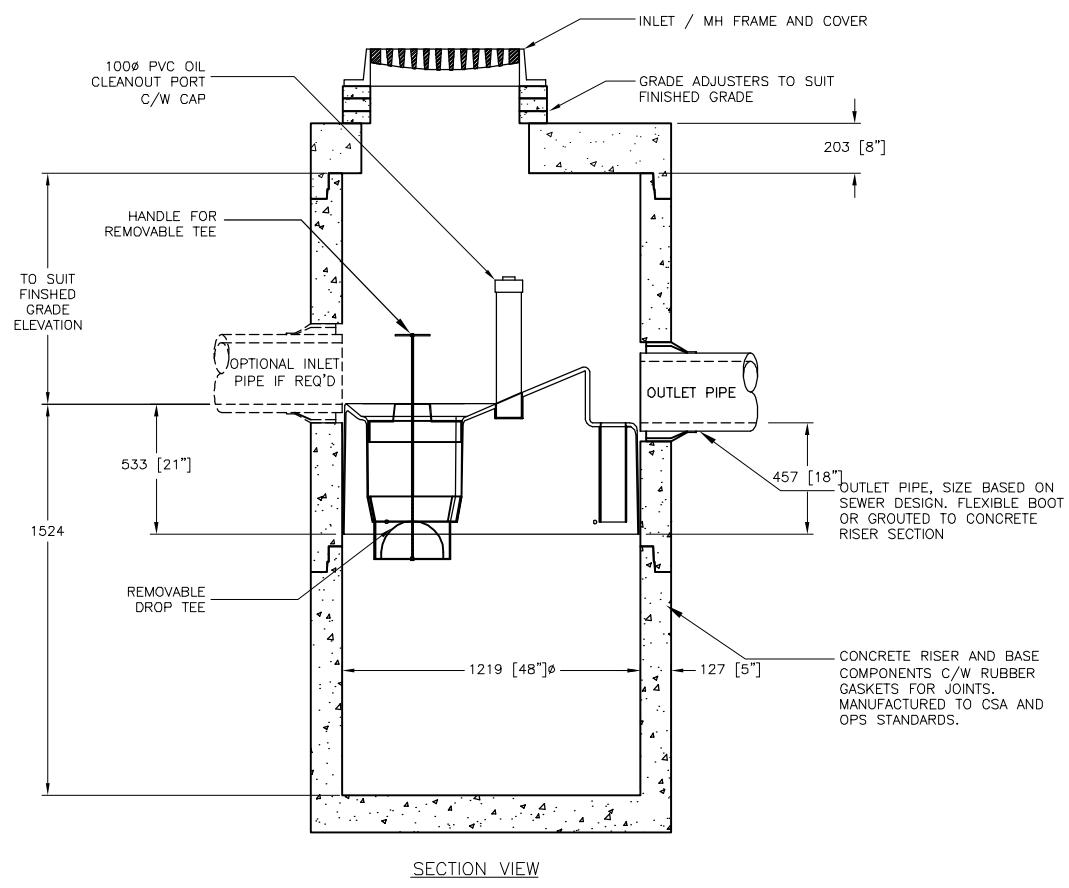
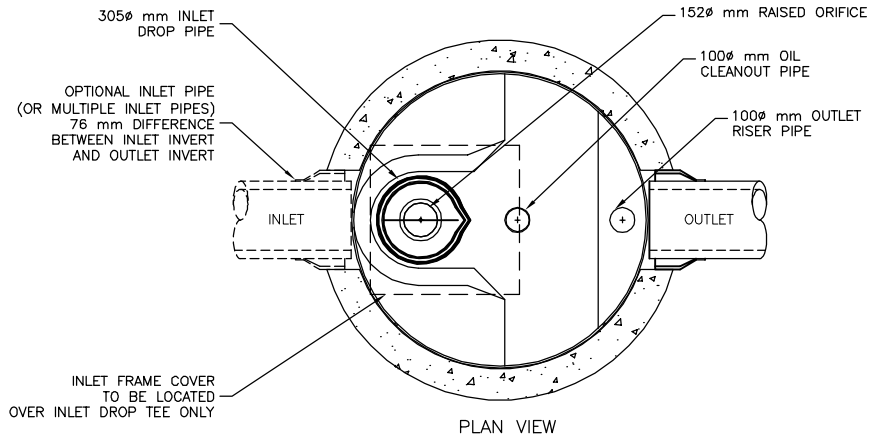
Rainfall Event Analysis				
Rainfall Depth (mm)	No. of Events	Percentage of Total Events (%)	Total Volume (mm)	Percentage of Annual Volume (%)
6.35	3113	76.1	5230	24.9
12.70	501	12.2	4497	21.4
19.05	225	5.5	3469	16.5
25.40	105	2.6	2317	11.0
31.75	62	1.5	1765	8.4
38.10	35	0.9	1206	5.8
44.45	28	0.7	1163	5.5
50.80	12	0.3	557	2.7
57.15	7	0.2	378	1.8
63.50	1	0.0	63	0.3
69.85	1	0.0	64	0.3
76.20	1	0.0	76	0.4
82.55	0	0.0	0	0.0
88.90	1	0.0	84	0.4
95.25	0	0.0	0	0.0
101.60	0	0.0	0	0.0
107.95	0	0.0	0	0.0
114.30	1	0.0	109	0.5
120.65	0	0.0	0	0.0
127.00	0	0.0	0	0.0



**For Stormceptor Specifications and Drawings Please Visit:**  
<http://www.imbriumsystems.com/technical-specifications>

# DRAWING NOT TO BE USED FOR CONSTRUCTION


THE STORMCEPTOR SYSTEM IS PROTECTED BY ONE OR MORE OF THE FOLLOWING PATENTS:  
 United States Patent No. 5,753,115 • 5,849,181 • 6,068,765 • 6,371,690 • 7,582,216 • 7,666,303 | Australia Patent No. 693,164 • 707,133 • 729,096 • 779,401 • 289,647 • 2008,279,378 • 2008,288,900 |  
 Canadian Patent No. 2,009,280 • 2,137,942 • 2,175,277 • 2,180,305 • 2,180,383 • 2,206,338 • 2,327,768 | Indonesian Patent No. 007058 | Japan Patent No. 3581233 • 9-11476 |  
 Korea Patent No. 10-1451593 • 0519212 | Malaysia Patent No. 118987 | New Zealand Patent No. 314,646 • 583,583 • 583,008 | South African Patent No. 2010/00683 • 2010/01796 |



## Stormceptor®

I:\IMBRIUM\PRODUCTS\STORMCEPTOR STC300.DWG & DETAILS\STANDARD DETAILS\METRIC\STC 300.DWG 4/12/2019 1:21 PM

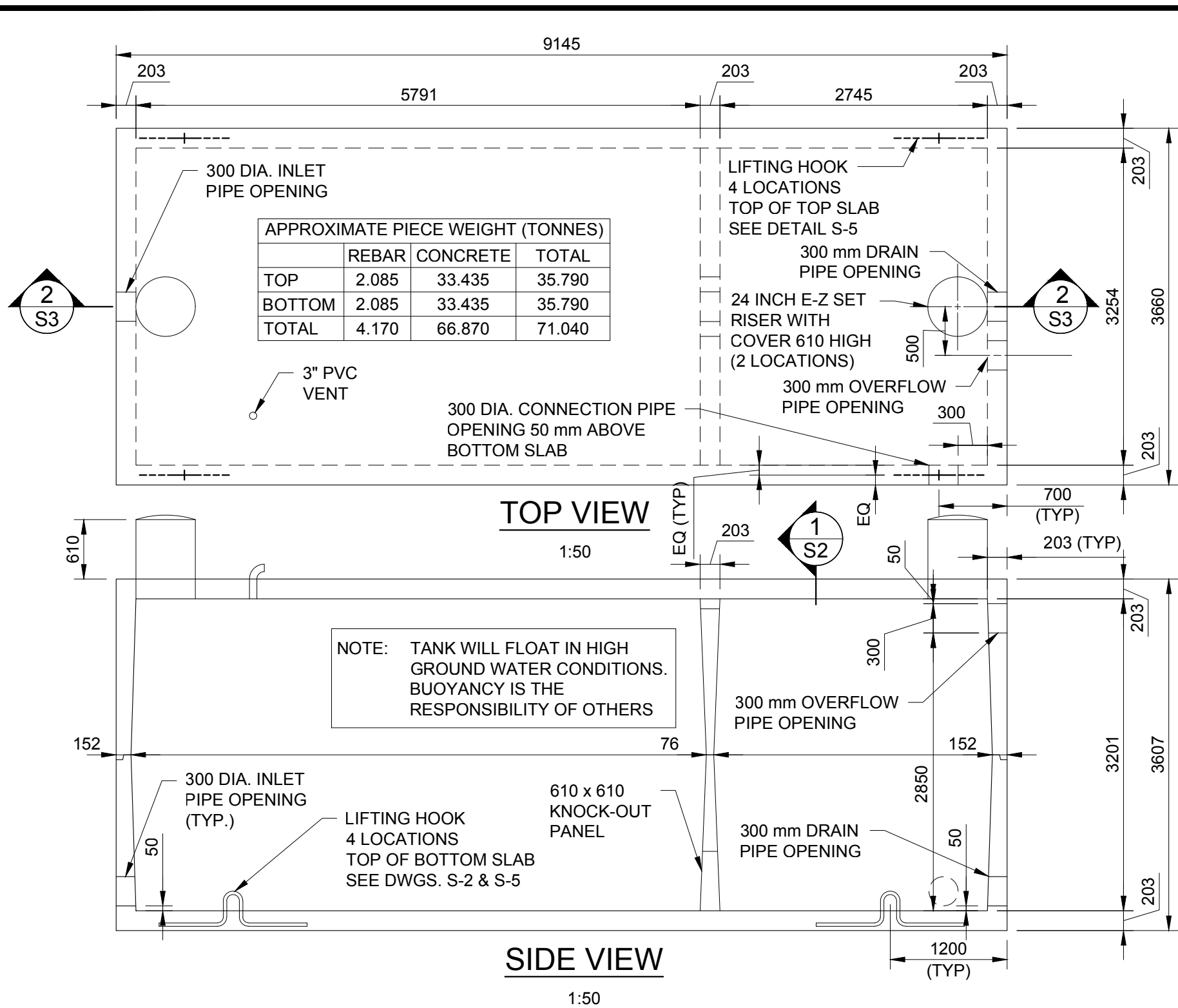
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**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**C.5 TANK SPECIFICATIONS**



**GENERAL NOTES:**

1. ALL DIMENSIONS ARE IN MILLIMETERS UNLESS NOTED OTHERWISE.
2. PRECAST CONCRETE TANK DESIGNED TO 2012 ONTARIO BUILDING CODE CSA B66-16. BEDDING, WATERPROOFING, BACKFILL, AND ALL OTHER SITE WORK BY OTHERS.
3. CONCRETE WORK TO BE IN ACCORDANCE WITH CAN/CSA A23.1 CONCRETE MATERIALS AND METHODS OF CONCRETE CONSTRUCTION, AND CAN/CSA A23.4 PRECAST CONCRETE MATERIALS AND CONSTRUCTION.
4. BOX DESIGNED FOR A MAXIMUM FILL HEIGHT OF 760 mm WITH 12 kPa LIVE LOAD. DESIGN FILL COVER ON THIS TANK IS 610 mm.
5. CONCRETE TO BE 45 MPa SCC WITH 600 mm ±70 mm SLUMP.
6. CONCRETE COVER AS NOTED, WITH ±10 mm TOLERANCE.
7. REINFORCING STEEL TO BE GRADE 400W BLACK DEFORMED BARS CONFORMING TO CAN/CSA G30.18.
8. DO NOT LIFT UNITS UNTIL CONCRETE HAS REACHED A MINIMUM STRENGTH OF 25 MPa, AS DETERMINED BY COMPRESSIVE TESTING OF CONCRETE CYLINDERS CURED IN SIMILAR CONDITIONS.
9. GROUT TO BE NON-SHRINK CEMENTITIOUS GROUT WITH MINIMUM COMPRESSIVE STRENGTH OF 45 MPa.
10. JOINT SEAL TO BE CONSEAL CS-102 BUTYL RUBBER SEALANT (CONFORMS TO AASHTO M-198B AND ASTM C-990-91). STORE, HANDLE AND APPLY JOINT SEALS IN STRICT ACCORDANCE WITH MANUFACTURER PRODUCT DATA SHEETS.
11. IT IS STRUCTURALLY IMPORTANT THAT THE SEALS IN THE HORIZONTAL JOINTS BE INSTALLED CORRECTLY. TOP PIECES MUST BE INSTALLED WITHOUT SLIDING THE PIECES ON THE SEALS.
12. DESIGN BASED ON GRANULAR BEDDING AND BACKFILL COMPACTED TO 95% SPDD.
13. LIFTING INSERTS TO BE GROUTED ON SITE BY OTHERS.
14. DELIVERY IS MADE BY CRANE-EQUIPPED TRUCKS.
15. EXCAVATION MUST BE READY, SAFE AND ACCESSIBLE FOR UNLOADING FROM THE REAR OF THE TRUCK.
16. MINIMUM OVERHEAD CLEARANCE OF 18 FT. IS REQUIRED.
17. ALL UNITS MUST BE HANDLED WITH PROPER LIFTING EQUIPMENT (i.e. SPREADER BARS).

APPROXIMATE PIECE WEIGHT (TONNES)			
	REBAR	CONCRETE	TOTAL
TOP	2.085	33.435	35.790
BOTTOM	2.085	33.435	35.790
TOTAL	4.170	66.870	71.040

NOTE: TANK WILL FLOAT IN HIGH GROUND WATER CONDITIONS. BUOYANCY IS THE RESPONSIBILITY OF OTHERS

5598 POWER ROAD  
OTTAWA, ONTARIO  
TEL: 613-822-1488

Version	Description	YYYY-MM-DD
1	ISSUED FOR REVIEW	2018-08-31

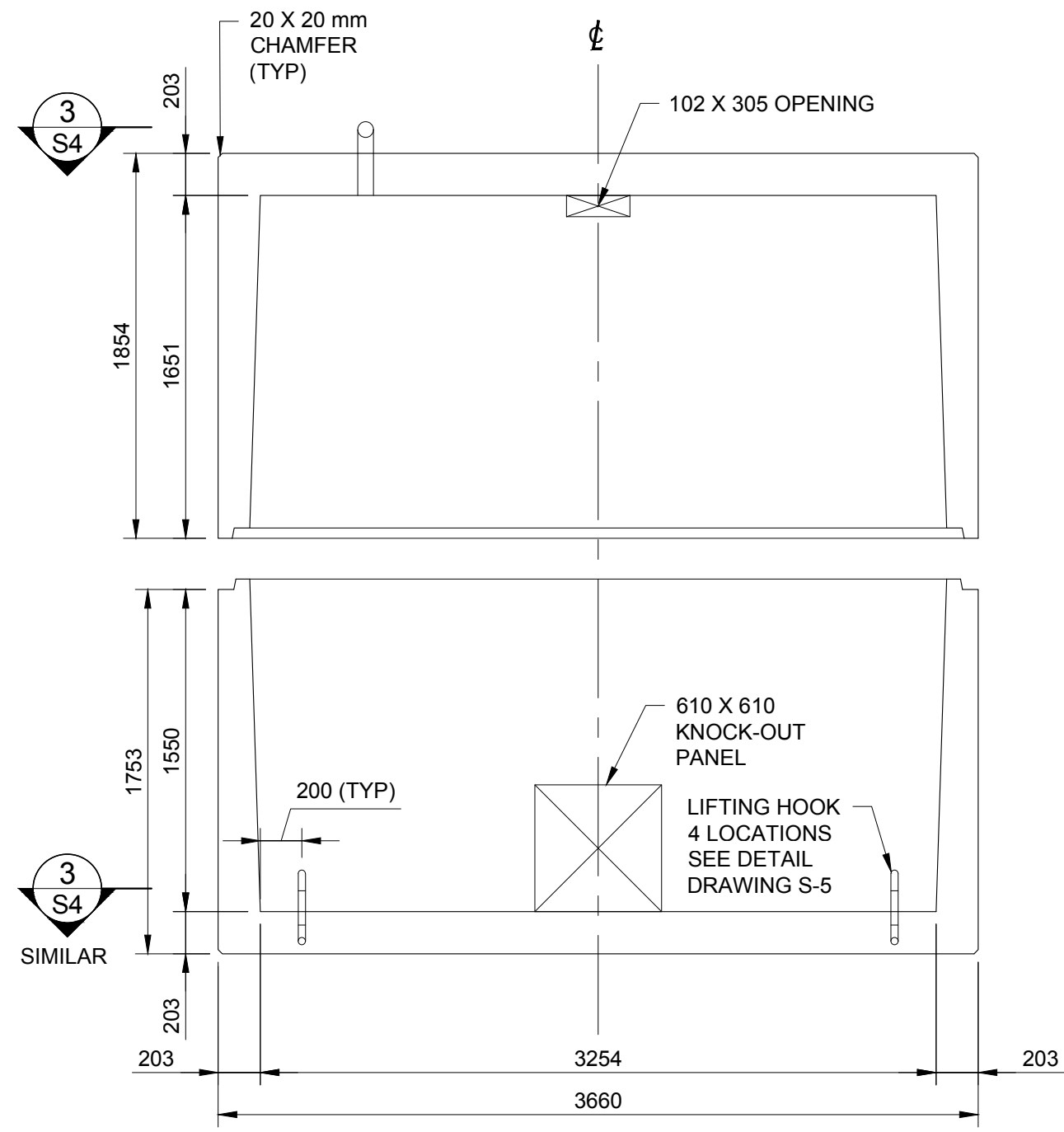
Stamp

WELLINGS OF STITTSVILLE

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MH JOB NO.: 2170737.34

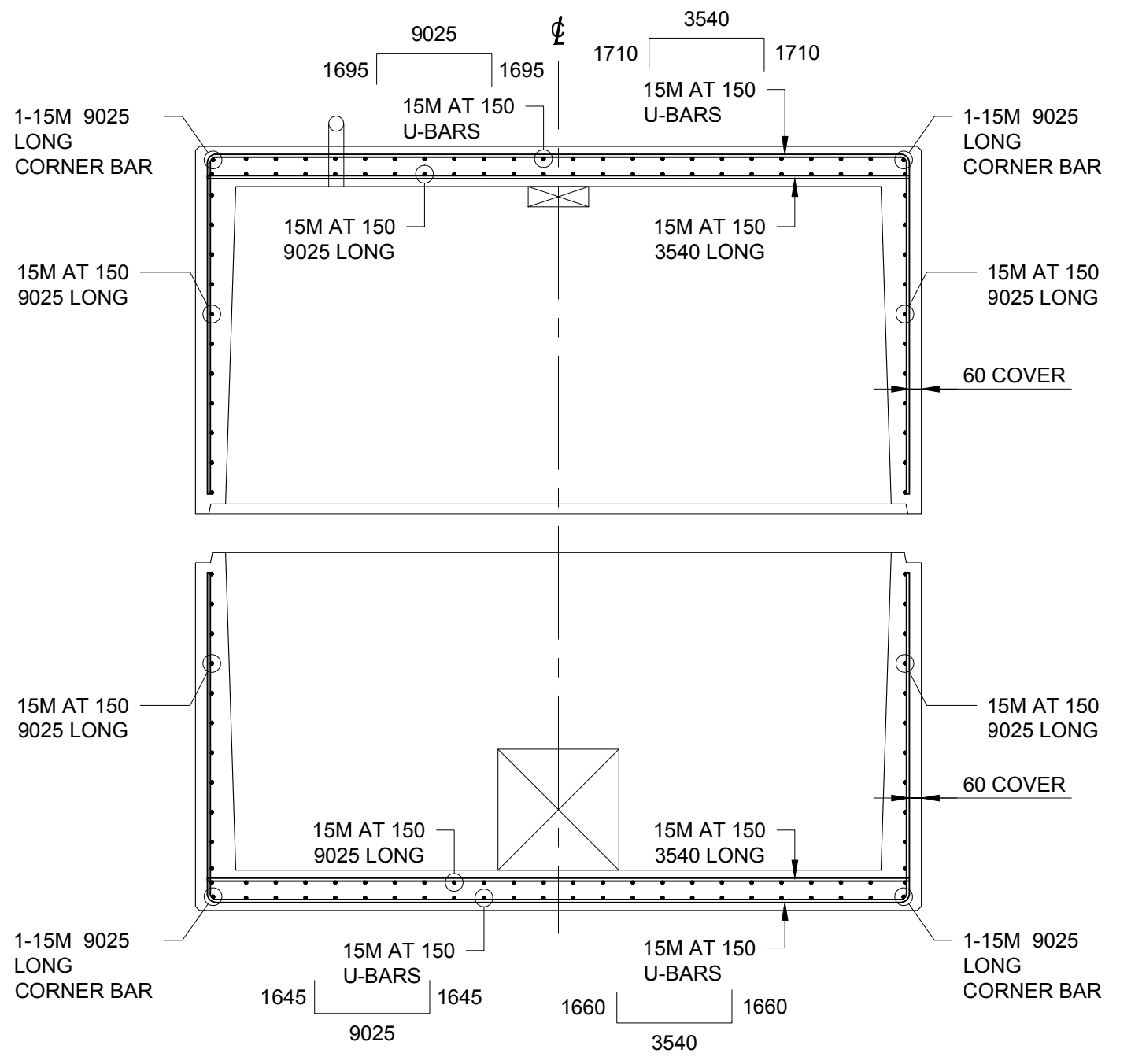
TANK DIMENSIONS		
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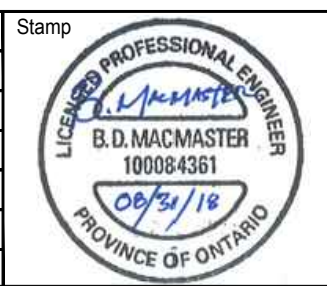


**SECTION 1**  
1:30



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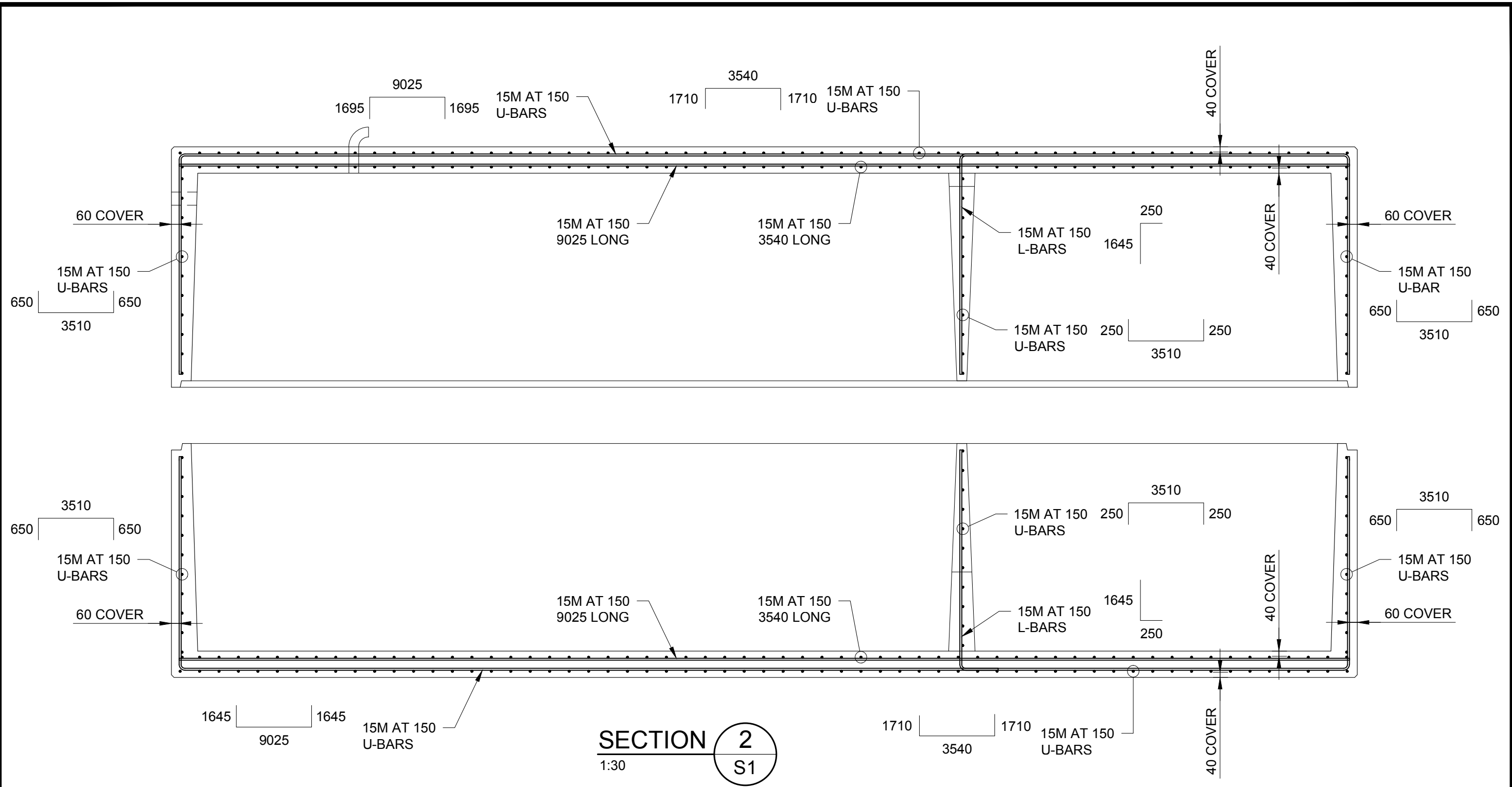


WELLINGS OF STITTSVILLE

PPS JOB NO.: 2018-0803  
MH JOB NO.: 2170737.34

SECTIONS		
Scale AS NOTED		
DWG No. S-2	Version 1	Date 2018-08-28

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**MORRISON HERSHFIELD**

**POWER PRECAST SOLUTIONS**

5598 POWER ROAD  
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Version	Description	YYYY-MM-DD
1	ISSUED FOR REVIEW	2018-08-31



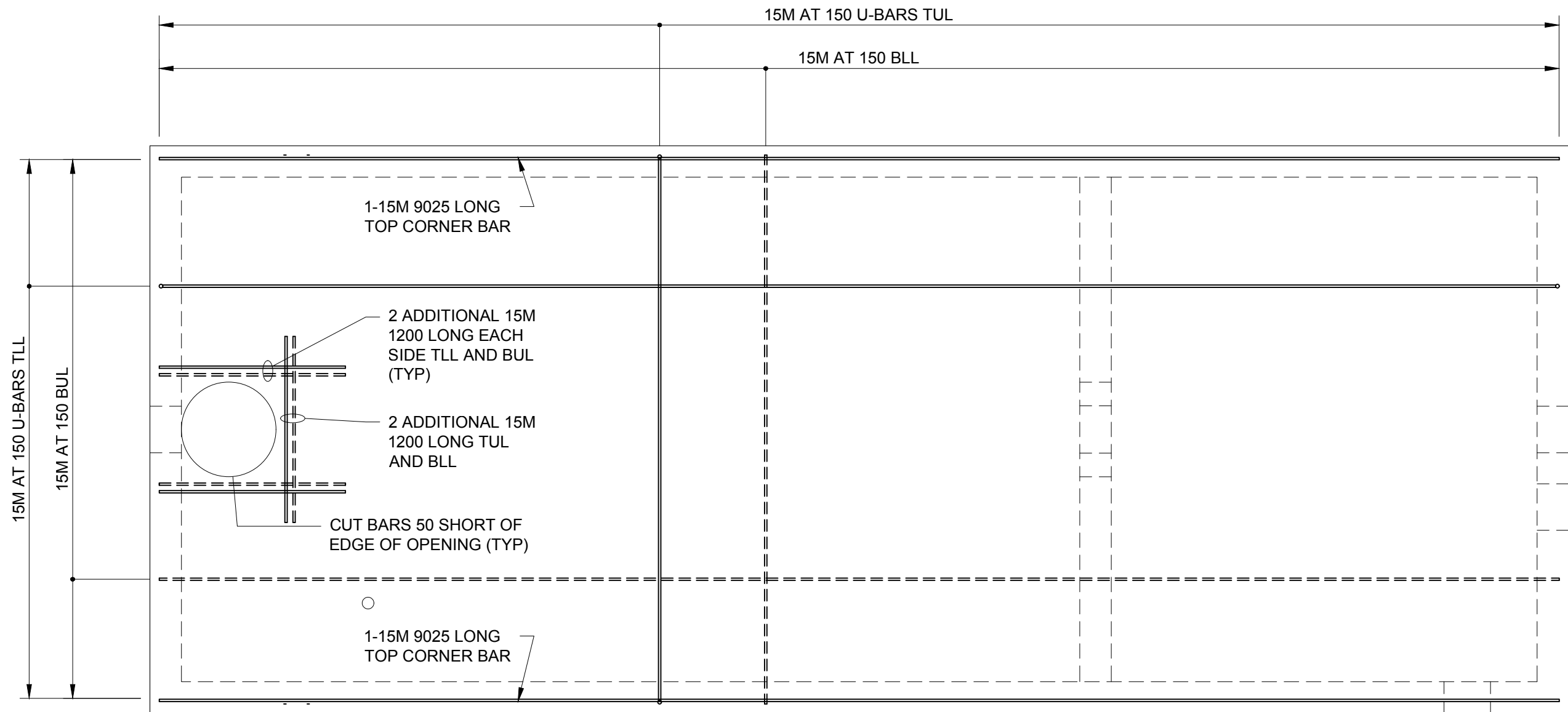
WELLINGS OF STITTSVILLE

PPS JOB NO.: 2018-0803  
MH JOB NO.: 2170737.34

SECTIONS		
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DWG No. S-3	Version 1	Date 2018-08-28

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SECTION 3  
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LEGEND

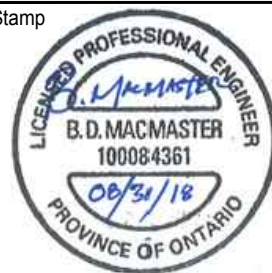
- DENOTES TOP MAT BARS
- ===== DENOTES BOTTOM MAT BARS
- TUL TOP UPPER LAYER
- TLL TOP LOWER LAYER
- BUL BOTTOM UPPER LAYER
- BLL BOTTOM LOWER LAYER



5598 POWER ROAD  
OTTAWA, ONTARIO  
TEL: 613-822-1488

Version	Description	YYYY-MM-DD
1	ISSUED FOR REVIEW	2018-08-31

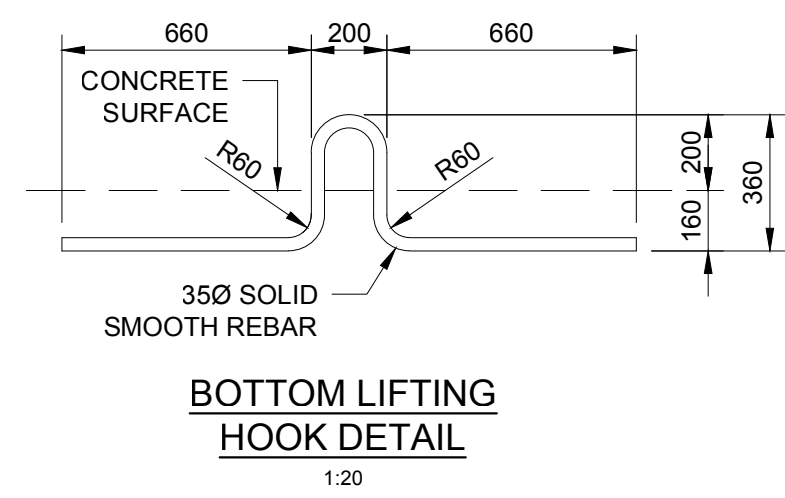
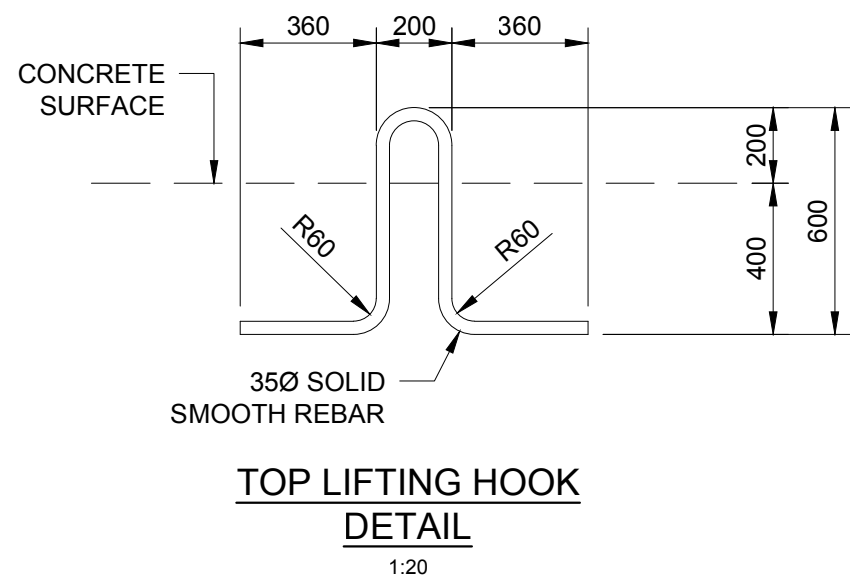
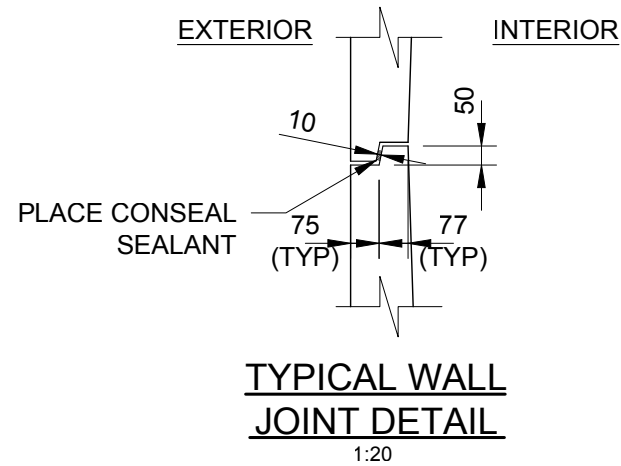
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WELLINGS OF STITTSVILLE

PPS JOB NO.: 2018-0803  
MH JOB NO.: 2170737.34

SECTIONS		
Scale AS NOTED		
DWG No. S-4	Version 1	Date 2018-08-28



			Stamp 	WELLINGS OF STITTSVILLE	<b>DETAILS</b>		
					Scale AS NOTED		
5598 POWER ROAD OTTAWA, ONTARIO TEL: 613-822-1488		1 ISSUED FOR REVIEW 2018-08-31	PPS JOB NO.: 2018-0803 MH JOB NO.: 2170737.34	DWG No. S-5	Version 1	Date 2018-08-28	
		Version Description YYY-MM-DD					

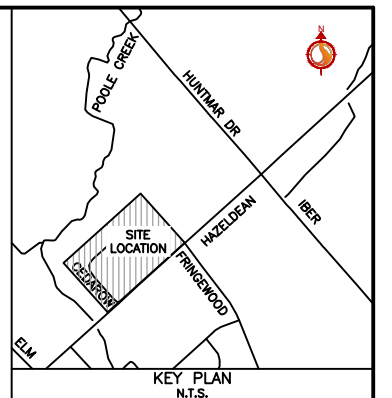
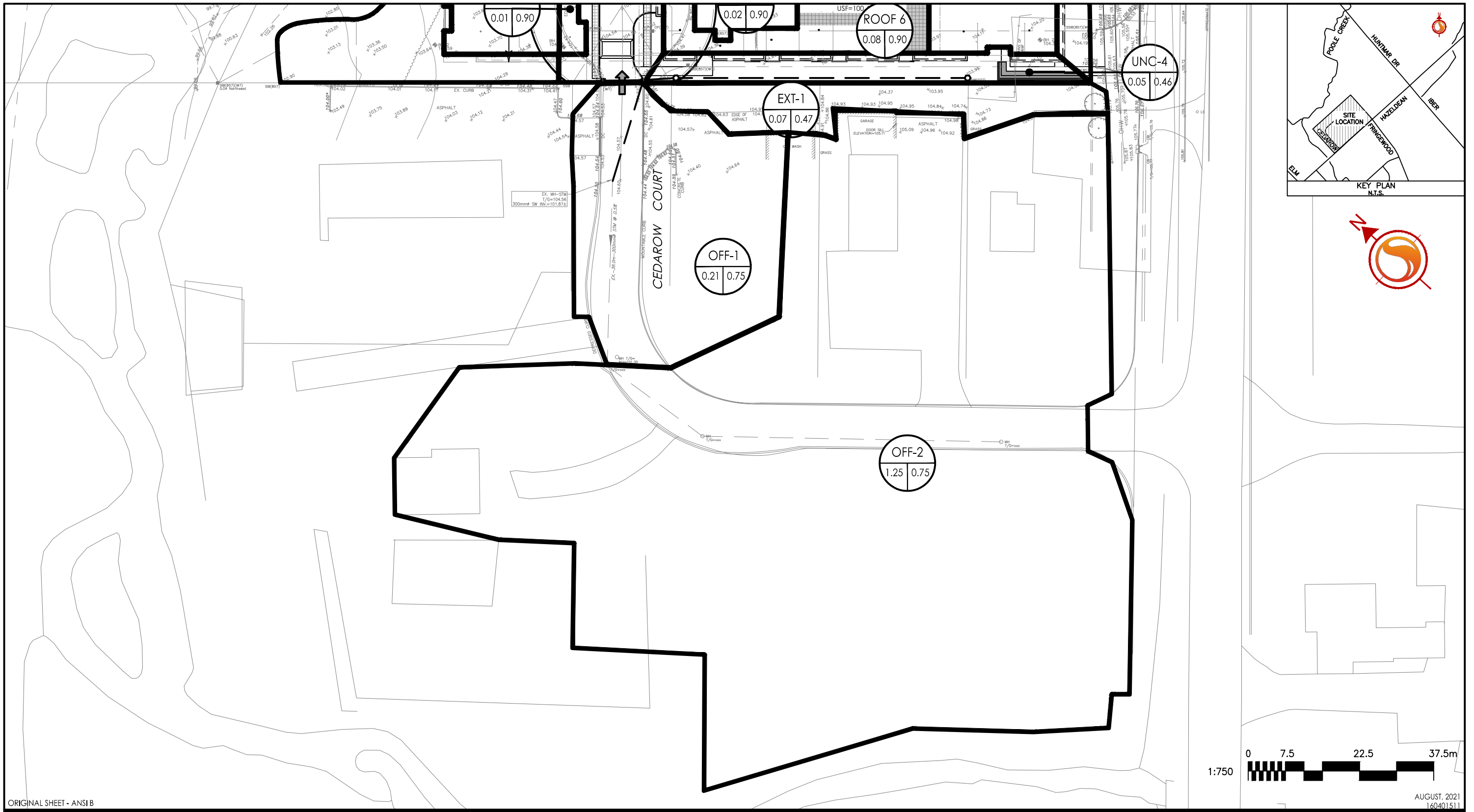
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**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**C.6 CEDAROW COURT STORM SEWER CAPACITY**

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 2021/08/31 7:53 AM By: Sharp, Mike








AUGUST, 2021  
 160401511

ORIGINAL SHEET - ANSI B

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

**Legend**

-  AREA ID
-  RUNOFF COEFFICIENT
-  STORM DRAINAGE AREA ha.
-  STORM DRAINAGE BOUNDARY
-  DIRECTION OF OVERLAND FLOW

Notes

Client/Project  
 NAUTICAL LANDS GROUP  
 WELLINGS OF STITTSVILLE PH 2  
 20 CEDAROW COURT

Figure No.  
 1.0

Title  
 CEDAROW COURT  
 EXISTING STORM DRAINAGE PLAN



Cedarow Court

**STORM SEWER  
DESIGN SHEET  
(City of Ottawa)**

DESIGN PARAMETERS

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

	1:2 yr	1:5 yr	1:10 yr	1:100 yr	
a =	732.951	998.071	1174.184	1735.688	MANNING'S n = 0.013
b =	6.199	6.053	6.014	6.014	MINIMUM COVER: 2.00 m
c =	0.810	0.814	0.816	0.820	TIME OF ENTRY 10 min

BEDDING CLASS = B

DATE: 2021-09-01

REVISION: 1  
DESIGNED BY: TR  
CHECKED BY: -

FILE NUMBER: 160401511

LOCATION			DRAINAGE AREA																PIPE SELECTION																					
AREA ID NUMBER	FROM M.H.	TO M.H.	AREA (2-YEAR) (ha)	AREA (5-YEAR) (ha)	AREA (10-YEAR) (ha)	AREA (100-YEAR) (ha)	AREA (ROOF) (ha)	C (2-YEAR) (-)	C (5-YEAR) (-)	C (10-YEAR) (-)	C (100-YEAR) (-)	A x C (2-YEAR) (ha)	ACCUM AxC (2YR) (ha)	A x C (5-YEAR) (ha)	ACCUM AxC (5YR) (ha)	A x C (10-YEAR) (ha)	ACCUM AxC (10YR) (ha)	A x C (100-YEAR) (ha)	ACCUM AxC (100YR) (ha)	T of C (min)	I <sub>2</sub> -YEAR (mm/h)	I <sub>5</sub> -YEAR (mm/h)	I <sub>10</sub> -YEAR (mm/h)	I <sub>100</sub> -YEAR (mm/h)	Q <sub>CONTROL</sub> (L/s)	ACCUM. Q <sub>CONTROL</sub> (L/s)	Q <sub>ACT</sub> (CIA/360) (L/s)	LENGTH (m)	PIPE WIDTH OR DIAMETE (mm)	PIPE HEIGHT (mm)	PIPE SHAPE (-)	MATERIAL (-)	CLASS (-)	SLOPE (%)	Q <sub>CAP</sub> (FULL) (L/s)	% FULL (-)	VEL. (FULL) (m/s)	VEL. (ACT) (m/s)	TIME OF FLOW (min)	
UNC-4 + EXT-1	CB507	EX1	0.00	0.12	0.00	0.00	0.00	0.00	0.47	0.00	0.00	0.000	0.000	0.056	0.056	0.000	0.000	0.000	0.000	0.000	10.00	76.81	104.19	122.14	178.56	0.0	0.0	16.2	21.3	250	250	CIRCULAR	CONCRETE	-	0.50	42.7	37.91%	0.86	0.68	0.52
OFF-1	EX1	EX2	0.00	0.21	0.00	0.00	0.00	0.00	0.75	0.00	0.00	0.000	0.000	0.154	0.210	0.000	0.000	0.000	0.000	0.000	10.52	74.85	101.50	118.97	173.90	0.0	0.0	59.2	39.0	300	300	CIRCULAR	CONCRETE	-	0.50	68.0	87.02%	0.97	0.98	0.67
OFF-2	EX2	POOLE CREEK	0.00	1.25	0.00	0.00	0.00	0.00	0.75	0.00	0.00	0.000	0.000	0.937	1.147	0.000	0.000	0.000	0.000	0.000	11.19	72.52	98.30	115.20	168.37	0.0	0.0	313.1	86.8	525	525	CIRCULAR	CONCRETE	-	0.62	353.3	88.62%	1.58	1.61	0.90

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
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**C.7 EXCERPTS FROM WOS PHASE 1**

## SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD

Stormwater Management  
March 22, 2017

### 5.0 STORMWATER MANAGEMENT

#### 5.1 OBJECTIVES

The objective of this stormwater management plan is to determine the measures necessary to control the quantity of stormwater released from the proposed development to established criteria, and to provide sufficient detail for approval and construction. The proposed development will discharge treated and controlled stormwater runoff to Poole Creek.

#### 5.2 SWM CRITERIA AND CONSTRAINTS

Criteria were established by combining current design practices outlined by the City of Ottawa Design Guidelines (2012), Ministry of Environment and Climate Change (MOECC) and Mississippi Valley Conservation Authority (MVCA). The following summarizes the criteria, with the source of each criterion indicated in italics:

##### General

- Use of the dual drainage principle (City of Ottawa)
- Wherever feasible and practical, site-level measures should be used to reduce and control the volume and rate of runoff (City of Ottawa)
- Site-level infiltration measures to be implemented to meet infiltration criteria of minimum 50 mm/yr (MVCA)
- Assess impact of 100 year event outlined in the City of Ottawa Sewer Design Guidelines, and climate change scenarios with a 20% increase of rainfall intensity, on major & minor drainage system (City of Ottawa)
- Quality control to be provided for 80% TSS removal (MVCA, MOECC)
- Site discharge to be controlled to pre-development rates (MVCA, City of Ottawa)
- Site design to mitigate erosion impacts on Poole Creek (City of Ottawa)

##### Storm Sewer & Inlet Controls

- Size storm sewers to convey the 5 year storm event under free-flow conditions using City of Ottawa I-D-F parameters (City of Ottawa) with the exception of the outlet sewer from the proposed underground storage facility.
- Minimum sewer inlet capture rates to be set such that no ponding occurs at the end of the 5-year event (City of Ottawa)
- Hydraulic Grade Line (HGL) analysis to be conducted using the 100 year 12 hour SCS storm distribution (City of Ottawa).
- 100-year Storm HGL to be a minimum of 0.30 m below building foundation footing otherwise foundation drains will be pumped (City of Ottawa)

## SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD

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March 22, 2017

### Surface Storage & Overland Flow

- Building openings to be a minimum of 0.30m above the 100-year water level (City of Ottawa)
- Maximum depth of flow under either static or dynamic conditions shall be less than 0.30m (City of Ottawa)
- Subdrains required in swales where longitudinal gradient is less than 1.5% (City of Ottawa)
- Provide adequate emergency overflow conveyance off-site (City of Ottawa)

#### 5.2.1 Pre-Development Conditions

A lumped catchment PCSWMM model was created for the subject site based on a site area of 2.9ha, and utilizing an existing SCS curve number of 80 per background documents. Additional subcatchment parameters were defined based upon recent topographical survey of the property:

Area (ha)	Width (m)	Slope (%)	Imperv. (%)	Subarea Routing
2.90	161.1	1.0	0.0	Outlet

Based on the above and during the 2 through 100-year 12hr SCS events (MTO Distribution curves), peak pre-development outflow rates from the subject site were identified per the tables below:

Storm Event	2-Year	5-Year	10-Year
Peak Outflow Rate	17.7L/s	43.9L/s	66.2L/s

Storm Event	25-Year	50-Year	100-Year
Peak Outflow Rate	103.7L/s	136.5L/s	176.3L/s

PCSWMM model input and output files for the predevelopment scenario are included within **Appendix C**.

## 5.3 STORMWATER MANAGEMENT DESIGN

### 5.3.1 Rationale for Design and Servicing Deviations

#### 5.3.1.1 Deviation from Kanata West MSS

Per the findings of the Kanata West MSS, stormwater outflows from the proposed site were intended to be directed to the storm sewer within Huntmar Drive, and in turn directed to the



## **SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD**

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downstream Fairwinds temporary pond 5. The MSS had assumed that the entire area of land west of Huntmar Drive and bound by Poole creek to the north and Hazeldean Road to the south was to be directed to the Huntmar Drive sewer, however, the proposed site forms only part of the tributary area, within lands owned by others blocking direct access to the storm sewer within Huntmar Drive. Rather than encumbering the adjacent property, and to avoid considerable connection fees associated with the outlet from the Kanata West Owners Group (KWOG), a separate outlet for the site to Poole Creek has been considered. As the downstream Pond 5 discharges to Poole Creek as well, by restricting flows to predevelopment levels, and assessing the erosive potential of such flows for the Poole Creek reach between the site outlet and that of the downstream Pond 5, no deleterious effects to the downstream watercourse are expected. Additionally, this option provides additional potential to supplement baseflows to Poole Creek in accordance with recommendations from the MVCA.

### **5.3.1.2 Deviation from Standard SWM Design**

The proposed SWM design includes three LID measures to encourage on-site infiltration and water re-use for irrigation. It is recognized that these measures are not currently standard SWM controls and when they are used for water balance purposes are not traditionally included in SWM calculations due to concerns over longterm reliability. The proposed SWM design has included some of the storage and infiltration/reuse rates from these measures in the supporting analysis as discussed in the following sections. However the analysis has also included simulations assuming that these measures fail in order to assess the potential associated impacts. The benefit of including some of the storage and infiltration losses associated with the LID measures was that the end-of-pipe underground storage component of the infiltration gallery was able to be reduced by 30% as compared to previous design requirements when no credit was assigned for the LID measures. As discussed later in this report, a monitoring plan will be developed and implemented to ensure that constructed LID measures are performing as designed.

### **5.3.2 Design Methodology**

The intent of the stormwater management plan presented herein is to mitigate negative impacts that the proposed development might have on the receiving watercourse (Poole Creek), while providing adequate capacity to service the proposed buildings, underground parking and access areas. The proposed stormwater management plan is designed to detain runoff on the rooftop, surface and in the subsurface (StormTech chamber) to ensure that peak flows after construction will not exceed the target discharge rates and erosion mitigation requirements.

Runoff from the site is captured via catchbasins, landscaping drains and roof drains and conveyed to a hydrodynamic separator for water quality treatment followed by an underground storage unit for quantity control. The storage unit is restricted by an ICD at the downstream end and is an open bottom unit designed to also promote infiltration. Roof runoff is controlled via roof drains discharging through the internal building plumbing to rainwater harvesting tanks. Two rainwater harvesting tanks are proposed for each building. Each rainwater

## SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD

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tank is capable of storing up to 91 m<sup>3</sup> of runoff (approximately 32mm of rainfall) beyond which it will overflow into the storm sewer and be conveyed to the storage unit. The underground storage unit is sized assuming that the rainwater harvesting tanks are available at the start of the rainfall event.

Additional infiltration will be achieved on-site through the implementation of a bioswale along the east side of the site. The granular subbase of the swale is sized to store runoff from its tributary area. An overflow drain is also provided to convey excess water to the underground storage unit.

The site discharge will be conveyed to the approved outlet location for the adjacent CMHC lands to the west of the subject site. The outlet will be sized to convey flows from both sites. Utilizing this location addresses concerns regarding an additional outlet to Poole Creek and prevents disturbance of the natural area to the north of the site.

### 5.3.3 Modeling Rationale

A comprehensive hydrologic modeling exercise was completed with PCSWMM, accounting for the estimated major and minor systems to evaluate the storm sewer infrastructure. The use of PCSWMM for modeling of the site hydrology and hydraulics allowed for an analysis of the systems response during various storm events. Surface storage estimates were based on the final grading plan design (see **Drawing GP-1**). The following assumptions were applied to the detailed model:

- Hydrologic parameters as per Ottawa Sewer Design Guidelines, including Horton infiltration, Manning's 'n', and depression storage values
- 12-hour SCS Storm distribution for the 100-year analysis to model 'worst-case' scenario in regards to on-site HGLs.
- 12hr SCS distributions (2 and 100-year events) with free flowing boundary condition to model 'worst-case' scenario in regards to site discharge rates to meet target rate.
- To 'stress test' the system a 'climate change' scenario was created by adding 20% of the individual intensity values of the 100-year SCS storm event at their specified time step.
- All LID measures were designed outside of PCSWMM (as documented in the report and calculations included in **Appendix E**) in order to allow routing of LID overflows to the next downstream LID which cannot be done in PCSWMM where an LID is defined as part of a given subcatchment. Total design storage and calculated infiltration losses were then input into PCSWMM as storage nodes with separate outlets for infiltration losses.
- Percent imperviousness calculated based on actual soft and hard surfaces on each subcatchment, converted to equivalent Runoff Coefficient using the relationship  $C = (\text{Imp.} \times 0.7) + 0.2$
- Subcatchment areas are defined from high-point to high-point where sags occur. Subcatchment width (average length of overland sheet flow) determined by dividing

## SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD

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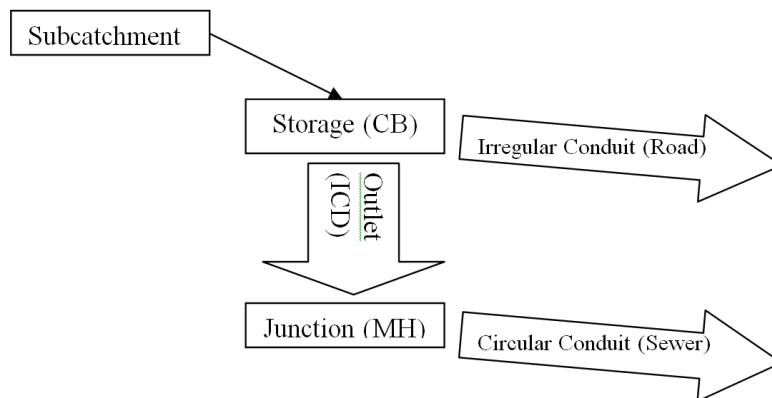
subcatchment area by subcatchment length (length of overland flow path measured from high-point to high-point).

- Number of catchbasins based on servicing plan (**Drawing SP-1**)
- Catchbasin inflow restricted with inlet-control devices (ICDs) as necessary to maintain inflow target rate and maximize use of surface storage where possible.
- Surface ponding in sag storage calculated based on grading plans (**Drawing GP-1**).

### 5.3.3.1 SWMM Dual Drainage Methodology

The proposed site is modeled in one modeling program as a dual conduit system (see **Figure 3**), with: 1) circular conduits representing the sewers & junction nodes representing manholes; 2) irregular conduits using street-shaped cross-sections to represent the sawtoothed overland road network from high-point to low-point and storage nodes representing catchbasins. The dual drainage systems are connected via outlet link objects (or orifices) from storage node (i.e. CB) to junction (i.e. MH), and represent inlet control devices (ICDs). Subcatchments are linked to the storage node on the surface so that generated hydrographs are directed there firstly.

**Figure 3: Schematic Representing Model Object Roles**



Storage nodes are used in the model to represent catchbasins as well as major system junctions. For storage nodes representing catchbasins (CBs), the invert of the storage node represents the invert of the CB and the rim of the storage node is the top of the CB plus the maximum above ground storage depth. An additional 0.3m has been added to rim elevations to allow routing from one surface storage to the next, and is unused where no spillage occurs between ponding areas. Ponding at low points is represented via storage area-depth curves for each individual storage node to match ponding volumes demonstrated on the grading plan **Drawing GP-1**. Storage volumes exceeding the sag storage available in the node will route through the connected irregular conduit to the next storage node and continue routing through the system until, ultimately, flows either re-enter the minor system or reach the outfall of the major system.

## SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD

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Inlet control devices, as represented by orifice links, use a user-specified discharge coefficient to approximate manufacturer's specifications for the chosen ICD model.

Subcatchment imperviousness was calculated via impervious area measured from **Drawing SSP-1**.

### 5.3.3.2 Boundary Conditions

The detailed PCSWMM hydrology and the proposed storm sewers were used to assess the peak inflows and hydraulic grade line (HGL) for the site. The elevation of the outlet sewer at MH100 immediately upstream of Poole Creek has been set conservatively to be above the 100-Year water elevation of the Creek per MVCA Flood Risk Mapping at an invert elevation of 99.7m to enable free-flowing model condition for the site outlet.

### 5.3.4 Input Parameters

**Drawing SD-1** summarizes the discretized subcatchments used in the analysis of the proposed site, and outlines the major overland flow paths. The grading plans are also enclosed for review.

Appendices A1 to A3 summarize the modeling input parameters and results for the subject area; an example input and output file are provided for the 100-year 12hr SCS storm. For all other input files and results of storm scenarios, please examine the electronic model files located on the CD provided with this report. This analysis was performed using PCSWMM, which is a front-end GUI to the EPA-SWMM engine. Model files can be examined in any program which can read EPA-SWMM files version 5.1.010.

#### 5.3.4.1 Hydrologic Parameters

**Table 4** presents the general subcatchment parameters used:

**Table 4: General Subcatchment Parameters**

Parameter	Value
Infiltration Method	Curve Number
Drying Time (days)	7
Curve Number	80
N Impervious	0.013
N Pervious	0.2
Dstore Imperv. (mm)	1.57
Dstore perv. (mm)	4.67
Zero Imperv. (%)	0

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**Table 5** presents the individual parameters that vary for each of the proposed subcatchments.

**Table 5: Subcatchment Parameters**

Name	Outlet	Area (ha)	Width (m)	Slope (%)	Imperv. (%)
EXT1	EXT1-OF	0.07	15.1	33.3	0
EXT2	EXT2-OF	0.06	14.4	2	72.857
ST104A	ST104A-S	0.15	69	2	84.286
ST107A	ST107A-S	0.37	225.0	1.5	64.286
ST108A	ST108A-S	0.40	90.9	1.5	100
ST108B	ST108B-S	0.36	82.0	1.5	100
ST108C	ST108C-S	0.05	12.1	1.5	100
ST108D	ST108D-S	0.05	10.9	1.5	100
ST108E	ST108E-S	0.03	25.0	1.5	100
ST108F	108	0.38	86.0	1.2	44.286
ST109A	109	0.01	18.2	10	100
ST109C	ST109C-S	0.06	25.8	1	100
ST109B	ST109B-S	0.05	24.8	1	100
ST110A	110	0.07	16.8	0.8	7.143
ST110B	110	0.03	24.5	10	100
ST110C	110	0.03	26.6	10	100
ST110D	110	0.07	16.7	0.8	7.143
ST111A	ST111A-S	0.24	107.5	0.8	72.857
ST111B	ST111B-S	0.04	88.0	0.8	100
ST111C	ST111C-S	0.04	36.8	1.5	85.714
ST507A	ST507A-S	0.05	33.5	1.5	72.857
ST508A	508	0.34	189.2	1	7.143

**Table 6** summarizes the storage node parameters used in the model. Storage curves for each node have been created based on volumes presented for each individual ponding area within **Drawing GP-1**. Rim elevations for each node correspond to the rim elevation of the associated area's catchbasin plus maximum depth of storage plus 0.30m to allow for demonstration of overland flow in the climate change event scenario. The 0.30m buffer is unused during other modeled events.

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Storage volumes and release rates for the rainwater harvesting tank, bioswale/rain garden, and infiltration basin were obtained through iterations between design sizing calculations (final sizing attached in **Appendix E**) and PCSWMM hydrologic/hydraulic modeling.

**Table 6: Storage Node Parameters**

Name	Invert El. (m)	Rim Elev. (m)	Depth (m)	Coefficient	Exponent	Constant (m <sup>2</sup> )	Curve Name	Storage Curve
108	99.27	104.37	5.10	0	0	0	RWhtank	TABULAR
508	101.06	102.85	1.79	0	0	0	ST508A-S	TABULAR
ST104A-S	101.52	103.62	2.1	1000	0	0	ST104A-S	TABULAR
ST107A-S	101.13	103.23	2.1	1000	0	0	ST107A-S	TABULAR
ST108A-S	118.6	118.75	0.15	1000	0	0	ST108A	TABULAR
ST108B-S	115.75	115.9	0.15	1000	0	0	ST108B	TABULAR
ST108C-S	110.4	110.55	0.15	1000	0	0	ST108C	TABULAR
ST108D-S	110.1	110.25	0.15	1000	0	0	ST108D	TABULAR
ST108E-S	107.2	107.35	0.15	1000	0	0	ST108E	TABULAR
ST109B-S	102.81	104.31	1.5	0	0	0	*	FUNCTIONAL
ST109C-S	102.81	104.31	1.5	0	0	0	*	FUNCTIONAL
ST111A-S	101.86	104.26	2.4	1000	0	0	ST111A-S	TABULAR
ST111C-S	101.95	104.05	2.1	0	0	0	*	FUNCTIONAL
ST507A-S	101.57	103.67	2.1	1000	0	0	ST504A-S	TABULAR
TANK	100.10	103.37	3.27	1000	0	0	TANK	TABULAR

### 5.3.4.2 Hydraulic Parameters

As per the Ottawa Sewer Design Guidelines (OSDG 2012), Manning's roughness values of 0.013 were used for sewer modeling and overland flow corridors representing roadways.

Storm sewers were modeled to confirm flow capacities and hydraulic grade lines (HGLs) in the proposed condition. The detailed storm sewer design sheet is included in **Appendix C**.

**Table 7** below presents the parameters for the orifice and outlet link objects in the model, which represent ICDs and restricted roof release drains respectively. CB leads modeled as orifices were assigned a discharge coefficient of 0.65. The roof release discharge curves assume the use of standard Zurn model Z-105-5 controlled release roof drains as noted in the calculation sheet in **Appendix C**. The number of roof notches for each building area is to be confirmed with the

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
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building mechanical engineer. Details for the IPEX ICDs and Zurn drains are included as part of **Appendix G**.

**Table 7: Outlet/Orifice Parameters**

Name	Inlet	Outlet	Inlet Elev.	Type	Diameter
OR1	TANK	102	100.10	CIRCULAR	0.11
OR2	TANK	102	100.70	CIRCULAR	0.15
OR3	TANK	102	101.00	CIRCULAR	0.15
ST104A-O	ST104A-S	104	101.52	IPEX HF	0.140
ST107A-O	ST107A-S	107	101.13	CIRCULAR	0.2
ST109B-O	ST109B-S	109	102.81	CIRCULAR	0.2
ST109C-O	ST109C-S	109	102.81	CIRCULAR	0.2
ST111A-O	ST111A-S	111	101.86	IPEX HF	0.076
ST111C-O	ST111C-S	111	101.95	CIRCULAR	0.2
ST111C-O1	ST111C-S	111	101.95	CIRCULAR	0.2
OL-1	TANK	P_OF1	100.10	0.66L/s	-
OL-2	508	P_OF2	101.06	0.3L/s	-
ST507A-O	ST507A-S	TANK	101.57	IPEX LMF 95	-
ST108A-O	ST108A-S	108	118.60	ROOF	-
ST108B-O	ST108B-S	108	115.75	ROOF	-
ST108C-O	ST108C-S	108	110.40	ROOF	-
ST108D-O	ST108D-S	108	110.10	ROOF	-
ST108E-O	ST108E-S	108	107.20	ROOF	-

### 5.3.5 Model Results

The following section summarizes the key hydrologic and hydraulic model results. For detailed model results or inputs please refer to the example input file in **Appendix C.2 and C.3** and the electronic model files on the enclosed CD.

#### 5.3.5.1 Hydrologic Results

The following tables demonstrate the peak outflow from each modeled outfall during the design storm (12hr SCS 2-100yr) events. A free-flowing outfall condition has been modeled for these events to be conservative with respect to site peak release rates. Outfalls EXT1-OF and EXT2-OF denote uncontrolled flows from the perimeter of the site that, due to grading restrictions, are captured by the existing right-of-way/Poole Creek at the south and north boundaries of the site.

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Flows from area EXT2 will have a minimal contribution to the infrastructure within Hazeldean Road. Peaks from these uncontrolled flows are non-coincident with peaks from the subsurface storage tank/weir, and as such, flows from the outlet headwall are the only values considered in meeting the release rate target. The required subsurface storage tank volume was determined through iteration of each event, and sized to mirror the site release rate target.

**Table 8: Site Peak Discharge Rates**

Event	Location	Discharge Rate (L/s)	Target (L/s)
2-Year 12 Hour SCS	Outlet Headwall	15.2	17.7
5-Year 12 Hour SCS	Outlet Headwall	38.6	43.9
10-Year 12 Hour SCS	Outlet Headwall	64.5	66.2
25-Year 12 Hour SCS	Outlet Headwall	98.7	103.7
50-Year 12 Hour SCS	Outlet Headwall	116.2	136.5
100-Year 12 Hour SCS	Outlet Headwall	136.3	176.3
100-Year 12 Hour SCS +20%	Outlet Headwall	317.0	-

### 5.3.5.2 Hydraulic Results

**Table 9** summarizes the HGL results within the site for the 100 year storm events and the 'climate change' scenario storm required by the City of Ottawa Sewer Design Guidelines (2012), where intensities are increased by 20%. The City of Ottawa requires that during major storm events, the maximum hydraulic grade line be kept at least 0.30 m below the underside-of-footing (USF) of any adjacent units connected to the storm sewer during design storm events. As the proposed building perimeter drain and ramp drains will be disconnected from the storm sewer and pumped to the surface, USFs are considered at 0.3m below the lowest finished floor elevation of the building.

**Table 9: Modeled Hydraulic Grade Line Results**

STM MH	Proposed Ground Elev. (m)	100-year 12hr SCS		100-year 12hr SCS + 20%	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
103	104.20	101.97	2.23	102.78	1.42
104	104.20	101.98	2.22	102.80	1.40
105	104.20	101.99	2.21	102.86	1.34
106	104.20	101.99	2.21	102.87	1.33
107	104.20	102.00	2.20	102.89	1.31



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STM MH	Proposed Ground Elev. (m)	100-year 12hr SCS		100-year 12hr SCS + 20%	
		HGL (m)	USF-HGL Clearance (m)	HGL (m)	USF-HGL Clearance (m)
108	104.20	102.04	2.16	103.04	1.16

As is demonstrated in the table above, the worst-case scenario results in HGL elevations remain at least 0.30 m below the proposed surface elevations, and HGL elevations remain below the proposed surface elevations during the 20% increased intensity 'climate change' scenario.

**Table 10** presents the maximum total surface water depths (static ponding depth + dynamic flow) above the top-of-grate of catchbasins for the 100-year design storm and climate change storm. Based on the model results, the total ponding depth (static + dynamic) does not exceed the required 0.30m maximum during the 100-year event. Total ponding depths during the climate change scenario are below adjacent building openings and should not impact the proposed building.

**Table 10: Maximum Surface Water Depths**

Storage node ID	Structure ID	Rim Elevation (m)	100 year, 12hr SCS		100 year, 12hr SCS +20%	
			Max HGL (m)	Total Surface Water Depth (m)	Max HGL (m)	Total Surface Water Depth (m)
ST104A-S	CB 506	103.32	103.23	0.00	103.47	0.15
ST107A-S	CB 505	102.93	102.85	0.00	103.09	0.16
ST111A-S	CB 501	103.96	104.18	0.22	104.22	0.26
ST111C-S	CB 504	103.75	102.03	0.00	102.91	0.00
ST507A-S	CB 507	103.37	103.44	0.07	103.47	0.10
508	CB T 508	102.60	102.01	0.00	102.83	0.23

### 5.3.6 Water Quality Control

On-site water quality control is required to provide 80% TSS removal prior to discharging to Poole Creek. A Stormceptor unit STC300 is proposed upstream of the underground storage/infiltration basin. The Stormceptor will provide greater than 80% TSS removal in the 25mm event and will act as pre-treatment for the storage/ infiltration basin thereby reducing maintenance requirements of the facility and improving long-term performance. The Stormceptor unit will be privately maintained. The location and general arrangement of the Stormceptor unit is indicated on **Drawing SD-1**. Detailed sizing calculations for the Stormceptor unit are included in **Appendix C.5**

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### 5.3.7 Infiltration Targets

The MVCA requires that BMP measures be implemented on-site to meet the minimum infiltration target rate of 50 mm/yr (as identified in the Kanata West Master Servicing Study, Stantec, 2006). For a site area of 2.9ha with an average imperviousness of 56% the total annual infiltration requirement is therefore 812m<sup>3</sup>/yr. The KWMSS also requires a 25% augmentation to site infiltration requirements to account for off-site road areas for which no infiltration measures were required. Therefore, the total site infiltration target is 1,015 m<sup>3</sup>/yr. Past correspondence with the MVCA indicated that the target infiltration rates were in fact “target hydrograph volume reduction rates”.

The LID bioswale and infiltration gallery proposed for the site will provide significant opportunity for stormwater infiltration. Infiltration calculations completed for the design and sizing of these LID measures were used to approximate an expected annual infiltration rate. Water balance calculations for a continuous rainfall scenario from August 2, 2009 to March 1, 2012 (see **Appendix E**) were used to determine an average daily infiltration rate over a one year period. The average rate was estimated to be 44m<sup>3</sup>/day. Note that this rate is averaged over 365 days per year and would underestimate summer months and overestimate winter. Nevertheless, the average annual infiltration that could be provided through the LID measures would be approximately 16,262 m<sup>3</sup>/yr. Therefore, only about 10% of the total possible infiltration is required to meet the infiltration target for the site.

The infiltration contribution from the bioswale and infiltration gallery is included in **Table 11** below. Note that this summary does not include infiltration resulting from the rainwater harvesting reuse for irrigation. The results in **Table 11** suggest that the infiltration target could be met with the bioswale infiltration only.

**Table 11: Summary of Infiltration from LID Features**

LID Feature	Estimated Total Annual Infiltration (m <sup>3</sup> /yr)
Bioswale	2,568
Infiltration Gallery	13,694
<b>Total Infiltration</b>	<b>16,262</b>

#### 5.3.7.1 Potential Groundwater Mounding

Groundwater levels at the site were measured by Paterson Group during two separate site visits and are summarized in the attached Paterson memos in **Appendix F**. Based on the results of the

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
5731 HAZELDEAN ROAD**

Stormwater Management  
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groundwater monitoring Paterson Group prepared a memo discussing the variation in groundwater level measurements and anticipated seasonally high and normal groundwater levels. The results for the boreholes near the LID features are summarized in **Table 12** below. The complete memo dated January 25, 2017 is included in **Appendix F**.

**Table 12: Expected Seasonal Variation of Groundwater Levels**

Borehole Number	Ground Elevation (m)	Long-term Groundwater Levels		Seasonally High Groundwater Level	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
BH 1	102.93	3.7	99.23	3.2	99.73
BH 2	103.02	3.7	99.32	3.2	99.82
BH 3	103.07	3.7	99.37	3.2	99.87
BH 4	103.15	3.7	99.45	3.2	99.95
BH 5	103.22	3.7	99.52	3.2	100.02
BH 6	103.25	3.7	99.55	3.2	100.05
BH 7	102.91	4.5	98.41	4.0	98.91

Since the clearance from the bottom of the infiltration tank to the groundwater table is less than 1.0m the potential for groundwater mounding was considered. Groundwater mounding calculations were completed for both the seasonally high groundwater condition and the normal groundwater condition. However, per the Paterson memo, the seasonally high level is expected to occur during March-April, as such historical rainfall data was used to establish the average rainfall event volume for March-April. The analysis indicated approximately a 10mm event. The duration of infiltration for the infiltration gallery was obtained from the PCSWMM hydraulic model based on the modeled time for the infiltration gallery to empty. No PCSWMM model was run for the 10mm event so the 2-year event was used as a conservative estimate. These durations were input into the groundwater mounding calculation spreadsheet in **Appendix E**. It is noted that the calculations are based on the Hantush (1967) equation for groundwater mounding and use the hydraulic conductivity (measured by Paterson and summarized in the attached memo from September 2016) as the recharge rate and typical specific yield for silty clay. It is also noted that spreadsheet inputs and results are in imperial units. **Table 13** below summarizes the results of the groundwater mounding calculations.

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**Table 13: Estimated Maximum Groundwater Mounding below Infiltration Gallery**

Groundwater Condition	Mounding Height (m)	Mounding Elevation (m)	Distance to Bottom of Infiltration Gallery (m)
Long-term (99.23)	0.31	99.54	0.56
Seasonally High (99.73)	0.26	99.99	0.11

It is noted the above mounding depths are still below the bottom of the infiltration gallery. Should a larger rainfall even occur during the seasonally high groundwater condition there could be potential for the groundwater mound to extend into the infiltration gallery. However, there is a storm sewer outlet proposed at the bottom of the infiltration gallery (Invert =100.10m per attached **Drawing SSP-1**) which will limit the maximum groundwater height to the bottom of the infiltration gallery. Once the mounding reaches the bottom, the stored stormwater would discharge only through the controlled outlets and would not infiltrate. Since the groundwater mounding is caused only by infiltrating stormwater and not by external sources, there should be no loss of storage volume due to groundwater mounding.

### 5.3.8 Thermal Controls

The MVCA and MOECC confirmed that Poole Creek is designated as a “cool-water fish habitat”. As the proposed development will increase the amount of impervious area on the site and roof top detention will increase water temperatures, thermal mitigation measures are required for the site.

As the majority of heat transfer from paved surfaces occurs during the first flush (considered as the initial 10mm of each design event), storage of the 10mm event has been given priority. With exception of the rooftop areas, the site is designed with minimal surface storage. All runoff will be captured and detained in the underground storage unit which will allow for heat dumping into the surrounding ground and granular material. Similarly, runoff conveyed through the granular subbase of the bioswale will experience cooling. Roof discharge will be the most thermally impacted water as it will be retained on the rooftops for several hours. This water will be discharged to the underground rainwater harvesting tank and will inlet at the bottom of the tank such that if the tank is full, the cooler water will be discharged first through the overflow. With 167m<sup>3</sup> of storage available in each of the rainwater harvesting tanks, the only occurrence where roof discharge would not experience any temperature mitigation via mixing or detention would be when total rainfall exceeds approximately a 2-year event. The reverse temperature mitigation effect (warming water during cold weather) would also occur with these measures as ground temperatures would warm the runoff.

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### 5.3.9 Monitoring Plan

In addition to monitoring requirements to be identified by the MOECC in the Environmental Compliance Approval (ECA), the site will require regular monitoring of the LID measures installed on the site. A detailed monitoring program will be developed through consultation with the City of Ottawa and MVCA. In general, the monitoring plan will required pre-construction, during construction and post-construction monitoring and include the following:

- Installation of water level loggers in both the rainwater harvesting tank, infiltration gallery, and bioswale (monitoring “well” to be installed) to assess frequency of overflow and drawdown rates and compare with design values;
- Installation of temperature logger in the outlet manhole from the site to monitor temperature of the storm discharge. The temperature logger cannot be installed at the outlet to Poole Creek as this outlet will include discharge from the CMHC lands as well and would not be representative of the subject site;
- Collection of water quality samples upstream and downstream of the proposed OGS unit;
- Visual inspection of all LID systems at least once per month and following all large rainfall events. Including observations for:
  - o Debris accumulation on the surface
  - o Measurement of/inspection for sediment accumulation in rainwater harvesting tank and infiltration gallery
  - o Presence of ponded water on the surface of the bioswale beyond design duration
  - o Outlet/inlet blockages of tanks and OGS

A detailed monitoring plan is included in **Appendix H**.

### 5.3.10 Contingency Plan

It is recognized that the proposed stormwater management plan is considered a “pilot project” by the City of Ottawa and has allowed for credit from the LID measures toward the stormwater management design. As such the monitoring plan for the site will be critical in assessing the performance of the system. Should either the pre-construction monitoring result in findings that will impact the function of the system, then additional assessment of the design will be required to assess system performance and determine whether additional storage is required. Additional storage would be provided by expanding the size of the proposed infiltration gallery. This assessment would be required prior to constructing the facility. A memo will be issued to the City of Ottawa outlining the monitoring results and confirming whether there is any need for expansion of the infiltration system.

Similarly, post-construction monitoring will assess the performance of the system. Data analysis and reporting will be completed and review whether any retrofits to the system are required. The

## SERVICING AND STORMWATER MANAGEMENT BRIEF – 5731 HAZELDEAN ROAD

Stormwater Management  
March 22, 2017

greatest benefit to the SWM design is the storage available in the rainwater harvesting system. It is estimated that the greatest impact to the system storage requirements would be if this system does not operate as designed and this entire volume cannot be relied upon for the SWM system. This would result in the need for an additional 335m<sup>3</sup> to be added to the infiltration gallery. While this extreme is assumed to be unlikely, it is recommended that the site MOECC ECA include this contingency volume of 335m<sup>3</sup> to allow for the expansion if needed without requiring an amendment to the ECA before proceeding.

Post-construction monitoring will include groundwater level monitoring and water level monitoring within the infiltration gallery. Results will be monitored to ensure no storage volume is lost as a result of groundwater influences and storage volume would be adjusted as necessary. However, it is anticipated that since the infiltration gallery design includes an outlet at the bottom of the storage area, there should be no significant loss of volume caused by seasonal groundwater fluctuations or mounding.

### 5.4 SUMMARY OF FINDINGS

Based on the preceding, the following conclusions can be drawn:

- The proposed stormwater management plan is in compliance with the criteria established for the site and the 2012 City of Ottawa Sewer Guidelines.
- Inlet control devices are proposed to limit inflow from the site area into the minor system to maximize the use of surface storage.
- Subsurface storage has been provided to further limit site outflows to the peak site discharge rate determined via PCSWMM model (See **Table 14** below).
- The storm sewer hydraulic grade line is maintained at least 0.30 m below finished ground elevations during design storm events.
- All dynamic surface water depths are less than 0.30 m during all design storm events.
- Quality control is provided by a Stormceptor model STC3000 upstream of the underground storage facility to maintain water quality objectives outlined in the background reports.

**Table 14: Site Peak Discharge Rates/Targets**

Storm Event	Site Peak Discharge Rate (L/s)	Target Discharge Rate (L/s)
2-Year 12 Hour SCS	15.2	17.7
5-Year 12 Hour SCS	38.6	43.9
10-Year 12 Hour SCS	64.5	66.2
25-Year 12 Hour SCS	98.7	103.7
50-Year 12 Hour SCS	116.2	136.5
100-Year 12 Hour SCS	136.3	176.3
100-Year 12 Hour SCS +20%	317.0	-

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
5731 HAZELDEAN ROAD**

Appendix C Stormwater Management  
March 22, 2017

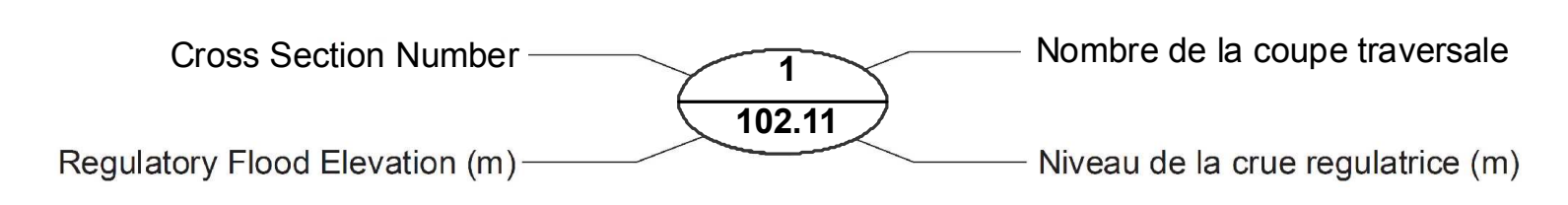
**Appendix C    STORMWATER MANAGEMENT**

**C.1    STORM SEWER DESIGN SHEET AND ROOF STORAGE  
CALCULATIONS**

# FLOOD RISK MAP POOLE CREEK CARTE DU RISQUE D'INONDATION

### LEGEND / LÉGENDE

-  Regulatory Floodplain / La Crue Régulatrice
-  Regulatory Limit / Limite Réglementaire
-  Contours / Courbes
-  Stream / Ruisseau
-  Cross Sections / La coupe transversale

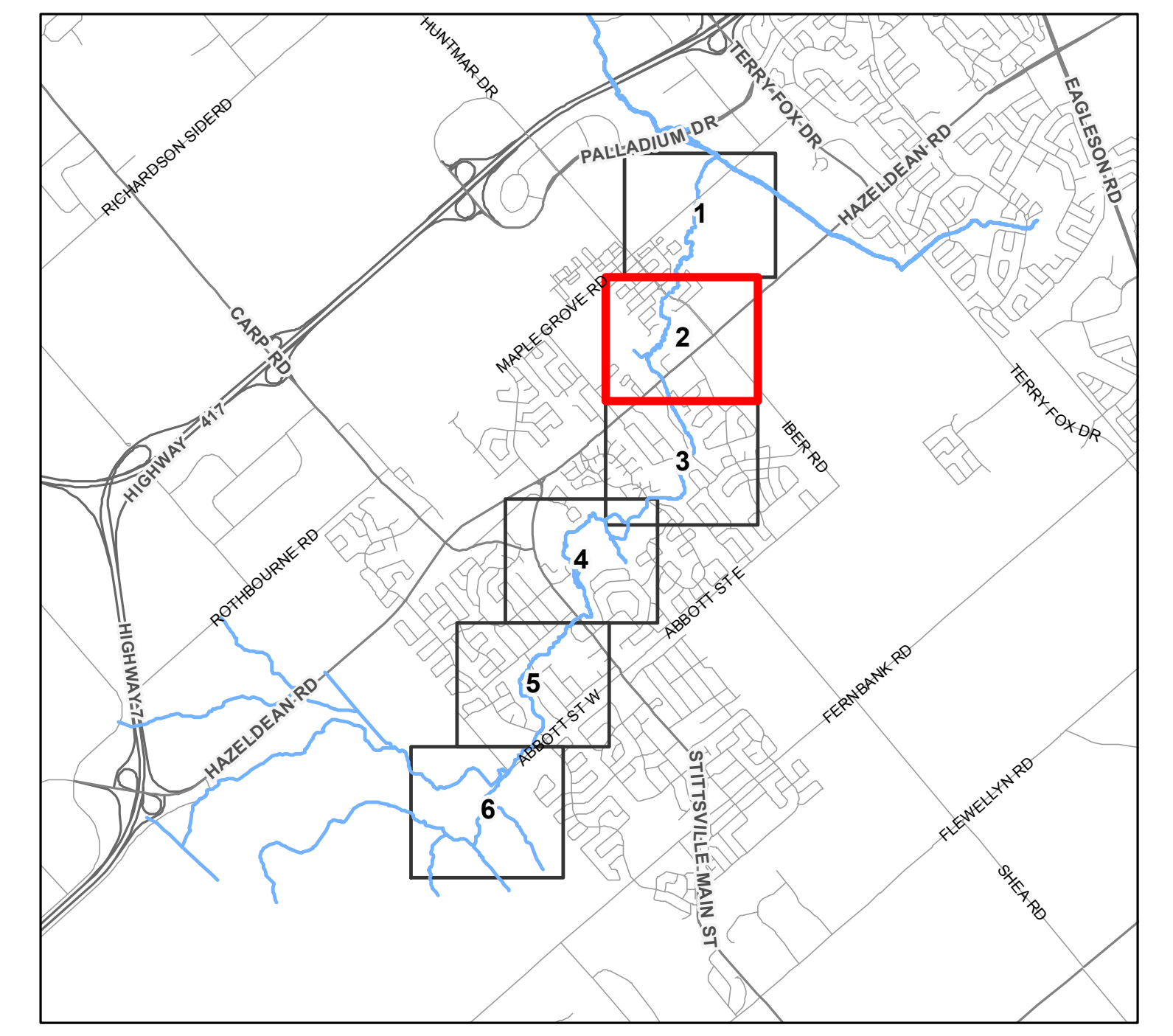


INDEX CONTOUR INTERVAL 2 METRES  
WITH 0.5 METRE INTERMEDIATE CONTOUR  
NORTH AMERICAN DATUM 1983  
COURBES DE NIVEAU PRINCIPALES DE 2.0 MÈTRE  
AVEC COURBES DE NIVEAU INTERMÉDIAIRES DE 0.5 MÈTRES  
SYSTEME DE RÉFÉRENCE GÉODÉSIQUE NORD-AMÉRIQUE 1983

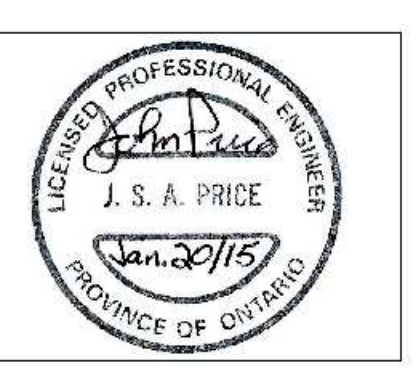
<b>GENERAL INFORMATION</b>	<b>RENSEIGNEMENTS GÉNÉRAUX</b>
Vertical Datum: Mean sea level	Niveau de référence vertical: Niveau moyen de la mer
Horizontal Datum: North American 1983	Niveau de référence horizontal: Nord-américain 1983
Map Projection: Ottawa Transverse Mercator Projection	Projection cartographique: Projection Mercator Transverse d'Ottawa



### SHEET INDEX / TABLEAU D'ASSEMBLAGE



Revision #	Issue
1 - Nov 14 2013	Public review
2 - Dec 4, 2013	Board approval
3 - Jan 21, 2015	Final

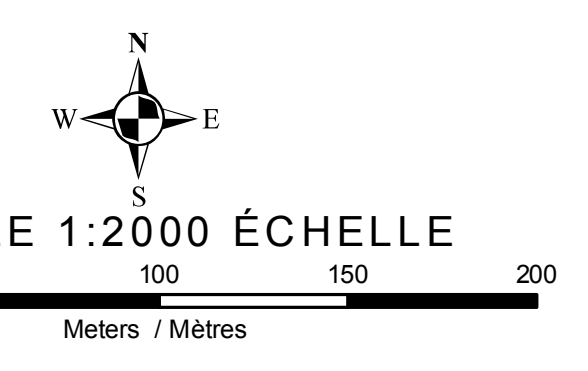


Date Modified / Date de modification: 29/01/2015

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5731 Hazeldean Road

STORM SEWER DESIGN SHEET (City of Ottawa)

FILE NUMBER: 1604-01195

DESIGN PARAMETERS

l = a / (t+b)^c (As per City of Ottawa Guidelines, 2012)
a = 998.071, b = 6.053, c = 0.814
1:5 yr: 1735.688, 6.014, 0.820
1:100 yr: 1735.688, 6.014, 0.820
MANNING'S n = 0.013
BEDDING CLASS = B
MINIMUM COVER: 2.00 m
TIME OF ENTRY 10 min

DATE: 23-Mar-2017
REVISION: 3
DESIGNED BY: DT
CHECKED BY: AL

Table with columns: LOCATION (AREA ID NUMBER, FROM M.H., TO M.H.), DRAINAGE AREA (AREA (5-YEAR), AREA (100-YEAR), AREA (ROOF), C, ACCUM. AREA (5YR), A x C (5-YEAR), ACCUM. Ax C (5YR), ACCUM. AREA (100YR), A x C (100-YEAR), ACCUM. Ax C (100YR), T of C, I 5-YEAR, I 10-YEAR, Q CONTROL (NOTE 1), ACCUM. Q CONTROL, Q ACT (CIA/360), PIPE SELECTION (LENGTH, PIPE WIDTH OR DIAMETER, PIPE HEIGHT, PIPE SHAPE, MATERIAL, CLASS, SLOPE, Q CAP (FULL), % FULL, VEL. (FULL), VEL. (ACT), TIME OF FLOW).

## Stormwater Management Calculations

File No: 160401195  
 Project: 5731 Hazeldean  
 Date: 30-Jan-17

SWM Approach:  
 Post-development to Pre-development flows

**Post-Development Site Conditions:**

**Overall Runoff Coefficient for Site and Sub-Catchment Areas**

Runoff Coefficient Table									
Catchment Type	Sub-catchment Area		Area (ha) "A"	Runoff Coefficient "C"		"A x C"	Overall Runoff Coefficient		
	ID / Description								
Roof	ST108D	Hard	0.05	0.9	0.047				
		Soft	0.00	0.2	0.000				
		Subtotal		0.052		0.047		0.900	
Roof	ST108E	Hard	0.03	0.9	0.024				
		Soft	0.00	0.2	0.000				
		Subtotal		0.027		0.024		0.900	
Roof	ST108B	Hard	0.36	0.9	0.328				
		Soft	0.00	0.2	0.000				
		Subtotal		0.364		0.328		0.900	
Roof	ST108A	Hard	0.40	0.9	0.363				
		Soft	0.00	0.2	0.000				
		Subtotal		0.404		0.363		0.900	
Roof	ST108C	Hard	0.05	0.9	0.042				
		Soft	0.00	0.2	0.000				
		Subtotal		0.047		0.042		0.900	
<b>Total</b>				<b>0.894</b>		<b>0.805</b>			
<b>Overall Runoff Coefficient= C:</b>								<b>0.90</b>	

Total Roof Areas	0.894 ha
Total Tributary Surface Areas (Controlled and Uncontrolled)	0.000 ha
Total Tributary Area to Outlet	0.894 ha
 Total Uncontrolled Areas (Non-Tributary)	 0.000 ha
 Total Site	 0.894 ha



# Stormwater Management Calculations

**Project #160401195, 5731 Hazeldean**  
**Modified Rational Method Calculatons for Storage**

50	37.65	38.04	21.66	16.39	49.16	100.7	0.00
55	35.12	35.49	21.30	14.19	46.81	99.1	0.00
60	32.94	33.28	20.88	12.40	44.65	97.1	0.00

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check	
5-year Water Level	105.67	0.11	22.72	58.20	161.52	0.00

Subdrainage Area: ST108C  
 Area (ha): 0.05  
 C: 0.90  
 Maximum Storage Depth: Roof 150 mm

tc (min)	I (5 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	Depth (mm)
5	141.18	16.63	2.74	13.90	4.17	89.1
10	104.19	12.27	3.06	9.21	5.53	99.6
15	83.56	9.84	3.14	6.71	6.03	102.1
20	70.25	8.28	3.15	5.12	<b>6.15</b>	102.7
25	60.90	7.17	3.14	4.03	6.05	102.2
30	53.93	6.35	3.11	3.24	5.84	101.2
35	48.52	5.72	3.07	2.65	5.56	99.9
40	44.18	5.21	3.00	2.20	5.29	97.7
45	40.63	4.79	2.93	1.85	5.00	95.5
50	37.65	4.44	2.87	1.57	4.71	93.3
55	35.12	4.14	2.80	1.34	4.42	91.0
60	32.94	3.88	2.73	1.15	4.14	88.9

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check	
5-year Water Level	102.66	0.10	3.15	6.15	18.83	0.00

**Project #160401195, 5731 Hazeldean**  
**Modified Rational Method Calculatons for Storage**

100	37.90	42.55	26.91	15.64	93.84	125.1	0.00
110	35.20	39.52	26.21	13.31	87.81	121.9	0.00
120	32.89	36.93	25.53	11.39	82.04	118.7	0.00

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check	
100-year Water Level	139.08	0.14	29.91	131.80	161.52	0.00

Subdrainage Area: ST108C  
 Area (ha): 0.05  
 C: 1.00  
 Maximum Storage Depth: Roof 150 mm

tc (min)	I (100 yr) (mm/hr)	Qactual (L/s)	Qrelease (L/s)	Qstored (L/s)	Vstored (m³)	Depth (mm)
10	178.56	23.37	3.92	19.46	11.67	127.4
20	119.95	15.70	4.13	11.57	13.89	134.4
30	91.87	12.02	4.16	7.87	<b>14.16</b>	135.3
40	75.15	9.84	4.11	5.72	13.73	133.9
50	63.95	8.37	4.04	4.33	12.99	131.6
60	55.89	7.32	3.96	3.36	12.10	128.8
70	49.79	6.52	3.86	2.65	11.14	125.8
80	44.99	5.89	3.75	2.14	10.27	122.0
90	41.11	5.38	3.63	1.75	9.45	118.2
100	37.90	4.96	3.52	1.44	8.66	114.5
110	35.20	4.61	3.41	1.20	7.91	111.0
120	32.89	4.31	3.31	1.00	7.20	107.6

Storage: Roof Storage

Depth (mm)	Head (m)	Discharge (L/s)	Vreq (cu. m)	Vavail (cu. m)	Discharge Check	
100-year Water Level	135.29	0.14	4.16	14.16	18.83	0.00

**Roof Drain Design Calculation Sheet**

**Project #160401195, 5731 Hazeldean  
Roof Drain Design Sheet, Area ST108A  
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0054	1	0.025	90	1	1	0.025
0.050	0.0008	0.0108	6	0.050	359	5	6	0.050
0.075	0.0012	0.0161	20	0.075	808	14	20	0.075
0.100	0.0015	0.0215	48	0.100	1436	28	48	0.100
0.125	0.0019	0.0269	93	0.125	2243	46	93	0.125
0.150	0.0023	0.0323	162	0.150	3230	68	162	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Total Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
5.2	486.8	5.2	0.13523
19.4	881.0	14.2	0.37995
47.1	1286.7	27.7	0.73735
92.7	1697.0	45.6	1.20874
160.8	2109.7	68.0	1.79476

**Rooftop Storage Summary**

Total Building Area (sq.m)		4038
Assume Available Roof Area (sq.	80%	3230
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		14
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		162
Estimated 100 Year Drawdown Time (h)		1.5

**From Zurn Drain Catalogue**

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

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

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

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.023	0.030	-
Depth (m)	0.106	0.139	0.150
Volume (cu.m)	58.2	131.8	161.5
Drain time (hrs)	0.9	1.5	

**Roof Drain Design Calculation Sheet**

**Project #160401195, 5731 Hazeldean  
Roof Drain Design Sheet, Area ST108B  
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0050	1	0.025	81	1	1	0.025
0.050	0.0008	0.0100	5	0.050	324	5	5	0.050
0.075	0.0012	0.0150	18	0.075	729	13	18	0.075
0.100	0.0015	0.0200	43	0.100	1296	25	43	0.100
0.125	0.0019	0.0250	84	0.125	2024	41	84	0.125
0.150	0.0023	0.0300	146	0.150	2915	61	146	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Total Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
4.7	473.1	4.7	0.13141
17.5	856.1	12.8	0.36921
42.5	1250.3	25.0	0.71652
83.7	1649.1	41.2	1.17459
145.1	2050.0	61.4	1.74404

**Rooftop Storage Summary**

Total Building Area (sq.m)		3644
Assume Available Roof Area (sq.	80%	2915
Roof Imperviousness		0.99
Roof Drain Requirement (sq.m/Notch)		232
Number of Roof Notches*		13
Max. Allowable Depth of Roof Ponding (m)		0.15
Max. Allowable Storage (cu.m)		146
Estimated 100 Year Drawdown Time (h)		1.5

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

**From Zurn Drain Catalogue**

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.021	0.028	-
Depth (m)	0.105	0.139	0.150
Volume (cu.m)	51.8	117.7	145.7
Drain time (hrs)	0.8	1.5	

**Roof Drain Design Calculation Sheet**

**Project #160401195, 5731 Hazeldean  
Roof Drain Design Sheet, Area ST108C  
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	10	0	0	0.025
0.050	0.0008	0.0015	1	0.050	42	1	1	0.050
0.075	0.0012	0.0023	2	0.075	94	2	2	0.075
0.100	0.0015	0.0031	6	0.100	167	3	6	0.100
0.125	0.0019	0.0038	11	0.125	262	5	11	0.125
0.150	0.0023	0.0046	19	0.150	377	8	19	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.6	397.4	0.6	0.11038
2.3	719.0	1.7	0.3101
5.5	1050.1	3.2	0.60181
10.8	1385.1	5.3	0.98655
18.7	1721.9	7.9	1.46485

**Rooftop Storage Summary**

Total Building Area (sq.m)	471
Assume Available Roof Area (sq. 80%)	377
Roof Imperviousness	0.99
Roof Drain Requirement (sq.m/Notch)	232
Number of Roof Notches*	2
Max. Allowable Depth of Roof Ponding (m)	0.15
Max. Allowable Storage (cu.m)	19
Estimated 100 Year Drawdown Time (h)	1.2

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

**From Zurn Drain Catalogue**

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

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

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.103	0.135	0.150
Volume (cu.m)	6.1	14.2	18.8
Drain time (hrs)	0.6	1.2	

**Roof Drain Design Calculation Sheet**

**Project #160401195, 5731 Hazeldean  
Roof Drain Design Sheet, Area ST108D  
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0008	0	0.025	11	0	0	0.025
0.050	0.0008	0.0015	1	0.050	46	1	1	0.050
0.075	0.0012	0.0023	3	0.075	103	2	3	0.075
0.100	0.0015	0.0031	6	0.100	184	4	6	0.100
0.125	0.0019	0.0038	12	0.125	287	6	12	0.125
0.150	0.0023	0.0046	21	0.150	414	9	21	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Total Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.7	436.4	0.7	0.12121
2.5	789.6	1.8	0.34055
6.0	1153.3	3.5	0.6609
11.9	1521.1	5.8	1.08342
20.6	1890.9	8.7	1.60868

**Rooftop Storage Summary**

Total Building Area (sq.m)	517
Assume Available Roof Area (sq. 80%)	414
Roof Imperviousness	0.99
Roof Drain Requirement (sq.m/Notch)	232
Number of Roof Notches*	2
Max. Allowable Depth of Roof Ponding (m)	0.15
Max. Allowable Storage (cu.m)	21
Estimated 100 Year Drawdown Time (h)	1.3

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

**From Zurn Drain Catalogue**

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

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

**Calculation Results**

	5yr	100yr	Available
Qresult (cu.m/s)	0.003	0.004	-
Depth (m)	0.104	0.137	0.150
Volume (cu.m)	7.1	16.2	20.7
Drain time (hrs)	0.7	1.3	



**Roof Drain Design Calculation Sheet**

**Project #160401195, 5731 Hazeldean  
Roof Drain Design Sheet, Area ST108E  
Standard Zurn Model Z-105-5 Control-Flo Single Notch Roof Drain**

Rating Curve				Volume Estimation				Water Depth (m)
Elevation (m)	Discharge Rate (cu.m/s)	Outlet Discharge (cu.m/s)	Storage (cu. m)	Elevation (m)	Area (sq. m)	Volume (cu. m)		
						Increment	Accumulated	
0.000	0.0000	0.0000	0	0.000	0	0	0	0.000
0.025	0.0004	0.0004	0	0.025	6	0	0	0.025
0.050	0.0008	0.0008	0	0.050	24	0	0	0.050
0.075	0.0012	0.0012	1	0.075	54	1	1	0.075
0.100	0.0015	0.0015	3	0.100	97	2	3	0.100
0.125	0.0019	0.0019	6	0.125	151	3	6	0.125
0.150	0.0023	0.0023	11	0.150	218	5	11	0.150

Drawdown Estimate			
Total Volume (cu.m)	Total Time (sec)	Total Vol (cu.m)	Detention Time (hr)
0.0	0.0	0.0	0
0.4	459.0	0.4	0.1275
1.3	830.5	1.0	0.3582
3.2	1213.0	1.9	0.69516
6.2	1599.9	3.1	1.13957
10.8	1988.9	4.6	1.69206

**Rooftop Storage Summary**

Total Building Area (sq.m)		272	
Assume Available Roof Area (sq.	80%	218	
Roof Imperviousness		0.99	
Roof Drain Requirement (sq.m/Notch)		232	
Number of Roof Notches*		1	
Max. Allowable Depth of Roof Ponding (m)		0.15	* As per Ontario Building Code section OBC 7.4.10.4.(2)(c).
Max. Allowable Storage (cu.m)		11	
Estimated 100 Year Drawdown Time (h)		1.4	

**From Zurn Drain Catalogue**

Head (m)	L/min	L/s	Notch Rating
0.051	45.5	0.00076	232

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

Calculation Results	5yr	100yr	Available
Qresult (cu.m/s)	0.002	0.002	-
Depth (m)	0.105	0.138	0.150
Volume (cu.m)	3.8	8.7	10.9
Drain time (hrs)	0.8	1.4	

# Outlet Rip-Rap Sizing

US Army Corps of Engineers

1991 Procedure

EM1601

Common values

---

V	1.62	m/s
y	0.37125	m
Z	3.00	H:1V
phi	42	degrees
r	300	m
W	1	m
Ss	2.5	rock specific gravity
g	9.806	m/s <sup>2</sup>
theta	18.4	degrees bank angle with horizontal

---

SF	1.1
Cs	0.3
Kl	1
Cv	0.79
Ct	1

D <sub>50</sub> =	0.048	m
M <sub>50</sub> =	0.147	kg

---

<b>Selected D50</b>	0.060	m
<b>Min. thickness</b>	0.120	m

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
5731 HAZELDEAN ROAD**

Appendix C Stormwater Management  
March 22, 2017

**C.2 SAMPLE PCSWMM MODEL INPUT (12HR 100YR SCS)**

[TITLE]

```

[OPTIONS]
;;Options      Value
-----
FLOW_UNITS    LPS
INFILTRATION  CURVE_NUMBER
FLOW_ROUTING  DYNWAVE
START_DATE    07/23/2009
START_TIME    00:00:00
REPORT_START_DATE 07/23/2009
REPORT_START_TIME 00:00:00
END_DATE      07/24/2009
END_TIME      00:00:00
SWEEP_START   01/01
SWEEP_END     12/31
DRY_DAYS      0
REPORT_STEP   00:05:00
WET_STEP      00:05:00
DRY_STEP      00:05:00
ROUTING_STEP  1
ALLOW_PONDING YES
INERTIAL_DAMPING PARTIAL
VARIABLE_STEP 0
LENGTHENING_STEP 0
MIN_SURFAREA  0
NORMAL_FLOW_LIMITED BOTH
SKIP_STEADY_STATE NO
FORCE_MAIN_EQUATION H-W
LINK_OFFSETS  ELEVATION
MIN_SLOPE     0
MAX_TRIALS    8
HEAD_TOLERANCE 0.0015
SYS_FLOW_TOL  5
LAT_FLOW_TOL  5
MINIMUM_STEP  0.5
THREADS       4
    
```

```

[EVAPORATION]
;;Type      Parameters
-----
CONSTANT    0.0
DRY_ONLY    NO
    
```

```

[RAINGAGES]
;;
;;Name      Rain    Time    Snow    Data
            Type    Intrvl  Catch   Source
    
```

```

-----
RG1          INTENSITY 0:15  1.0  TIMESERIES 100SCS
[SUBCATCHMENTS]
;;
;;Name      Raingage      Outlet      Total Area  Pcnt.  width  Pcnt.  Curb  Snow
            Raingage      Outlet      Area       Imperv  width  Slope  Length Snow
            -----
EXT1        RG1          EXT1-OF     0.067219  0      15.124  33.3  0
EXT2        RG1          EXT2-OF     0.063791  72.857  14.353  2      0
ST104A     RG1          ST104A-S   0.149935  84.286  69      2      0
ST107A     RG1          ST107A-S   0.373344  64.286  225     1.5    0
ST108A     RG1          ST108A-S   0.403809  100     90.857  1.5    0
ST108B     RG1          ST108B-S   0.36437   100     81.983  1.5    0
ST108C     RG1          ST108C-S   0.047083  100     12.1    1.5    0
ST108D     RG1          ST108D-S   0.051706  100     10.9    1.5    0
ST108E     RG1          ST108E-S   0.027193  100     25      1.5    0
ST108F     RG1          108        0.382042  44.286  85.96   1.2    0
ST109A     RG1          109        0.013537  100     18.2    10     0
ST109B     RG1          ST109B-S   0.054305  100     24.8    1      0
ST109C     RG1          ST109C-S   0.058618  100     25.8    1      0
ST110A     RG1          110        0.074661  7.143   16.799  0.8    0
ST110B     RG1          110        0.031906  100     24.5    10     0
ST110C     RG1          110        0.029561  100     26.6    10     0
ST110D     RG1          110        0.074098  7.143   16.672  0.8    0
ST111A     RG1          ST111A-S   0.242699  72.857  107.5   0.8    0
ST111B     RG1          111        0.037296  100     88      0.8    0
ST111C     RG1          ST111C-S   0.043507  85.714  36.8    1.5    0
ST507A     RG1          ST507A-S   0.054432  72.857  33.5    1.5    0
    
```

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ST508A RG1 508 0.3356 7.143 189.2 1 0

[SUBAREAS]

Subcatchment	N-Imperv	N-Perv	S-Imperv	S-Perv	PctZero	RouteTo	PctRouted
EXT1	0.013	0.2	1.57	4.67	0	PERVIOUS	100
EXT2	0.013	0.2	1.57	4.67	0	PERVIOUS	100
ST104A	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST107A	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST108A	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST108B	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST108C	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST108D	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST108E	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST108F	0.013	0.2	1.57	4.67	0	PERVIOUS	100
ST109A	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST109B	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST109C	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST110A	0.013	0.2	1.57	4.67	0	PERVIOUS	100
ST110B	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST110C	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST110D	0.013	0.2	1.57	4.67	0	PERVIOUS	100
ST111A	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST111B	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST111C	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST507A	0.013	0.2	1.57	4.67	0	IMPERVIOUS	100
ST508A	0.013	0.2	1.57	4.67	0	PERVIOUS	100

[INFILTRATION]

Subcatchment	CurveNum	HydCon	DryTime
EXT1	80	0	7
EXT2	80	0	7
ST104A	80	0	7
ST107A	80	0	7
ST108A	80	0	7
ST108B	80	0	7
ST108C	80	0	7
ST108D	80	0	7
ST108E	80	0	7
ST108F	80	0	7
ST109A	80	0	7
ST109B	80	0	7
ST109C	80	0	7
ST110A	80	0	7
ST110B	80	0	7

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ST110C	80	0	7
ST110D	80	0	7
ST111A	80	0	7
ST111B	80	0.5	7
ST111C	80	0	7
ST507A	80	0	7
ST508A	80	0	7

[JUNCTIONS]

Name	Invert Elev.	Max. Depth	Init. Depth	Surcharge Depth	Ponded Area
100	99.4	2.735	0	0	0
101	99.60313	3.691	0	0	0
102	99.7793	3.752	0	0	0
103	100.035	3.526	0	0	0
104	100.0785	3.492	0	0	0
105	100.2317	3.406	0	0	0
106	100.3418	2.976	0	0	0
107	100.8033	2.417	0	0	0
109	100.83	3.67	0	0	0
110	100.686	3.814	0	0	0
111	101.4225	2.65	0	0	0

[OUTFALLS]

Name	Invert Elev.	Outfall Type	Stage/Table Time Series	Tide Gate	Route To
EXT1-OF	102.88	FREE		NO	
EXT2-OF	104.2	FREE		NO	
HEADWALL	98.7	FREE		NO	
POOLE_OF1	100.1	FREE		NO	
POOLE_OF2	101.059	FREE		NO	
ST104A-OF	0	FREE		NO	
ST107A-OF	0	FREE		NO	

[STORAGE]

Name	Invert Elev.	Max. Depth	Init. Depth	Storage Curve	Curve Params	Ponded Area	Evap. Frac.
Infiltration parameters							
108	97.239	7.127	0	TABULAR	RWhtank	0	0
508	101.06	1.79	0	TABULAR	ST508A-S	0	0
ST104A-S	101.52	2.1	0	TABULAR	ST104A-S	0.01	0
ST107A-S	101.13	2.1	0	TABULAR	ST107A-S	0.01	0
ST108A-S	118.6	0.15	0	TABULAR	ST108A	0.01	0
ST108B-S	115.75	0.15	0	TABULAR	ST108B	0.01	0

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ST108C-S	110.4	0.15	0	TABULAR	ST108C	0.01	0
ST108D-S	110.1	0.15	0	TABULAR	ST108D	0.01	0
ST108E-S	107.2	0.15	0	TABULAR	ST108E	0.01	0
ST109B-S	102.81	1.5	0	FUNCTIONAL	0	0	0
ST109C-S	102.81	1.5	0	FUNCTIONAL	0	0	0
ST111A-S	101.86	2.4	0	TABULAR	ST111A-S	0.01	0
ST111C-S	101.95	2.1	0	FUNCTIONAL	0	0	0
ST507A-S	101.57	2.1	0	TABULAR	ST504A-S	0.01	0
TANK	100.1	3.27	0	TABULAR	TANK	0.01	0

[CONDUITS]							
;;	Inlet	Outlet		Manning	Inlet	Outlet	Init.
Max. Name Flow	Node	Node	Length	N	Offset	Offset	Flow
;;	-----						
Pipe_13	100	HEADWALL	11.135	0.013	99.548	99.52	0 0
Pipe_14	106	105	17.55995	0.013	100.411	100.376	0 0
Pipe_14_(1)	105	104	39.10268	0.013	100.301	100.222	0 0
Pipe_15	109	104	38.24086	0.013	100.83	100.447	0 0
Pipe_16	110	106	12.50838	0.013	100.686	100.561	0 0
Pipe_17	111	107	110.3626	0.013	101.65	100.877	0 0
Pipe_21	104	103	16.284	0.013	100.143	100.11	0 0
Pipe_23	508	TANK	8.730414	0.013	101.6588	101.637	0 0
Pipe_26	101	100	101.5684	0.013	99.936	99.733	0 0
Pipe_27	108	105	36.34	0.013	100.441	100.332	0 0
Pipe_29	107	106	63.29649	0.013	100.802	100.486	0 0
Pipe_31	102	101	70.701	0.013	100.083	99.942	0 0
Pipe_34	103	TANK	2.81	0.013	100.106	100.1	0 0
ST104A-T	ST104A-S	ST104A-OF	2.5	0.025	103.47	102.88	0 0
ST107A-T	ST107A-S	ST107A-OF	2.5	0.025	103.08	103.04	0 0
ST111A-T	ST111A-S	ST111C-S	40.9	0.013	104.26	103.75	0 0

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ST111B-T	ST111C-S	ST107A-S	60	0.013	103.75	103.08	0 0
ST507A-T	ST507A-S	ST104A-S	14.9	0.013	103.5	103.47	0 0
W1	103	102	3	0.013	102	101.97	0 0

[ORIFICES]							
;;	Inlet	Outlet	Orifice	Crest	Disch.	Flap	Open/Close
;;Name	Node	Node	Type	Height	Coeff.	Gate	Time
;;	-----						
OR1	TANK	102	SIDE	100.1	0.65	NO	0
OR2	TANK	102	SIDE	100.7	0.65	NO	0
OR3	TANK	102	SIDE	101	0.65	NO	0
ST104A-0	ST104A-S	104	SIDE	101.52	0.572	NO	0
ST107A-0	ST107A-S	107	SIDE	101.13	0.65	NO	0
ST109B-0	ST109B-S	109	SIDE	102.81	0.65	NO	0
ST109C-0	ST109C-S	109	SIDE	102.81	0.65	NO	0
ST111A-0	ST111A-S	111	SIDE	101.86	0.572	NO	0
ST111C-0	ST111C-S	111	SIDE	101.95	0.65	NO	0
ST111C-01	ST111C-S	111	SIDE	101.95	0.65	NO	0

[OUTLETS]							
;;	Inlet	Outlet	Outflow	Outlet	Qcoeff/		
Flap Name Gate	Node	Node	Height	Type	QTable	Qexpon	
;;	-----						
OL1	TANK	POOLE_OF1	100.1	TABULAR/HEAD	TANK_BASEFLOW		
NO							
OL2	508	POOLE_OF2	101.06	TABULAR/HEAD	BIOSWALE_BASEFLOW		
NO							
ST108A-0	ST108A-S	108	118.6	TABULAR/HEAD	ST108A-0		
NO							
ST108B-0	ST108B-S	108	115.75	TABULAR/HEAD	ST108B-0		
NO							
ST108C-0	ST108C-S	108	110.4	TABULAR/HEAD	ST108C-0		
NO							
ST108D-0	ST108D-S	108	110.1	TABULAR/HEAD	ST108D-0		
NO							
ST108E-0	ST108E-S	108	107.2	TABULAR/HEAD	ST108E-0		
NO							
ST507A-0	ST507A-S	TANK	101.57	FUNCTIONAL/HEAD	7.996	0.499	
NO							

[XSECTIONS]

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;;Link	Shape	Geom1	Geom2	Geom3	Geom4	Barrels
Pipe_13	CIRCULAR	0.675	0	0	0	1
Pipe_14	CIRCULAR	0.525	0	0	0	1
Pipe_14_(1)	CIRCULAR	0.6	0	0	0	1
Pipe_15	CIRCULAR	0.375	0	0	0	1
Pipe_16	CIRCULAR	0.375	0	0	0	1
Pipe_17	CIRCULAR	0.375	0	0	0	1
Pipe_21	CIRCULAR	0.675	0	0	0	1
Pipe_23	CIRCULAR	0.25	0	0	0	1
Pipe_26	CIRCULAR	0.45	0	0	0	1
Pipe_27	CIRCULAR	0.45	0	0	0	1
Pipe_29	CIRCULAR	0.45	0	0	0	1
Pipe_31	CIRCULAR	0.45	0	0	0	1
Pipe_34	CIRCULAR	0.675	0	0	0	1
ST104A-T	IRREGULAR	overland	0	0	0	1
ST107A-T	IRREGULAR	overland	0	0	0	1
ST111A-T	IRREGULAR	overland	0	0	0	1
ST111B-T	IRREGULAR	overland	0	0	0	1
ST507A-T	IRREGULAR	overland	0	0	0	1
W1	CIRCULAR	0.45	0	0	0	1
OR1	CIRCULAR	0.11	0	0	0	
OR2	CIRCULAR	0.15	0	0	0	
OR3	CIRCULAR	0.15	0	0	0	
ST104A-O	CIRCULAR	0.14	0	0	0	
ST107A-O	CIRCULAR	0.2	0	0	0	
ST109B-O	CIRCULAR	0.2	0	0	0	
ST109C-O	CIRCULAR	0.2	0	0	0	
ST111A-O	CIRCULAR	0.076	0	0	0	
ST111C-O	CIRCULAR	0.2	0	0	0	
ST111C-O1	CIRCULAR	0.2	0	0	0	

[TRANSECTS]

NC 0.013	0.013	0.013								
X1 overland	5	0	0.15	6.85	0.0	0.0	0.0	0.0	0.0	0.0
GR 0.15	0	0	0.15	0	6.85	0.15	7	0.15	7	

;;[LE: 0][RE: 7]

NC 0.013	0.013	0.013								
X1 overland(orig)	4	0	0.15	6.85	0.0	0.0	0.0	0.0	0.0	0.0
GR 0.15	0	0	0.15	0	6.85	0.15	7			

[LOSSES]

;;Link	Inlet	Outlet	Average	Flap Gate	SeepageRate
Pipe_14	0	0.053	0	NO	0
Pipe_14_(1)	0	0.022	0	NO	0

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Pipe_15	0	1.344	0	NO	0
Pipe_16	0	1.344	0	NO	0
Pipe_17	0	1.344	0	NO	0
Pipe_21	0	0.022	0	NO	0
Pipe_26	0	0.423	0	NO	0
Pipe_27	0	1.344	0	NO	0
Pipe_29	0	0.053	0	NO	0
Pipe_31	0	1.344	0	NO	0
W1	0	1.344	0	NO	0

[INFLOWS]

;;Node	Parameter	Time Series	Param Type	Units Factor	Scale Factor	Baseline Value	Baseline Pattern
100	FLOW	""	FLOW	1.0	1	175	

[CURVES]

;;Name	Type	X-Value	Y-Value
BIOSWALE_BASEFLOW	Rating	0.00	0
BIOSWALE_BASEFLOW		0.01	0.3
BIOSWALE_BASEFLOW		10.00	0.3
ST108A-O	Rating	0	0
ST108A-O		0.025	5.4
ST108A-O		0.050	10.8
ST108A-O		0.075	16.1
ST108A-O		0.100	21.5
ST108A-O		0.125	26.9
ST108A-O		0.150	32.3
ST108B-O	Rating	0	0
ST108B-O		0.025	5.0
ST108B-O		0.050	10.0
ST108B-O		0.075	15.0
ST108B-O		0.100	20.0
ST108B-O		0.125	25.0
ST108B-O		0.150	30.0
ST108C-O	Rating	0	0
ST108C-O		0.025	0.8
ST108C-O		0.050	1.5
ST108C-O		0.075	2.3
ST108C-O		0.100	3.1
ST108C-O		0.125	3.8
ST108C-O		0.150	4.6
ST108D-O	Rating	0	0

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ST108D-0		0.025	0.8
ST108D-0		0.050	1.5
ST108D-0		0.075	2.3
ST108D-0		0.100	3.1
ST108D-0		0.125	3.8
ST108D-0		0.150	4.6
ST108E-0	Rating	0	0
ST108E-0		0.025	0.4
ST108E-0		0.050	0.8
ST108E-0		0.075	1.2
ST108E-0		0.100	1.5
ST108E-0		0.125	1.9
ST108E-0		0.150	2.3
TANK_BASEFLOW	Rating	0.00	0
TANK_BASEFLOW		0.01	0.66
TANK_BASEFLOW		10.00	0.66
RWhtank	Storage	0	113.747
RWhtank		3.202	113.747
RWhtank		3.203	0
RWhtank		5	0
ST104A-S	Storage	0	0
ST104A-S		1.8	0
ST104A-S		1.95	32
ST107A-S	Storage	0	0
ST107A-S		1.8	0
ST107A-S		1.95	30.67
ST108A	Storage	0	0
ST108A		0.025	90
ST108A		0.050	359
ST108A		0.075	808
ST108A		0.100	1436
ST108A		0.125	2243
ST108A		0.150	3230
ST108B	Storage	0	0
ST108B		0.025	81
ST108B		0.050	324
ST108B		0.075	729
ST108B		0.100	1296
ST108B		0.125	2024
ST108B		0.150	2915

ST108C	Storage	0	0
ST108C		0.025	10
ST108C		0.050	42
ST108C		0.075	94
ST108C		0.100	167
ST108C		0.125	262
ST108C		0.150	377
ST108D	Storage	0	0
ST108D		0.025	11
ST108D		0.050	46
ST108D		0.075	103
ST108D		0.100	184
ST108D		0.125	287
ST108D		0.150	414
ST108E	Storage	0	0
ST108E		0.025	6
ST108E		0.050	24
ST108E		0.075	54
ST108E		0.100	97
ST108E		0.125	151
ST108E		0.150	218
ST111A-S	Storage	0	0
ST111A-S		2.10	0
ST111A-S		2.40	724
ST504A-S	Storage	0	0
ST504A-S		1.8	0
ST504A-S		1.93	181.54
ST508A-S	Storage	0	0
ST508A-S		0.7	152
ST508A-S		0.701	0
ST508A-S		1.741	0
ST508A-S		1.991	114.4
TANK	Storage	0	560.7
TANK		0.026	560.7
TANK		0.051	560.7
TANK		0.077	560.7
TANK		0.102	559.44
TANK		0.127	559.44
TANK		0.153	558.18
TANK		0.178	556.92
TANK		0.204	555.66
TANK		0.229	554.4



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TANK	0.254	551.88
TANK	0.28	549.36
TANK	0.305	546.84
TANK	0.331	543.06
TANK	0.356	539.28
TANK	0.381	534.24
TANK	0.407	527.94
TANK	0.432	521.64
TANK	0.458	514.08
TANK	0.483	505.26
TANK	0.508	495.18
TANK	0.534	483.84
TANK	0.559	478.8
TANK	0.585	464.94
TANK	0.61	449.82
TANK	0.635	434.7
TANK	0.661	419.58
TANK	0.686	403.2
TANK	0.712	383.04
TANK	0.737	360.36
TANK	0.762	347.76
TANK	0.796	335.16
TANK	0.813	320.04
TANK	0.839	304.92
TANK	0.864	289.8
TANK	0.889	272.16
TANK	0.915	258.3
TANK	0.94	244.44
TANK	0.965	233.1
TANK	0.991	221.76
TANK	1.016	211.68
TANK	1.041	201.6
TANK	1.067	192.78
TANK	1.092	185.22
TANK	1.118	180.18
TANK	1.143	176.4
TANK	1.168	172.62
TANK	1.194	170.1
TANK	1.219	167.58
TANK	1.245	165.06
TANK	1.27	163.8
TANK	1.295	162.54
TANK	1.321	162.54
TANK	1.346	162.54
TANK	1.372	161.28
TANK	1.397	161.28
TANK	1.422	161.28
TANK	1.448	161.28

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TANK	1.473	161.28
TANK	1.499	161.28
TANK	1.524	161.28
TANK	1.549	161.28
TANK	1.575	161.28
TANK	1.6	161.28
TANK	1.626	161.28
TANK	1.651	161.28
TANK	1.676	161.28
TANK	1.702	161.28
TANK	1.727	161.28
TANK	1.753	161.28
TANK	1.778	161.28
TANK	1.803	161.28
TANK	1.829	161.28
TANK	1.83	0
TANK	5	0

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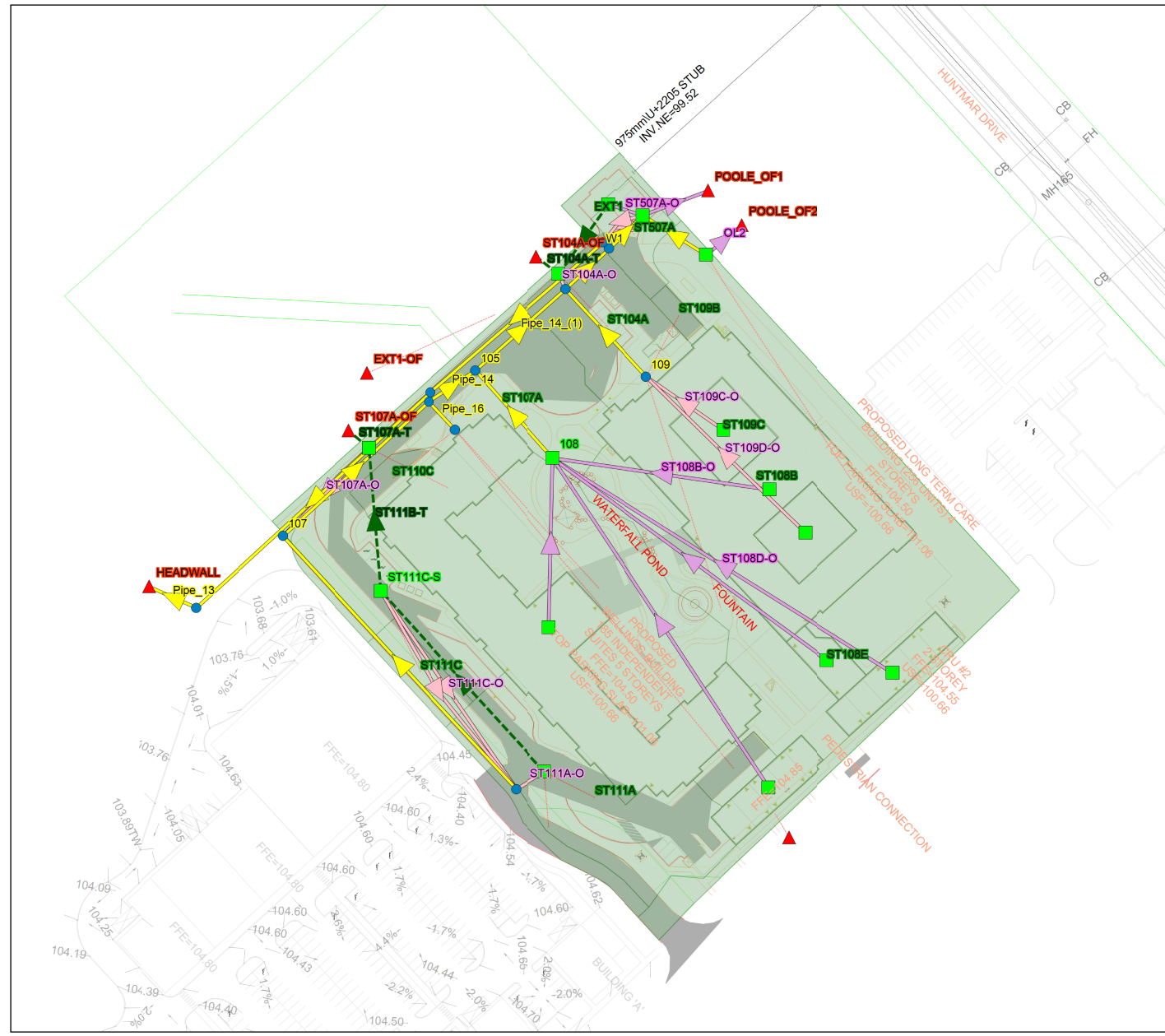
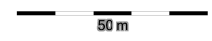
**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
5731 HAZELDEAN ROAD**

Appendix C Stormwater Management  
March 22, 2017

**C.3 SAMPLE PCSWMM MODEL OUTPUT (12HR 100YR SCS)**

### Legend

- Junctions
- ▲ Outfalls
- Storages
- Conduits
  - Visible
  - Major\_Overland
  - Orifices
  - Outlets
- Subcatchments
- ACAD-160401195 SP



WARNING 03: negative offset ignored for Link Pipe\_29

\*\*\*\*\*  
Element Count

\*\*\*\*\*  
Number of rain gages ..... 1  
Number of subcatchments ... 22  
Number of nodes ..... 33  
Number of links ..... 37  
Number of pollutants ..... 0  
Number of land uses ..... 0

\*\*\*\*\*  
Raingage Summary  
\*\*\*\*\*

Name	Data Source	Data Type	Recording Interval
RG1	100SCS	INTENSITY	15 min.

\*\*\*\*\*  
Subcatchment Summary  
\*\*\*\*\*

Name	Area	Width	%Imperv	%Slope	Rain Gage	Outlet
EXT1	0.07	15.12	0.00	33.3000	RG1	EXT1-OF
EXT2	0.06	14.35	72.86	2.0000	RG1	EXT2-OF
ST104A	0.15	69.00	84.29	2.0000	RG1	ST104A-S
ST107A	0.37	225.00	64.29	1.5000	RG1	ST107A-S
ST108A	0.40	90.86	100.00	1.5000	RG1	ST108A-S
ST108B	0.36	81.98	100.00	1.5000	RG1	ST108B-S
ST108C	0.05	12.10	100.00	1.5000	RG1	ST108C-S
ST108D	0.05	10.90	100.00	1.5000	RG1	ST108D-S
ST108E	0.03	25.00	100.00	1.5000	RG1	ST108E-S
ST108F	0.38	85.96	44.29	1.2000	RG1	108
ST109A	0.01	18.20	100.00	10.0000	RG1	109
ST109B	0.05	24.80	100.00	1.0000	RG1	ST109B-S
ST109C	0.06	25.80	100.00	1.0000	RG1	ST109C-S
ST110A	0.07	16.80	7.14	0.8000	RG1	110
ST110B	0.03	24.50	100.00	10.0000	RG1	110
ST110C	0.03	26.60	100.00	10.0000	RG1	110

Name	Area	Width	%Imperv	%Slope	Rain Gage	Outlet
ST110D	0.07	16.67	7.14	0.8000	RG1	110
ST111A	0.24	107.50	72.86	0.8000	RG1	ST111A-S
ST111B	0.04	88.00	100.00	0.8000	RG1	111
ST111C	0.04	36.80	85.71	1.5000	RG1	ST111C-S
ST507A	0.05	33.50	72.86	1.5000	RG1	ST507A-S
ST508A	0.34	189.20	7.14	1.0000	RG1	508

\*\*\*\*\*  
Node Summary  
\*\*\*\*\*

Name	Type	Invert Elev.	Max. Depth	Ponded Area	External Inflow
100	JUNCTION	99.40	2.73	0.0	Yes
101	JUNCTION	99.60	3.69	0.0	
102	JUNCTION	99.78	3.75	0.0	
103	JUNCTION	100.04	3.53	0.0	
104	JUNCTION	100.08	3.49	0.0	
105	JUNCTION	100.23	3.41	0.0	
106	JUNCTION	100.34	2.98	0.0	
107	JUNCTION	100.80	2.42	0.0	
109	JUNCTION	100.83	3.67	0.0	
110	JUNCTION	100.69	3.81	0.0	
111	JUNCTION	101.42	2.65	0.0	
EXT1-OF	OUTFALL	102.88	0.00	0.0	
EXT2-OF	OUTFALL	104.20	0.00	0.0	
HEADWALL	OUTFALL	98.70	1.50	0.0	
POOLE_OF1	OUTFALL	100.10	0.00	0.0	
POOLE_OF2	OUTFALL	101.06	0.00	0.0	
ST104A-OF	OUTFALL	0.00	103.03	0.0	
ST107A-OF	OUTFALL	0.00	103.19	0.0	
108	STORAGE	97.24	7.13	0.0	
508	STORAGE	101.06	1.79	0.0	
ST104A-S	STORAGE	101.52	2.10	0.0	
ST107A-S	STORAGE	101.13	2.10	0.0	
ST108A-S	STORAGE	118.60	0.15	0.0	
ST108B-S	STORAGE	115.75	0.15	0.0	
ST108C-S	STORAGE	110.40	0.15	0.0	
ST108D-S	STORAGE	110.10	0.15	0.0	
ST108E-S	STORAGE	107.20	0.15	0.0	
ST109B-S	STORAGE	102.81	1.50	0.0	
ST109C-S	STORAGE	102.81	1.50	0.0	
ST111A-S	STORAGE	101.86	2.40	0.0	
ST111C-S	STORAGE	101.95	2.10	0.0	
ST507A-S	STORAGE	101.57	2.10	0.0	
TANK	STORAGE	100.10	3.27	0.0	

\*\*\*\*\*  
Link Summary  
\*\*\*\*\*

Name	From Node	To Node	Type	Length	%Slope	Roughness
Pipe_13	100	HEADWALL	CONDUIT	11.1	0.2515	0.0130
Pipe_14	106	105	CONDUIT	17.6	0.1993	0.0130
Pipe_14_(1)	105	104	CONDUIT	39.1	0.2020	0.0130
Pipe_15	109	104	CONDUIT	38.2	1.0016	0.0130
Pipe_16	110	106	CONDUIT	12.5	0.9994	0.0130
Pipe_17	111	107	CONDUIT	110.4	0.7004	0.0130
Pipe_21	104	103	CONDUIT	16.3	0.2027	0.0130
Pipe_23	508	TANK	CONDUIT	8.7	0.2497	0.0130
Pipe_26	101	100	CONDUIT	101.6	0.1999	0.0130
Pipe_27	108	105	CONDUIT	36.3	0.2999	0.0130
Pipe_29	107	106	CONDUIT	63.3	0.5013	0.0130
Pipe_31	102	101	CONDUIT	70.7	0.1994	0.0130
Pipe_34	103	TANK	CONDUIT	2.8	0.2135	0.0130
ST104A-T	ST104A-S	ST104A-OF	CONDUIT	2.5	24.2860	0.0130
ST107A-T	ST107A-S	ST107A-OF	CONDUIT	2.5	1.6002	0.0130
ST111A-T	ST111A-S	ST111C-S	CONDUIT	40.9	1.2470	0.0130
ST111B-T	ST111C-S	ST107A-S	CONDUIT	60.0	1.1167	0.0130
ST507A-T	ST507A-S	ST104A-S	CONDUIT	14.9	0.2013	0.0130
w1	103	102	CONDUIT	3.0	1.0001	0.0130
OR1	TANK	102	ORIFICE			
OR2	TANK	102	ORIFICE			
OR3	TANK	102	ORIFICE			
ST104A-O	ST104A-S	104	ORIFICE			
ST107A-O	ST107A-S	107	ORIFICE			
ST109B-O	ST109B-S	109	ORIFICE			
ST109C-O	ST109C-S	109	ORIFICE			
ST111A-O	ST111A-S	111	ORIFICE			
ST111C-O	ST111C-S	111	ORIFICE			
ST111C-01	ST111C-S	111	ORIFICE			
OL1	TANK	POOLE_OF1	OUTLET			
OL2	508	POOLE_OF2	OUTLET			
ST108A-O	ST108A-S	108	OUTLET			
ST108B-O	ST108B-S	108	OUTLET			
ST108C-O	ST108C-S	108	OUTLET			
ST108D-O	ST108D-S	108	OUTLET			
ST108E-O	ST108E-S	108	OUTLET			
ST507A-O	ST507A-S	TANK	OUTLET			

\*\*\*\*\*  
Cross Section Summary  
\*\*\*\*\*

Conduit	Shape	Full Depth	Full Area	Hyd. Rad.	Max. Width	No. of Barrels	Full Flow
Pipe_13	CIRCULAR	0.68	0.36	0.17	0.68	1	421.55
Pipe_14	CIRCULAR	0.53	0.22	0.13	0.53	1	192.01
Pipe_14_(1)	CIRCULAR	0.60	0.28	0.15	0.60	1	276.00
Pipe_15	CIRCULAR	0.38	0.11	0.09	0.38	1	175.48
Pipe_16	CIRCULAR	0.38	0.11	0.09	0.38	1	175.29
Pipe_17	CIRCULAR	0.38	0.11	0.09	0.38	1	146.75
Pipe_21	CIRCULAR	0.68	0.36	0.17	0.68	1	378.43
Pipe_23	CIRCULAR	0.25	0.05	0.06	0.25	1	29.72
Pipe_26	CIRCULAR	0.45	0.16	0.11	0.45	1	127.47
Pipe_27	CIRCULAR	0.45	0.16	0.11	0.45	1	156.15
Pipe_29	CIRCULAR	0.45	0.16	0.11	0.45	1	201.87
Pipe_31	CIRCULAR	0.45	0.16	0.11	0.45	1	127.33
Pipe_34	CIRCULAR	0.68	0.36	0.17	0.68	1	388.45
ST104A-T	overland	0.15	1.03	0.14	7.00	1	10684.06
ST107A-T	overland	0.15	1.03	0.14	7.00	1	2742.50
ST111A-T	overland	0.15	1.03	0.14	7.00	1	2421.02
ST111B-T	overland	0.15	1.03	0.14	7.00	1	2291.05
ST507A-T	overland	0.15	1.03	0.14	7.00	1	972.81
w1	CIRCULAR	0.45	0.16	0.11	0.45	1	285.13

\*\*\*\*\*  
Transect Summary  
\*\*\*\*\*

Transect Overland Area:

0.0196	0.0392	0.0588	0.0784	0.0980
0.1177	0.1374	0.1571	0.1768	0.1965
0.2162	0.2360	0.2558	0.2756	0.2954
0.3152	0.3351	0.3550	0.3748	0.3947
0.4147	0.4346	0.4546	0.4745	0.4945
0.5145	0.5346	0.5546	0.5747	0.5947
0.6148	0.6350	0.6551	0.6752	0.6954
0.7156	0.7358	0.7560	0.7762	0.7965
0.8168	0.8371	0.8574	0.8777	0.8980
0.9184	0.9388	0.9592	0.9796	1.0000
Hrad:				
0.0208	0.0415	0.0622	0.0829	0.1036
0.1242	0.1448	0.1653	0.1858	0.2063
0.2268	0.2472	0.2676	0.2879	0.3083
0.3285	0.3488	0.3690	0.3892	0.4094
0.4295	0.4496	0.4697	0.4897	0.5097
0.5297	0.5496	0.5695	0.5894	0.6093

	0.6291	0.6489	0.6686	0.6884	0.7081
	0.7277	0.7474	0.7670	0.7865	0.8061
	0.8256	0.8451	0.8646	0.8840	0.9034
	0.9228	0.9421	0.9614	0.9807	1.0000
width:	0.9580	0.9589	0.9597	0.9606	0.9614
	0.9623	0.9631	0.9640	0.9649	0.9657
	0.9666	0.9674	0.9683	0.9691	0.9700
	0.9709	0.9717	0.9726	0.9734	0.9743
	0.9751	0.9760	0.9769	0.9777	0.9786
	0.9794	0.9803	0.9811	0.9820	0.9829
	0.9837	0.9846	0.9854	0.9863	0.9871
	0.9880	0.9889	0.9897	0.9906	0.9914
	0.9923	0.9931	0.9940	0.9949	0.9957
	0.9966	0.9974	0.9983	0.9991	1.0000

Transect overland(orig)

Area:	0.0196	0.0392	0.0588	0.0784	0.0980
	0.1177	0.1374	0.1571	0.1768	0.1965
	0.2162	0.2360	0.2558	0.2756	0.2954
	0.3152	0.3351	0.3550	0.3748	0.3947
	0.4147	0.4346	0.4546	0.4745	0.4945
	0.5145	0.5346	0.5546	0.5747	0.5947
	0.6148	0.6350	0.6551	0.6752	0.6954
	0.7156	0.7358	0.7560	0.7762	0.7965
	0.8168	0.8371	0.8574	0.8777	0.8980
	0.9184	0.9388	0.9592	0.9796	1.0000

Hrad:	0.0208	0.0415	0.0622	0.0829	0.1036
	0.1242	0.1448	0.1653	0.1858	0.2063
	0.2268	0.2472	0.2676	0.2879	0.3083
	0.3285	0.3488	0.3690	0.3892	0.4094
	0.4295	0.4496	0.4697	0.4897	0.5097
	0.5297	0.5496	0.5695	0.5894	0.6093
	0.6291	0.6489	0.6686	0.6884	0.7081
	0.7277	0.7474	0.7670	0.7865	0.8061
	0.8256	0.8451	0.8646	0.8840	0.9034
	0.9228	0.9421	0.9614	0.9807	1.0000

width:	0.9580	0.9589	0.9597	0.9606	0.9614
	0.9623	0.9631	0.9640	0.9649	0.9657
	0.9666	0.9674	0.9683	0.9691	0.9700
	0.9709	0.9717	0.9726	0.9734	0.9743
	0.9751	0.9760	0.9769	0.9777	0.9786
	0.9794	0.9803	0.9811	0.9820	0.9829
	0.9837	0.9846	0.9854	0.9863	0.9871
	0.9880	0.9889	0.9897	0.9906	0.9914

	0.9923	0.9931	0.9940	0.9949	0.9957
	0.9966	0.9974	0.9983	0.9991	1.0000

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 NOTE: The summary statistics displayed in this report are based on results found at every computational time step, not just on results from each reporting time step.  
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Analysis Options

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Flow Units ..... LPS  
 Process Models:  
   Rainfall/Runoff ..... YES  
   RDII ..... NO  
   Snowmelt ..... NO  
   Groundwater ..... NO  
   Flow Routing ..... YES  
   Ponding Allowed ..... YES  
   Water Quality ..... NO  
 Infiltration Method ..... CURVE\_NUMBER  
 Flow Routing Method ..... DYNWAVE  
 Starting Date ..... 07/23/2009 00:00:00  
 Ending Date ..... 07/24/2009 00:00:00  
 Antecedent Dry Days ..... 0.0  
 Report Time Step ..... 00:05:00  
 Wet Time Step ..... 00:05:00  
 Dry Time Step ..... 00:05:00  
 Routing Time Step ..... 1.00 sec  
 Variable Time Step ..... NO  
 Maximum Trials ..... 8  
 Number of Threads ..... 4  
 Head Tolerance ..... 0.001500 m

*****		Volume	Depth
*****		hectare-m	mm
*****		-----	-----
Total Precipitation	.....	0.285	95.520
Evaporation Loss	.....	0.000	0.000
Infiltration Loss	.....	0.041	13.902
Surface Runoff	.....	0.239	80.308
Final Storage	.....	0.005	1.589
Continuity Error (%)	.....	-0.292	

Flow Routing Continuity	Volume hectare-m	Volume 10 <sup>6</sup> ltr
Dry weather Inflow	0.000	0.000
Wet weather Inflow	0.239	2.394
Groundwater Inflow	0.000	0.000
RDII Inflow	0.000	0.000
External Inflow	1.512	15.120
External Outflow	1.711	17.108
Flooding Loss	0.000	0.000
Evaporation Loss	0.000	0.000
Exfiltration Loss	0.000	0.000
Initial Stored Volume	0.000	0.000
Final Stored Volume	0.041	0.409
Continuity Error (%)	-0.017	

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Highest Flow Instability Indexes  
\*\*\*\*\*  
Link Pipe\_34 (3)

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Routing Time Step Summary  
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Minimum Time Step	: 1.00 sec
Average Time Step	: 1.00 sec
Maximum Time Step	: 1.00 sec
Percent in Steady State	: 0.00
Average Iterations per Step	: 2.00
Percent Not Converging	: 0.05

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Subcatchment Runoff Summary  
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Subcatchment	Total Precip mm	Total Runon mm	Total Evap mm	Total Infil mm	Total Runoff mm	Total Runoff 10 <sup>6</sup> ltr	Peak Runoff LPS	Runoff Coeff
EXT1	95.52	0.00	0.00	41.82	52.63	0.04	16.91	0.551
EXT2	95.52	0.00	0.00	11.35	83.12	0.05	20.73	0.870
ST104A	95.52	0.00	0.00	6.56	87.63	0.13	50.18	0.917
ST107A	95.52	0.00	0.00	14.94	79.28	0.30	115.66	0.830
ST108A	95.52	0.00	0.00	0.00	94.37	0.38	142.13	0.988

ST108B	95.52	0.00	0.00	0.00	94.37	0.34	128.25	0.988
ST108C	95.52	0.00	0.00	0.00	94.35	0.04	16.57	0.988
ST108D	95.52	0.00	0.00	0.00	94.37	0.05	18.20	0.988
ST108E	95.52	0.00	0.00	0.00	94.08	0.03	9.57	0.985
ST108F	95.52	0.00	0.00	23.30	70.82	0.27	102.04	0.741
ST109A	95.52	0.00	0.00	0.00	93.98	0.01	4.76	0.984
ST109B	95.52	0.00	0.00	0.00	94.28	0.05	19.11	0.987
ST109C	95.52	0.00	0.00	0.00	94.29	0.06	20.63	0.987
ST110A	95.52	0.00	0.00	38.83	54.74	0.04	12.92	0.573
ST110B	95.52	0.00	0.00	0.00	93.99	0.03	11.23	0.984
ST110C	95.52	0.00	0.00	0.00	93.99	0.03	10.41	0.984
ST110D	95.52	0.00	0.00	38.83	54.74	0.04	12.82	0.573
ST111A	95.52	0.00	0.00	11.35	82.89	0.20	77.09	0.868
ST111B	95.52	0.00	0.00	0.00	94.00	0.04	13.13	0.984
ST111C	95.52	0.00	0.00	5.96	88.16	0.04	14.64	0.923
ST507A	95.52	0.00	0.00	11.34	82.85	0.05	17.48	0.867
ST508A	95.52	0.00	0.00	38.83	55.31	0.19	79.58	0.579

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Node Depth Summary  
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Node	Type	Average Depth Meters	Maximum Depth Meters	Maximum HGL Meters	Time of Max Occurrence days hr:min	Reported Max Depth Meters
100	JUNCTION	0.44	0.52	99.92	0 06:53	0.52
101	JUNCTION	0.41	0.74	100.34	0 06:53	0.74
102	JUNCTION	0.39	0.76	100.54	0 06:52	0.76
103	JUNCTION	0.45	1.94	101.97	0 06:52	1.93
104	JUNCTION	0.41	1.90	101.98	0 06:52	1.89
105	JUNCTION	0.34	1.76	101.99	0 06:52	1.75
106	JUNCTION	0.29	1.65	101.99	0 06:52	1.64
107	JUNCTION	0.12	1.20	102.00	0 06:52	1.19
109	JUNCTION	0.10	1.15	101.98	0 06:52	1.14
110	JUNCTION	0.13	1.31	101.99	0 06:52	1.30
111	JUNCTION	0.25	0.59	102.02	0 06:51	0.59
EXT1-OF	OUTFALL	0.00	0.00	102.88	0 00:00	0.00
EXT2-OF	OUTFALL	0.00	0.00	104.20	0 00:00	0.00
HEADWALL	OUTFALL	0.00	0.00	98.70	0 00:00	0.00
POOLE_OF1	OUTFALL	0.00	0.00	100.10	0 00:00	0.00
POOLE_OF2	OUTFALL	0.00	0.00	101.06	0 00:00	0.00
ST104A-OF	OUTFALL	0.00	0.00	0.00	0 00:00	0.00
ST107A-OF	OUTFALL	0.00	0.00	0.00	0 00:00	0.00
108	STORAGE	2.74	4.80	102.04	0 06:48	4.79
508	STORAGE	0.46	0.95	102.01	0 06:14	0.95

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ST104A-S	STORAGE	0.05	1.71	103.23	0	06:15	1.71
ST107A-S	STORAGE	0.09	1.72	102.85	0	06:15	1.71
ST108A-S	STORAGE	0.02	0.14	118.74	0	06:20	0.14
ST108B-S	STORAGE	0.02	0.14	115.89	0	06:20	0.14
ST108C-S	STORAGE	0.02	0.14	110.54	0	06:19	0.14
ST108D-S	STORAGE	0.02	0.14	110.24	0	06:19	0.14
ST108E-S	STORAGE	0.02	0.14	107.34	0	06:19	0.14
ST109B-S	STORAGE	0.01	0.15	102.96	0	06:10	0.15
ST109C-S	STORAGE	0.01	0.16	102.97	0	06:15	0.16
ST111A-S	STORAGE	0.23	2.32	104.18	0	06:24	2.32
ST111C-S	STORAGE	0.01	0.08	102.03	0	06:15	0.08
ST507A-S	STORAGE	0.06	1.87	103.44	0	06:17	1.87
TANK	STORAGE	0.38	1.87	101.97	0	06:52	1.86

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Node Inflow Summary  
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Node	Type	Maximum Lateral Inflow LPS	Maximum Total Inflow LPS	Time of Max Occurrence days hr:min	Lateral Inflow Volume 10^6 ltr	Total Inflow Volume 10^6 ltr	Flow Balance Error Percent
100	JUNCTION	175.00	311.33	0 06:53	15.1	16.9	0.017
101	JUNCTION	0.00	136.63	0 06:52	0	1.83	-0.075
102	JUNCTION	0.00	136.71	0 06:52	0	1.83	0.036
103	JUNCTION	0.00	457.04	0 06:14	0	1.71	-0.003
104	JUNCTION	0.00	457.02	0 06:14	0	1.71	-0.145
105	JUNCTION	0.00	368.34	0 06:14	0	1.49	0.141
106	JUNCTION	0.00	206.19	0 06:14	0	0.707	-0.315
107	JUNCTION	0.00	159.99	0 06:15	0	0.569	0.194
109	JUNCTION	4.76	44.51	0 06:10	0.0127	0.119	0.473
110	JUNCTION	47.37	74.61	0 06:14	0.139	0.139	0.082
111	JUNCTION	13.13	45.04	0 06:15	0.0351	0.275	0.493
EXT1-OF	OUTFALL	16.91	16.91	0 06:15	0.0354	0.0354	0.000
EXT2-OF	OUTFALL	20.73	20.73	0 06:15	0.053	0.053	0.000
HEADWALL	OUTFALL	0.00	311.33	0 06:54	0	16.9	0.000
POOLE_OF1	OUTFALL	0.00	0.66	0 01:27	0	0.054	0.000
POOLE_OF2	OUTFALL	0.00	0.30	0 05:14	0	0.0203	0.000
ST104A-OF	OUTFALL	0.00	0.00	0 00:00	0	0	0.000
ST107A-OF	OUTFALL	0.00	0.00	0 00:00	0	0	0.000
108	STORAGE	102.04	240.16	0 06:13	0.271	1.14	-0.083
508	STORAGE	79.58	79.58	0 06:15	0.186	0.186	0.039
ST104A-S	STORAGE	50.18	50.18	0 06:15	0.131	0.131	0.004
ST107A-S	STORAGE	115.66	115.66	0 06:15	0.296	0.296	0.014
ST108A-S	STORAGE	142.13	142.13	0 06:15	0.381	0.381	-0.001

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ST108B-S	STORAGE	128.25	128.25	0 06:15	0.344	0.344	-0.001
ST108C-S	STORAGE	16.57	16.57	0 06:15	0.0444	0.0444	-0.001
ST108D-S	STORAGE	18.20	18.20	0 06:15	0.0488	0.0488	-0.001
ST108E-S	STORAGE	9.57	9.57	0 06:10	0.0256	0.0256	-0.000
ST109B-S	STORAGE	19.11	19.11	0 06:10	0.0512	0.0512	-0.000
ST109C-S	STORAGE	20.63	20.63	0 06:15	0.0553	0.0553	-0.000
ST111A-S	STORAGE	77.09	77.09	0 06:15	0.201	0.201	-0.001
ST111C-S	STORAGE	14.64	14.64	0 06:15	0.0384	0.0384	0.078
ST507A-S	STORAGE	17.48	17.48	0 06:15	0.0451	0.0451	-0.002
TANK	STORAGE	0.00	548.41	0 06:15	0	1.89	-0.001

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Node Surcharge Summary  
\*\*\*\*\*

Surcharging occurs when water rises above the top of the highest conduit.

Node	Type	Hours Surcharged	Max. Height Above Crown Meters	Min. Depth Below Rim Meters
104	JUNCTION	4.28	1.156	1.592
105	JUNCTION	3.62	1.092	1.645
106	JUNCTION	3.40	1.058	1.324
107	JUNCTION	2.19	0.747	1.220
109	JUNCTION	2.28	0.773	2.522
110	JUNCTION	2.81	0.933	2.506

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Node Flooding Summary  
\*\*\*\*\*

No nodes were flooded.

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Storage Volume Summary  
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Storage Unit	Average Volume 1000 m3	Avg Pcnt Full	Evap Pcnt Loss	Exfil Pcnt Loss	Maximum Volume 1000 m3	Max Pcnt Full	Time of Max Occurrence days hr:min	Maximum Outflow LPS
108	0.288	79	0	0	0.364	100	0 06:13	210.40
508	0.030	55	0	0	0.053	99	0 06:13	79.31



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ST104A-S	0.000	0	0	0	0.000	0	0	00:00	49.96
ST107A-S	0.000	0	0	0	0.000	0	0	00:00	114.99
ST108A-S	0.010	6	0	0	0.141	86	0	06:20	30.68
ST108B-S	0.008	6	0	0	0.126	85	0	06:20	28.40
ST108C-S	0.001	4	0	0	0.015	80	0	06:19	4.26
ST108D-S	0.001	5	0	0	0.017	83	0	06:19	4.30
ST108E-S	0.001	5	0	0	0.009	85	0	06:19	2.17
ST109B-S	0.000	0	0	0	0.000	0	0	00:00	19.11
ST109C-S	0.000	0	0	0	0.000	0	0	00:00	20.63
ST111A-S	0.003	3	0	0	0.058	53	0	06:24	17.36
ST111C-S	0.000	0	0	0	0.000	0	0	00:00	14.64
ST507A-S	0.000	0	0	0	0.004	6	0	06:17	10.93
TANK	0.175	30	0	0	0.592	100	0	06:47	137.37

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 Outfall Loading Summary  
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Outfall Node	Flow Freq Pcmt	Avg Flow LPS	Max Flow LPS	Total Volume 10^6 ltr
EXT1-OF	30.18	1.36	16.91	0.035
EXT2-OF	45.84	1.34	20.73	0.053
HEADWALL	100.00	196.13	311.33	16.945
POOLE_OF1	95.39	0.65	0.66	0.054
POOLE_OF2	78.72	0.30	0.30	0.020
ST104A-OF	0.00	0.00	0.00	0.000
ST107A-OF	0.00	0.00	0.00	0.000
System	50.02	199.78	316.10	17.108

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 Link Flow Summary  
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Link	Type	Maximum  Flow  LPS	Time of Max Occurrence days hr:min	Maximum  Veloc  m/sec	Max/Full Flow	Max/Full Depth
Pipe_13	CONDUIT	311.33	0 06:54	1.60	0.74	0.54
Pipe_14	CONDUIT	210.46	0 06:14	1.40	1.10	1.00
Pipe_14_(1)	CONDUIT	362.62	0 06:14	1.29	1.31	1.00
Pipe_15	CONDUIT	45.52	0 06:11	1.24	0.26	1.00

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Pipe_16	CONDUIT	64.13	0 06:14	0.94	0.37	1.00
Pipe_17	CONDUIT	45.00	0 06:15	1.00	0.31	0.99
Pipe_21	CONDUIT	457.04	0 06:14	1.28	1.21	1.00
Pipe_23	CONDUIT	79.01	0 06:15	1.65	2.66	1.00
Pipe_26	CONDUIT	136.33	0 06:53	1.08	1.07	0.74
Pipe_27	CONDUIT	210.40	0 06:14	1.32	1.35	1.00
Pipe_29	CONDUIT	158.73	0 06:14	1.45	0.79	1.00
Pipe_31	CONDUIT	136.63	0 06:52	0.88	1.07	0.94
Pipe_34	CONDUIT	458.73	0 06:15	1.35	1.18	1.00
ST104A-T	CHANNEL	0.00	0 00:00	0.00	0.00	0.00
ST107A-T	CHANNEL	0.00	0 00:00	0.00	0.00	0.00
ST111A-T	CHANNEL	0.00	0 00:00	0.00	0.00	0.00
ST111B-T	CHANNEL	0.00	0 00:00	0.00	0.00	0.00
ST507A-T	CHANNEL	0.00	0 00:00	0.00	0.00	0.00
W1	CONDUIT	0.00	0 00:00	0.00	0.00	0.00
OR1	ORIFICE	32.79	0 06:48			1.00
OR2	ORIFICE	55.72	0 06:52			1.00
OR3	ORIFICE	48.25	0 06:52			1.00
ST104A-O	ORIFICE	49.96	0 06:15			1.00
ST107A-O	ORIFICE	114.99	0 06:15			1.00
ST109B-O	ORIFICE	19.11	0 06:10			0.76
ST109C-O	ORIFICE	20.63	0 06:15			0.80
ST111A-O	ORIFICE	17.36	0 06:24			1.00
ST111C-O	ORIFICE	7.32	0 06:15			0.40
ST111C-O1	ORIFICE	7.32	0 06:15			0.40
OL1	DUMMY	0.66	0 01:27			
OL2	DUMMY	0.30	0 05:14			
ST108A-O	DUMMY	30.68	0 06:20			
ST108B-O	DUMMY	28.40	0 06:20			
ST108C-O	DUMMY	4.26	0 06:19			
ST108D-O	DUMMY	4.30	0 06:19			
ST108E-O	DUMMY	2.17	0 06:19			
ST507A-O	DUMMY	10.93	0 06:17			

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 Flow Classification Summary  
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Conduit	Adjusted /Actual Length	Fraction of Time in Flow Class								
		Dry	Up Dry	Down Dry	Sub Crit	Sup Crit	Up Crit	Down Crit	Norm Ltd	Inlet Ctrl
Pipe_13	1.00	0.00	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00
Pipe_14	1.00	0.04	0.00	0.00	0.39	0.00	0.00	0.56	0.02	0.00
Pipe_14_(1)	1.00	0.04	0.00	0.00	0.51	0.00	0.00	0.45	0.06	0.00
Pipe_15	1.00	0.04	0.00	0.00	0.36	0.00	0.00	0.61	0.19	0.00

Pipe_16	1.00	0.04	0.00	0.00	0.31	0.00	0.00	0.65	0.05	0.00
Pipe_17	1.00	0.04	0.00	0.00	0.15	0.00	0.00	0.80	0.11	0.00
Pipe_21	1.00	0.04	0.00	0.00	0.94	0.00	0.00	0.02	0.14	0.00
Pipe_23	1.00	0.62	0.00	0.00	0.05	0.00	0.00	0.33	0.00	0.00
Pipe_26	1.00	0.00	0.09	0.00	0.72	0.00	0.00	0.19	0.54	0.00
Pipe_27	1.00	0.05	0.20	0.00	0.42	0.00	0.02	0.32	0.06	0.00
Pipe_29	1.00	0.04	0.00	0.00	0.34	0.00	0.00	0.63	0.17	0.00
Pipe_31	1.00	0.07	0.00	0.00	0.90	0.00	0.00	0.02	0.00	0.00
Pipe_34	1.00	0.04	0.00	0.00	0.89	0.07	0.00	0.00	0.01	0.00
ST104A-T	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ST107A-T	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ST111A-T	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ST111B-T	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
ST507A-T	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W1	1.00	1.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

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 Conduit Surcharge Summary  
 \*\*\*\*\*

Conduit	----- Both Ends	Hours Full Upstream	----- Dnstream	Hours Above Full Normal Flow	Hours Capacity Limited
Pipe_14	3.40	3.40	3.62	0.05	0.01
Pipe_14_(1)	3.61	3.62	4.28	0.07	0.05
Pipe_15	2.28	2.28	4.28	0.01	0.01
Pipe_16	2.81	2.81	3.40	0.01	0.01
Pipe_17	0.01	0.01	2.19	0.01	0.01
Pipe_21	4.31	4.31	4.63	0.05	0.03
Pipe_23	0.34	0.42	0.44	0.19	0.01
Pipe_26	0.01	0.01	0.01	0.72	0.01
Pipe_27	3.71	3.71	4.69	0.06	0.16
Pipe_29	2.19	2.19	3.40	0.01	0.01
Pipe_31	0.01	0.16	0.01	0.73	0.01
Pipe_34	4.68	4.68	4.75	0.04	0.02

Analysis begun on: Thu Mar 23 14:03:56 2017  
 Analysis ended on: Thu Mar 23 14:03:58 2017  
 Total elapsed time: 00:00:02

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
5731 HAZELDEAN ROAD**

Appendix C Stormwater Management  
March 22, 2017

**C.4 OIL/GRIT SEPARATOR SIZING CALCULATIONS**



## Stormceptor Design Summary

### PCSWMM for Stormceptor

#### Project Information

Date	11/4/2016
Project Name	5731 Hazeldean
Project Number	160401195
Location	Ottawa, ON

#### Designer Information

Company	Stantec Consulting Ltd.
Contact	N/A

#### Rainfall

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

#### Notes

N/A
-----

#### Water Quality Objective

TSS Removal (%)	80
-----------------	----

#### Drainage Area

Total Area (ha)	2.72
Imperviousness (%)	70

#### Upstream Storage

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

The Stormceptor System model STC 3000 achieves the water quality objective removing 80% TSS for a CLOCA (clay, silt and sand) particle size distribution.

#### Stormceptor Sizing Summary

Stormceptor Model	TSS Removal %
STC 300	60
STC 750	73
STC 1000	73
STC 1500	74
STC 2000	79
<b>STC 3000</b>	<b>80</b>
STC 4000	84
STC 5000	84
STC 6000	87
STC 9000	90
STC 10000	90
STC 14000	92



### Particle Size Distribution

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

CLOCA (clay, silt and sand)							
Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s	Particle Size µm	Distribution %	Specific Gravity	Settling Velocity m/s
850	3.3	2.65	0.1465	50	3.9	2.65	0.0022
425	23.4	2.65	0.0698	36	2.6	2.65	0.0012
300	17.5	2.65	0.0439	22	1.3	2.65	0.0004
250	6.5	2.65	0.0335	12	1.9	2.65	0.0004
212	6.5	2.65	0.0259	9	0	2.65	0.0004
150	11.7	2.65	0.0145	6.5	1.3	2.65	0.0004
125	5.2	2.65	0.0105	3	1.3	2.65	0.0004
100	3.9	2.65	0.0070	1.5	1.3	2.65	0.0004
75	3.9	2.65	0.0040	1	4.5	2.65	0.0004

### Stormceptor Design Notes

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

#### Inlet and Outlet Pipe Invert Elevations Differences

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

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

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Legend

- 1 AREA ID
- 1.13 0.00 RUNOFF COEFFICIENT
- STORM DRAINAGE AREA NO.
- STORM DRAINAGE BOUNDARY
- 100YEAR PONDING LIMITS
- DIRECTION OF OVERLAND FLOW
- PROPOSED STORM SEWER
- PROPOSED CATCHBASIN MANHOLE
- PROPOSED CATCHBASIN
- PROPOSED BUILDING CATCHBASIN
- PROPOSED SUB DRAIN CATCH BASIN AS PER CITY OF OTTAWA STANDARD DETAIL DRAWINGS L10 AND L11.
- PROPOSED PERFORATED SUBDRAIN
- EXISTING STORM SEWER
- EXISTING CATCHBASIN

APPROVED  REFUSED   
THIS 17 DAY OF May 20 17  
*Derick Moodie*  
Derick Moodie, Manager  
Development Review West  
Planning, Infrastructure and Economic  
Development Department, City of Ottawa

Revision	By	Appd.	YYMMDD
6	MJS	PM	17.04.12
5	MJS	PM	17.03.24
4	MJS	GR	17.03.22
3	DT	PM	17.01.27
2	MJS	PM	16.09.09
1	MJS	PM	16.04.11

File Name:	MJS	PM	MJS	16.02.08
160401195-D8.dwg	Dwn.	Chkd.	Dgn.	YYMMDD

Permit-Seal



Client/Project  
WELLINGS OF STITTSVILLE INC. AND  
EXTENDICARE (CANADA) INC.  
WELLINGS OF STITTSVILLE SENIOR'S LIVING  
AND EXTENDICARE L.T.C. STITTSVILLE  
5731 HAZELDEAN ROAD  
OTTAWA, ON

Title  
STORM DRAINAGE PLAN

Project No. 160401195 Scale 1:500

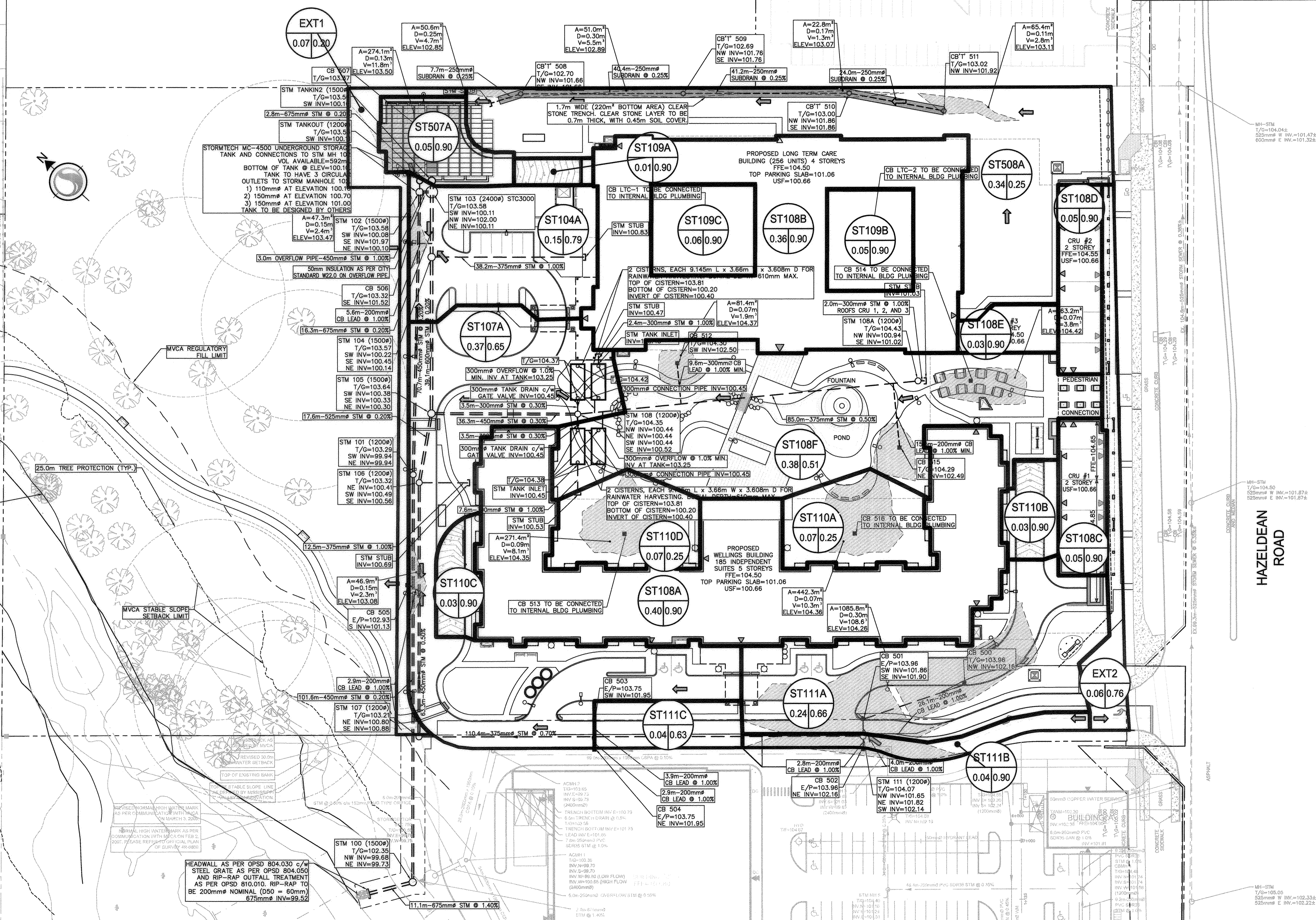
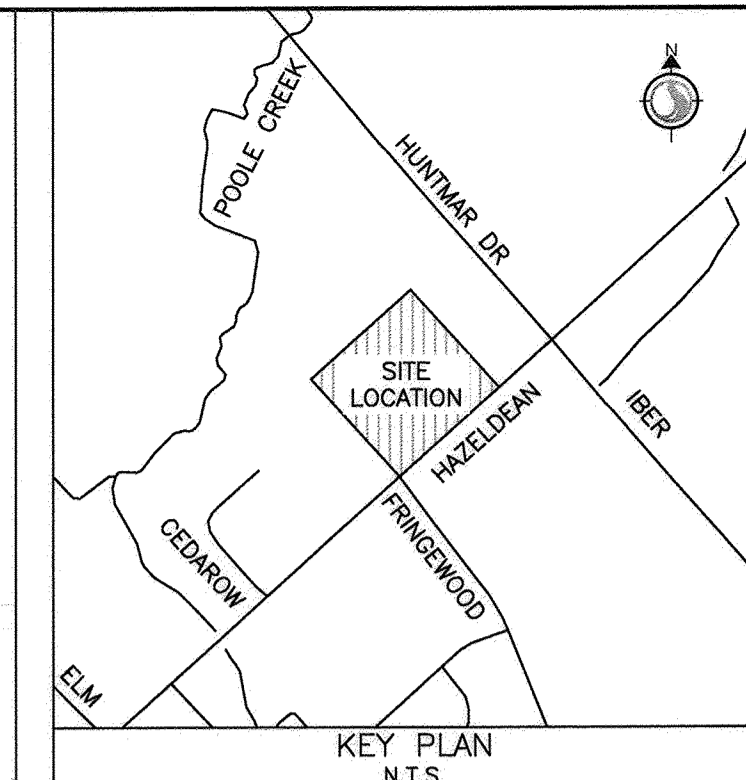
Drawing No. SD-1 Sheet 6 of 7 Revision 6 Plan # 17404

SCHEDULE OF INLET CONTROL DEVICES

STRUCTURE ID	DRAINAGE AREA	ORIFICE DIA. (mm)	100 YEAR HEAD (m)	100 YEAR FLOW (L/s)
CB 501	ST111A	IPX HF (3.07)	101.86	2.32
CB 503/504	ST111C	200mm PIPE (42)	101.85	0.08
CB 505	ST107A	200mm PIPE	101.13	1.72
CB 506	ST104A	IPX HF (5.57)	101.82	1.71
CB 507	ST507A	IPX LMF 95	101.57	1.87
SWM TANK	-	150	101.00	0.97
SWM TANK	-	150	100.70	1.27
SWM TANK	-	110	100.10	1.87

SCHEDULE OF ROOF RELEASE RATES

BUILDING	Head (m)	ICD TYPE	100R RELEASE RATE (L/s)
WELLINGS BUILDING	0.14	ZURN Z-105-5	29.9
LONG TERM CARE BUILDING	0.14	ZURN Z-105-5	27.7
CRU 1	0.14	ZURN Z-105-5	4.2
CRU 2	0.14	ZURN Z-105-5	4.2
CRU 3	0.14	ZURN Z-105-5	2.1



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2017/04/12 2:54 PM By: Short, Mike

**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix C Stormwater Management  
March 29, 2022

**C.8 WATER BALANCE CALCULATIONS**

**Project #160401511 - 20 Cedarow Drive**

**Infiltration calculations**

Required Infiltration Rate (KWMSS) 73 mm/yr  
Site Area 2.29 ha  
Pre-Development Imperviousness 0 %  
Pre-Development Infiltration 1674.5 m<sup>3</sup>/yr

Post Development Imperviousness 66.3 %  
Post Development Pervious Area 0.69 ha  
Post Development Infiltration in Pervious Areas 503.4 m<sup>3</sup>/yr

**Post Development Infiltration Volume Req. 1171.1 m<sup>3</sup>/yr**

**Determine Volume of Water to be Sequestered in Infiltration Trench (Assume storage up to 25mm event)**

Area Tributary to Infiltration Trench 3517 m<sup>2</sup>  
Impervious Area to Infiltration Trench 3517 m<sup>2</sup> 100.0 % *Impervious*  
Total Depth of Annual Runoff to Infiltration Trench 760.5 mm/yr (910.5mm/yr annual precipitation less urban ET of 150mm/yr)  
**Volume of Runoff from Impervious Area to Infiltration Trench 2674.7 m<sup>3</sup>/yr** for events with rainfall <25mm

In order to store up to 25mm from catchment area:

Max. Capacity Required (25mm)= 88 m<sup>3</sup> volume of runoff  
Trench Length (m) 40.00 m 40% Trench Porosity  
Trench Width (m) 5.50 m  
Trench Height (m) 1.00 m  
**Volume Provided 88 m<sup>3</sup>**



**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix D Geotechnical Investigation  
March 29, 2022

**Appendix D      GEOTECHNICAL INVESTIGATION**

Geotechnical  
Engineering

Environmental  
Engineering

Hydrogeology

Geological  
Engineering

Materials Testing

Building Science

Archaeological Services

## Geotechnical Investigation

Proposed Mixed-Use Development  
Wellings of Stittsville - Phase 2  
20 Cedarow Court  
Ottawa, Ontario

Prepared For

Nautical Lands Group

### Paterson Group Inc.

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

Tel: (613) 226-7381  
Fax: (613) 226-6344  
[www.patersongroup.ca](http://www.patersongroup.ca)

March 7, 2019

Report PG4772-1

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## Appendices

- Appendix 1      Soil Profile and Test Data Sheets  
                     Symbols and Terms  
                     Analytical Testing Results
- Appendix 2      Figure 1 - Key Plan  
                     Figures 2 to 4 - Slope Stability Analysis Sections  
                     Drawing PG4772-1 - Test Hole Location Plan

## 1.0 Introduction

Paterson Group (Paterson) was commissioned by Nautical Lands Group to conduct a geotechnical investigation for the proposed mixed-use development to be located at 20 Cedarow Court in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- Determine the subsurface conditions by means of boreholes.
- Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project. This report contains geotechnical findings and includes recommendations pertaining to the design and construction of the proposed development as understood at the time of writing this report.

## 2.0 Proposed Development

Based on the available drawings, it is our understanding that the proposed development will consist of four, five (5) storey mixed-use buildings with a shared underground parking level occupying the majority of the footprint of the subject site. The buildings are understood to include retail, office space and residential units. A one (1) storey restaurant building is also proposed within the centre of the site. At-grade parking areas, access lanes and landscaped areas are also anticipated a part of the development. It is anticipated that the proposed development will be municipally serviced.

## **3.0 Method of Investigation**

### **3.1 Field Investigation**

#### **Field Program**

The field program for the current investigation was carried out from January 14, 2019 to January 18, 2019. At that time, 29 boreholes were drilled to a maximum depth of 4 m below existing grade. The borehole locations were distributed in a manner to provide general coverage of the proposed development. The locations of the boreholes are shown on Drawing PG4772-1 - Test Hole Location Plan included in Appendix 2.

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

#### **Sampling and In Situ Testing**

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

Standard Penetration Tests (SPT) were conducted and recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength tests were conducted in cohesive soils with a field vane apparatus.

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

## **Groundwater**

Flexible polyethylene standpipes were installed in the majority of the boreholes to permit groundwater results subsequent to the sampling program completion. Monitoring wells were installed in BH 4, BH 9, BH 15, BH 22, and BH 27 to provide general site coverage as part of our hydrogeological study. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

## **Sample Storage**

All samples will be stored in the laboratory for a period of one month after issuance of this report at which time the samples will be discarded unless otherwise directed.

### **3.2 Field Survey**

The borehole locations were selected by Paterson taking in consideration site features. The ground surface at the test pit locations was located and surveyed by Annis, O'Sullivan, Vollebakk LTD. It is understood that the ground surface elevations at the borehole locations were referenced to a geodetic datum. The locations and ground surface elevation at the boreholes are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

### **3.3 Laboratory Testing**

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs. All samples will be stored in the laboratory for a period of one month after the issuance of this report. They will then be discarded unless we are otherwise directed.

### **3.4 Analytical Testing**

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the sulphate potential against subsurface concrete structures. The results are discussed further in Subsection 6.7.

## **4.0 Observations**

### **4.1 Surface Conditions**

The subject site is currently undeveloped and grass covered with a tree-line located along the west boundary line of Cedarow Court. The ground surface across the site is relatively flat and approximately 1 m lower than adjacent properties and Hazeldean Road. Poole Creek ravine runs along the western border of the subject site approximately 3 m below the subject site.

The subject site is bordered by an active construction site for Phase 1 of the Wellings of Stittsville development along the north, Hazeldean Road along the east, and commercial buildings at the edge of Cedarow Court along the south.

### **4.2 Subsurface Profile**

#### **Overburden**

The subsurface profile at the borehole locations consists of topsoil overlying a hard to very stiff silty clay crust followed by a grey, very stiff to stiff silty clay layer. Glacial till was encountered below the silty clay layer consisting of compact silty sand to sandy silt with clay, gravel, cobbles and boulders. A deposit of very stiff to hard clayey silt was encountered below the topsoil in BH 17, BH 18, BH 24, BH 25, BH 26, and BH 27. Practical refusal to augering on inferred bedrock was encountered in all boreholes at depths ranging between 1.6 to 4.0 m. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

#### **Bedrock**

Based on available geological mapping, the subject site consists of interbedded dolostone and limestone of the Gull River formation and an approximate drift thickness of 2 to 15 m.

### **4.3 Groundwater**

The measured groundwater levels at the borehole locations are presented in Table 1. Groundwater readings recorded in flexible piezometers could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater level can also be estimated based on observations of the recovered soil samples, such as the moisture level, soil consistency and colouring. Based on these observations, the long-term groundwater level is anticipated at a depth ranging between 2.5 to 3.5 m below existing grade. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.



<b>Table 1 - Groundwater Readings Summary</b>				
<b>Test Hole Number</b>	<b>Ground Elevation (m)</b>	<b>Groundwater Levels (m)</b>		<b>Recording Date</b>
		<b>Depth</b>	<b>Elevation</b>	
BH 1	104.37	DRY	n/a	January 29, 2019
BH 2	103.59	3.05	100.54	January 29, 2019
BH 3	103.55	1.81	101.74	January 29, 2019
BH 4	103.28	3.05	100.23	January 29, 2019
BH 5	103.45	3.05	100.40	January 29, 2019
BH 6	103.49	3.04	100.45	January 29, 2019
BH 7	103.41	DRY	n/a	January 29, 2019
BH 8	103.46	DRY	n/a	January 29, 2019
BH 9	103.42	3.17	100.25	January 29, 2019
BH 10	103.31	2.18	101.13	January 29, 2019
BH 11	103.44	DRY	n/a	January 29, 2019
BH 12	103.58	DRY	n/a	January 29, 2019
BH 13	103.55	DRY	n/a	January 29, 2019
BH 14	104.18	DRY	n/a	January 29, 2019
BH 15	103.65	2.92	100.73	January 29, 2019
BH 16	103.66	DRY	n/a	January 29, 2019
BH 17	104.19	DRY	n/a	January 29, 2019
BH 18	104.15	DRY	n/a	January 29, 2019
BH 19	103.78	DRY	n/a	January 29, 2019
BH 20	103.59	DRY	n/a	January 29, 2019
BH 21	103.58	DRY	n/a	January 29, 2019
BH 22	103.65	DRY	n/a	January 29, 2019
BH 23	103.87	2.62	101.25	January 29, 2019
BH 24	104.04	2.55	101.49	January 29, 2019
BH 25	104.07	1.68	102.39	January 29, 2019
BH 26	104.30	DRY	n/a	January 29, 2019
BH 27	103.97	DRY	n/a	January 29, 2019
BH 28	103.78	DRY	n/a	January 29, 2019
BH 29	103.71	DRY	n/a	January 29, 2019

**Note:** The ground surface elevation at the borehole locations was provided by Annis, O'Sullivan, Vollebakk Ltd.

## 5.0 Discussion

### 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed structures will be founded on conventional shallow foundations placed on an undisturbed, hard to very stiff silty clay, compact to dense glacial till and/or clean, surface sounded bedrock bearing surface. Alternatively, conventional shallow footings can be placed over a near vertical, zero entry, concrete in-filled trenches extending to a clean, surface sounded bedrock bearing surface.

Permissible grade raise restriction areas are also required due to the silty clay deposit. A permissible grade raise restriction of **2 m** is recommended for areas where settlement sensitive structures are founded over the silty clay deposit.

Depending on the extent of the underground parking garage and potential grade raise, the bedrock may be encountered during excavation and construction. All contractors should be prepared for bedrock removal within the subject site.

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

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

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

### 5.2 Site Grading and Preparation

#### Stripping Depth

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

#### Bedrock Removal

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

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

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

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

Excavation side slopes in sound bedrock can be excavated almost vertical side walls. A minimum 1 m horizontal ledge, should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

### **Vibration Considerations**

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

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

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

## Fill Placement

Fill placed for grading beneath the structure(s) or other settlement sensitive areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The engineered fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If the non-specified backfill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material's SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

## 5.3 Foundation Design

### Bearing Resistance Values (Shallow Foundation)

Footings for the proposed buildings can be designed with the following bearing resistance values presented in Table 2.

<b>Table 2 - Bearing Resistance Values</b>		
<b>Bearing Surface</b>	<b>Bearing Resistance Value at SLS (kPa)</b>	<b>Factored Bearing Resistance Value at ULS (kPa)</b>
Very stiff to hard silty clay	150	250
Compact to dense glacial till	200	300
<b>Lean Concrete In-filled Trenches</b>	-	<b>1,500</b>
Clean, Surface Sounded Limestone Bedrock	-	1,500
<p><b>Note:</b> Strip footings, up to 3 m wide, and pad footings, up to 8 m wide, placed over an undisturbed, silty clay bearing surface can be designed using the abovenoted bearing resistance values.                      - A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.</p>		

The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

The bearing resistance values are provided on the assumption that the footings are placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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

### **Lean Concrete Filled Trenches**

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**15 MPa** 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

## **Bedrock/Soil Transition**

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

## **Lateral Support**

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an engineered fill, stiff silty clay or glacial till above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

## **Permissible Grade Raise Restriction**

Based on the current borehole information, a **permissible grade raise restriction of 2 m** is recommended for the proposed buildings and settlement sensitive structures where founded over a silty clay deposit. A post-development groundwater lowering of 0.5 m was assumed for our calculations.

## **5.4 Design for Earthquakes**

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. However, a higher site class, such as Class A or B can be provided if a site specific shear wave velocity test is completed to confirm the seismic site classification. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.

## 5.5 Basement Slab

The basement area for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

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

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

## 5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the proposed structure's basement walls. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m<sup>3</sup>.

The foundation wall is anticipated to be provided with a perimeter drainage system; therefore, the retained soils should be considered drained. For the undrained conditions, the applicable effective unit weight of the retained soil can be designed with 13 kN/m<sup>3</sup>. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure ( $P_{AE}$ ) includes both the static earth pressure component ( $P_o$ ) and the seismic component ( $\Delta P_{AE}$ ).

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

### Static Conditions

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

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

An additional pressure with a magnitude equal to  $K_o \cdot q$  and acting on the entire height of the wall should be added to the above formula for any surcharge loading,  $q$  (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure should only be applicable for static analyses and not be calculated in conjunction with the seismic loading case. Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

### Seismic Conditions

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

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

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

The peak ground acceleration, ( $a_{max}$ ), for the Ottawa area is  $0.32g$  according to OBC 2012. The vertical seismic coefficient is assumed to be zero. The earth force component ( $P_o$ ) under seismic conditions could be calculated using  $P_o = 0.5 K_o \gamma H^2$ , where  $K_o = 0.5$  for the soil conditions presented above.

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

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

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

## 5.7 Pavement Structure

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



<b>Table 3 - Recommended Flexible Pavement Structure - At-Grade Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
50	<b>Wear Course</b> - HL-3 or Superpave 12.5 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
300	<b>SUBBASE</b> - OPSS Granular B Type II
	<b>SUBGRADE</b> - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

<b>Table 4 - Recommended Flexible Pavement Structure - Access Lanes and Heavy Truck Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
40	<b>Wear Course</b> - HL-3 or Superpave 12.5 Asphaltic Concrete
50	<b>Binder Course</b> - HL-8 or Superpave 19.0 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
450	<b>SUBBASE</b> - OPSS Granular B Type II
	<b>SUBGRADE</b> - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

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

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD.

## **6.0 Design and Construction Precautions**

### **6.1 Foundation Drainage and Backfill**

#### **Foundation Drainage**

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The composite drainage system (such as Miradrain G100N, Delta Drain 6000 or an approved equivalent) is recommended to extend to the footing level. Sleeves, 150 mm diameter, at 3 m centres are recommended to be placed in the footing or at the foundation wall/footing interface for blind sided pours to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

#### **Underfloor Drainage**

Underfloor drainage is recommend to control water infiltration for the proposed structures. For design purposes, Paterson recommends 150 mm diameter PVC, corrugated, perforated pipes be placed at 3 to 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

#### **Adverse Effects of Dewatering on Adjacent Properties**

Due to the low permeability of the subsoils profile, any minor dewatering will be considered relatively minor due to the proposed building. Therefore, adverse effects to the surrounding buildings or properties are not expected with respect to any groundwater lowering.

#### **Foundation Backfill**

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls where frost heave sensitive structures, such as a concrete sidewalk, will be placed. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material may be used for this purpose. A composite drainage system, such as Delta Drain 6000, Miradrain G100 or an approved equivalent, should be placed against the foundation wall to promote drainage toward the perimeter drainage pipe.

## **6.2 Protection of Footings Against Frost Action**

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

## **6.3 Excavation Side Slopes**

### **Temporary Side Slopes**

The temporary excavation side slopes should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

A trench box is recommended to be installed at all times to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not remain exposed for extended periods of time.

## Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the excavation depths, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

<b>Table 6 - Soil Parameters</b>	
<b>Parameters</b>	<b>Values</b>
Active Earth Pressure Coefficient ( $K_a$ )	0.33
Passive Earth Pressure Coefficient ( $K_p$ )	3
At-Rest Earth Pressure Coefficient ( $K_0$ )	0.5
Dry Unit Weight ( $\gamma$ ), kN/m <sup>3</sup>	20
Effective Unit Weight ( $\gamma'$ ), kN/m <sup>3</sup>	13

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

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

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

## **6.4 Pipe Bedding and Backfill**

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

A minimum of a 150 mm layer of OPSS Granular A crushed stone should be placed for pipe bedding for sewer and water pipes for a soil subgrade. The bedding thickness should be increased to 300 mm for areas where the subgrade consists of bedrock. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the SPMDD.

The site excavated material may be placed above cover material if the excavation operations are completed in dry weather conditions and the site excavated material is approved by the geotechnical consultant. All cobbles greater than 200 mm in the longest dimension should be removed prior to the site materials being reused.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD. Within the frost zone (1.8 m below finished grade), non frost susceptible materials should be used when backfilling trenches below the original bedrock level.

Clay seals are recommended for the subject site. The seals should be a minimum of 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries, roadway intersections and at a maximum distance of every 50 m in the service trenches.

## **6.5 Groundwater Control**

### **Groundwater Control for Building Construction**

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

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

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

## **6.6 Winter Construction**

Precautions must be provided if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions in the contract documents should be provided to protect the excavation walls from freezing, if applicable.

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

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

## **6.7 Corrosion Potential and Sulphate**

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

## **6.8 Limit of Hazard Lands**

### **Field Observations**

Paterson conducted a site visit on January 13, 2019 to review the slope located along the west boundary of the subject site, assess the current slope conditions and confirm the grades provided in the existing topographic mapping. A section of Poole Creek is located within the west portion of the site and shown in Drawing PG4772-1 - Test Hole Location Plan.

Three (3) slope cross-sections were reviewed in the field as the worst case scenarios. The cross section locations are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2. Generally, the riverbanks along both sides of Poole Creek are currently well vegetated and were observed in an acceptable condition. Poole Creek was observed within a 20 to 40 m wide flood plain. The slope along the east side of Poole Creek ranged in height between 3 and 5 m with an inclination ranging between 2.3H:1V and 3.3H:1V. The upper slope was observed to be well vegetated with little to no signs of active surficial erosion.

## **Slope Stability Analysis**

### Limit of Hazard Lands

The slope condition was reviewed based on available topographic mapping along the east side slopes of Poole Creek within the west portion of the subject development. A total of 3 slope cross-sections were assessed as the worst case scenarios. The cross section locations are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

A slope stability assessment was carried out to determine the required stable slope allowance setback from the top of slope based on a factor of safety of 1.5. A toe erosion and 6 m erosion access allowances were also included in the determination of limits of hazard lands and are discussed below. The proposed limit of hazard lands (as shown on Drawing PG4772-1 - Test Hole Location Plan) includes:

- a geotechnical slope stability allowance with a factor of safety of 1.5
- a toe erosion allowance
- a 6 m erosion access allowance and top of slope



### Slope Stability Analysis

The analysis of the stability of the slope sections was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16G was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The cross-sections were analysed taking into account a groundwater level at ground surface, which represents a worse-case scenario that can be reasonably expected to occur in cohesive soils. The stability analysis assumes full saturation of the soil with groundwater flow parallel to the slope face. Subsoil conditions at the cross-sections were inferred based on the findings at borehole locations along the top of slope and general knowledge of the area's geology.

### Stable Slope Allowance

The results of the stability analysis for static conditions at Sections A through C are presented in Figures 2A to 4A in Appendix 2. All the reviewed slope sections along the subject creek were noted to be shaped to at least a 2.3H:1V. Based on the soil conditions observed and the results of the slope stability analysis, the slope stability factor of safety was calculated to be 1.5 or greater for all the slope sections which indicates that a stable slope allowance is not required for the subject slope.

The results of the analyses including seismic loading are shown in Figures 2B to 4B for the slope sections. The results indicate that the factor of safety for the sections are greater than 1.1.

It should be noted that the existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed and/or topped with an erosion control blanket be which can be placed across the exposed slope face.

#### *Toe Erosion and Erosion Access Allowance*

The toe erosion allowance for the valley corridor wall slope was based on the cohesive nature of the top layers of the subsoils, the observed current erosional activities and the width and location of the current watercourse. It should be noted that if the flood plain is measured to be greater than 20 m, no toe erosion will be required. Therefore, based on the above factors, no toe erosion allowance is considered for the subject slope.

An erosion access allowance of 6 m is required from the top of slope to ensure access is provided should future maintenance to the slope face is required. The limit of hazard lands, which includes these allowances, is indicated on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

## **6.9 Landscaping Considerations**

### **Tree Planting Restrictions**

According to the City of Ottawa Guidelines for tree planting, where a sensitive silty clay deposit is present within the vicinity of the site, tree planting restrictions should be determined. However, for this site, based on the founding medium of the underground parking level which will occupy the majority of the site, tree planting restrictions are not required from a geotechnical perspective.

## 7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- Review detailed grading plan(s) from a geotechnical perspective.
- Review groundwater conditions at the time of construction to determine if waterproofing is required.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that the construction work has been conducted in general accordance with the above recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

## 8.0 Statement of Limitations

The recommendations provided in the report are in accordance with Paterson's present understanding of the project. Paterson request permission to review the recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions encountered during construction differ from the borehole locations, Paterson requests immediate notification to permit reassessment of the recommendations provided herein.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Nautical Lands Group or their agent(s) is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

### Paterson Group Inc.



Faisal I. Abou-Seido, P.Eng.



David J. Gilbert, P.Eng.

### Report Distribution:

- Nautical Lands Group (3 copies)
- Paterson Group (1 copy)

# **APPENDIX 1**

**SOIL PROFILE AND TEST DATA SHEETS**

**SYMBOLS AND TERMS**

**ANALYTICAL TESTING RESULTS**

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

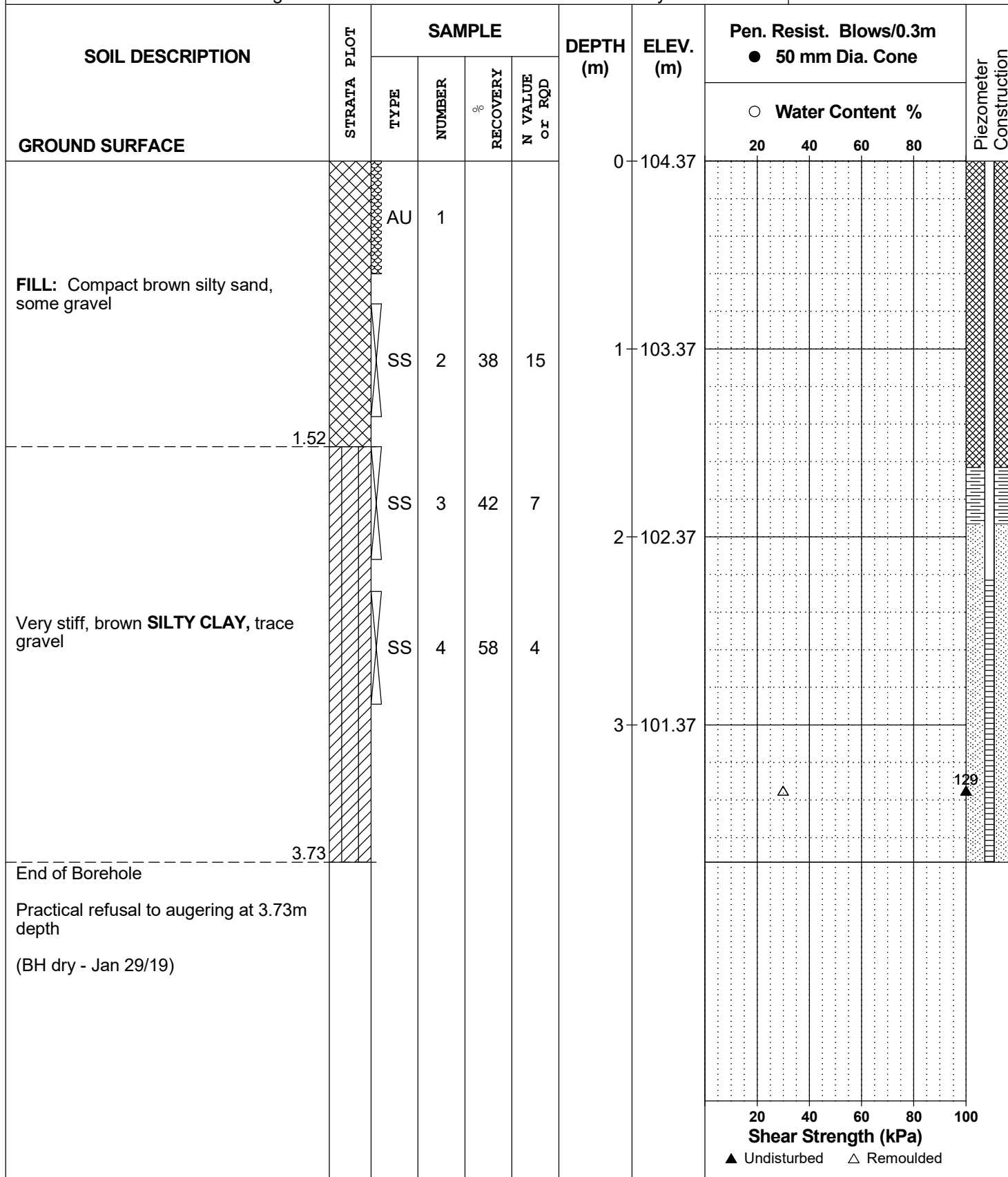
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REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



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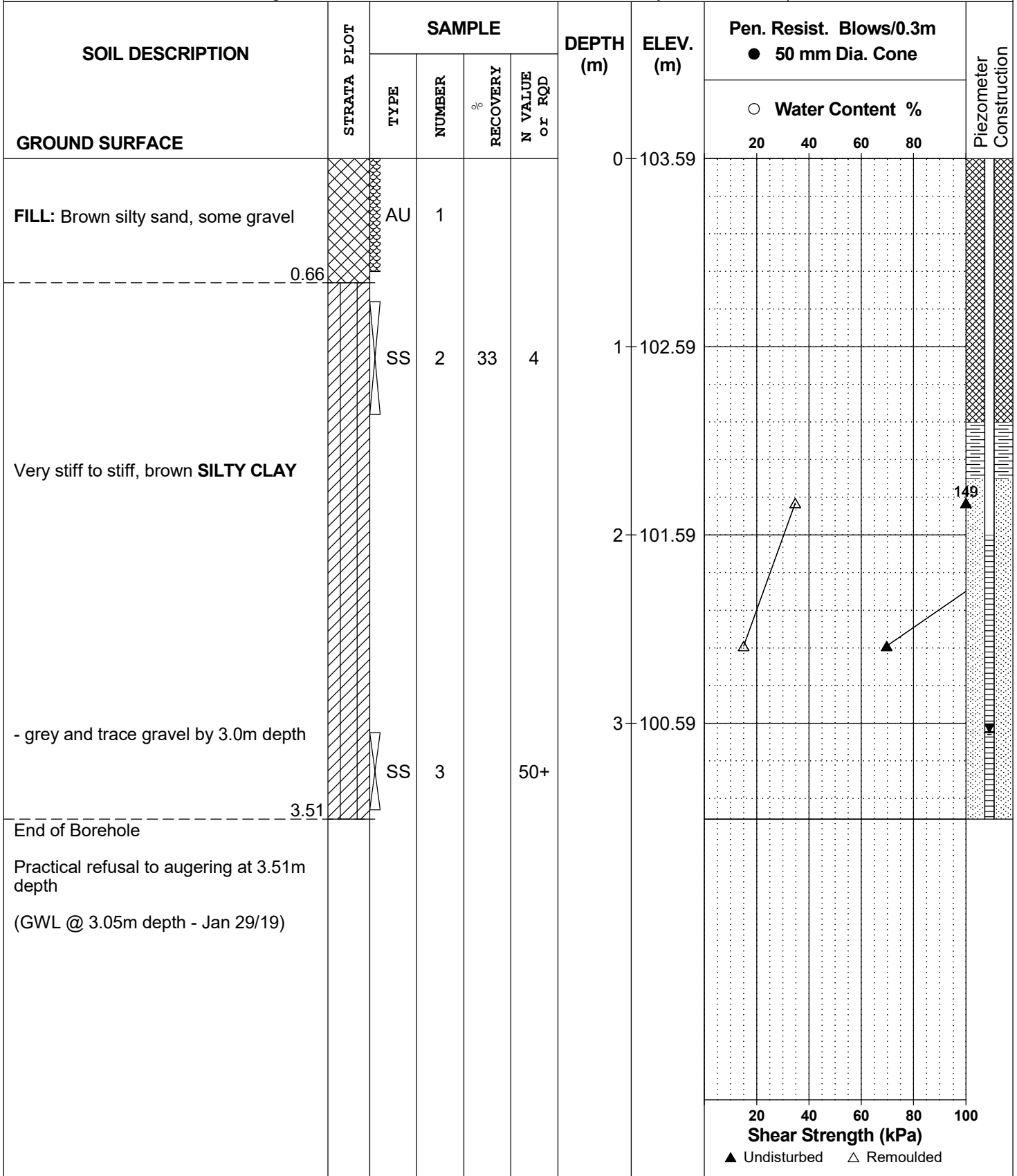
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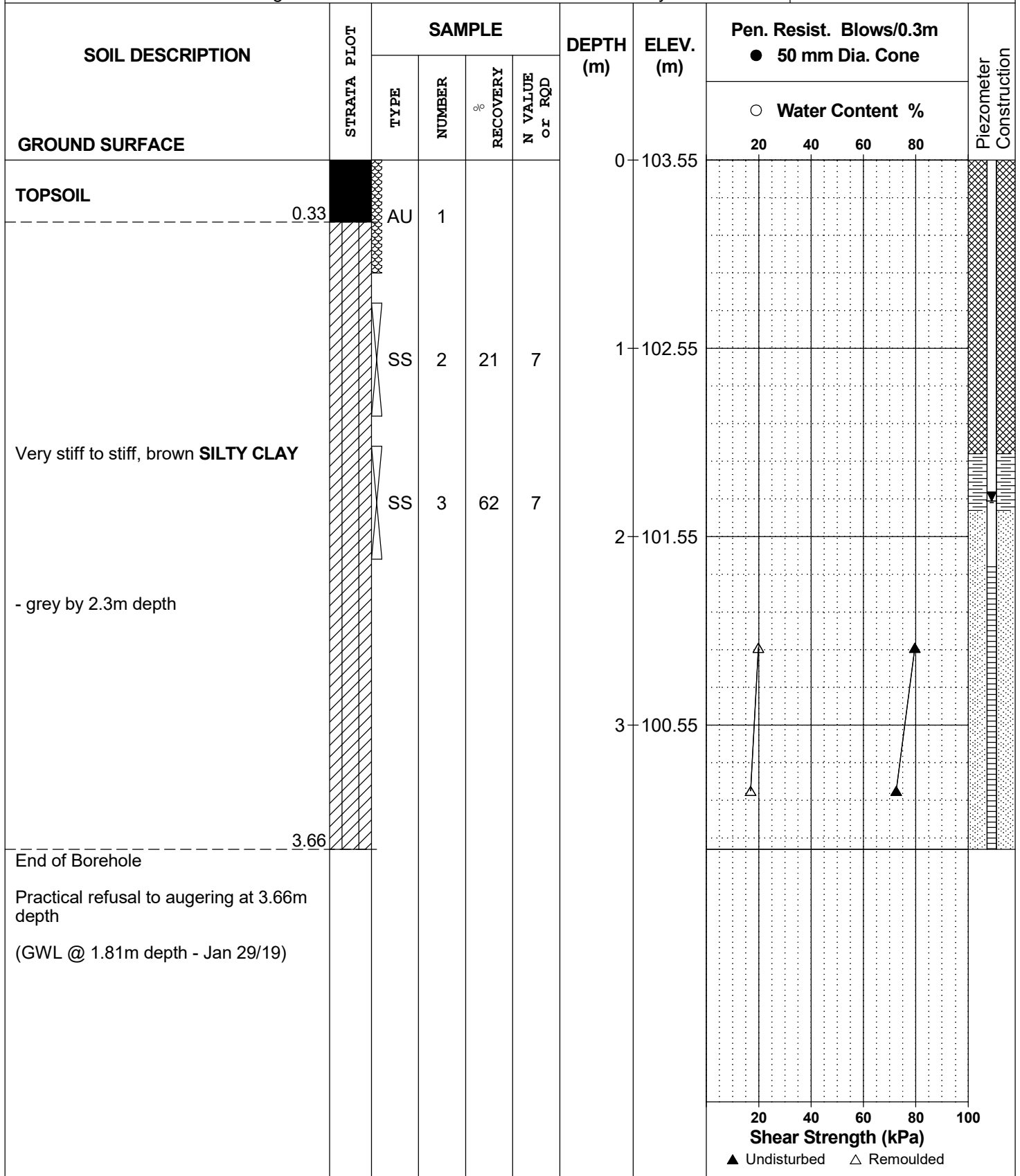
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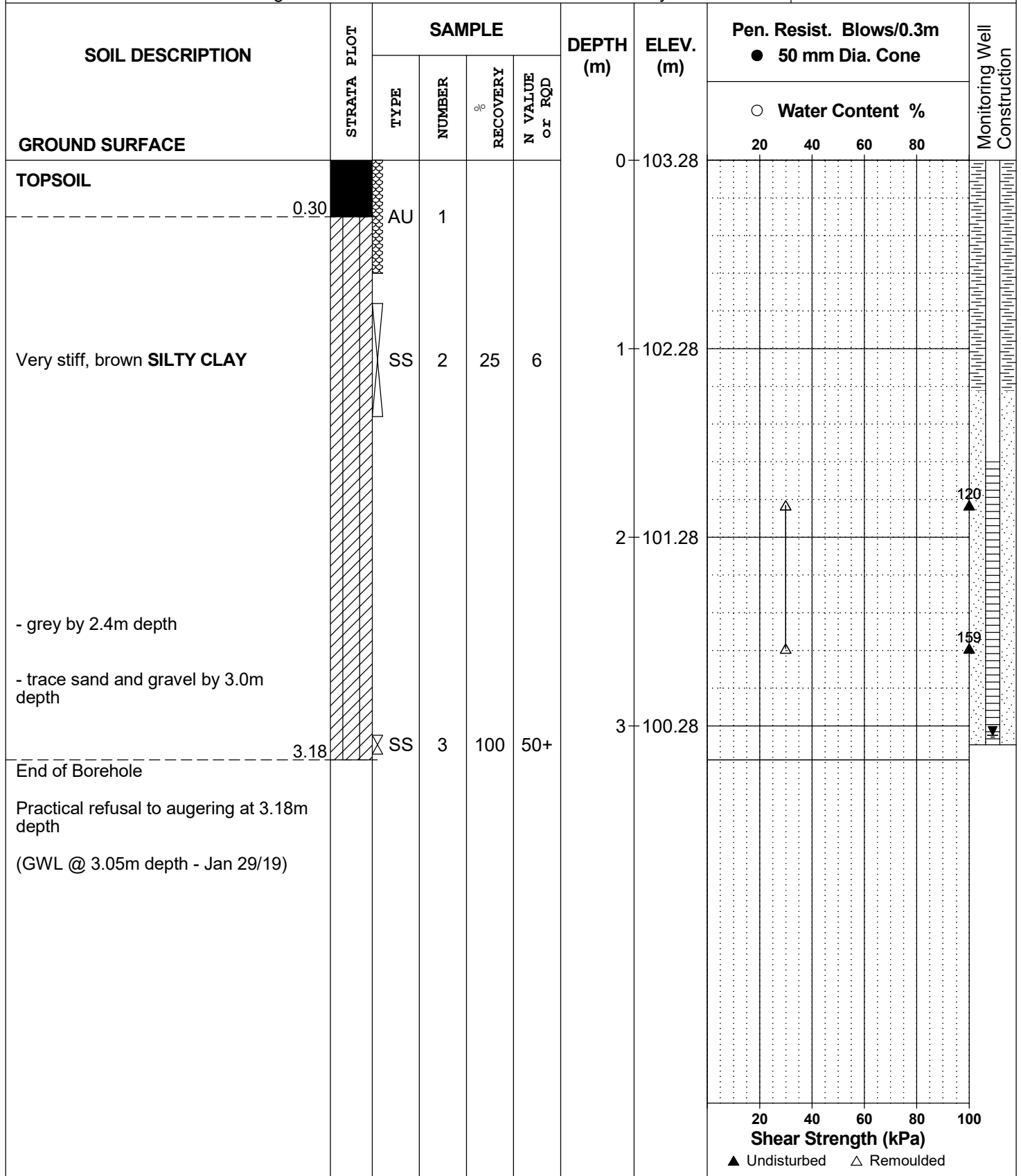
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DATE 2019 January 14



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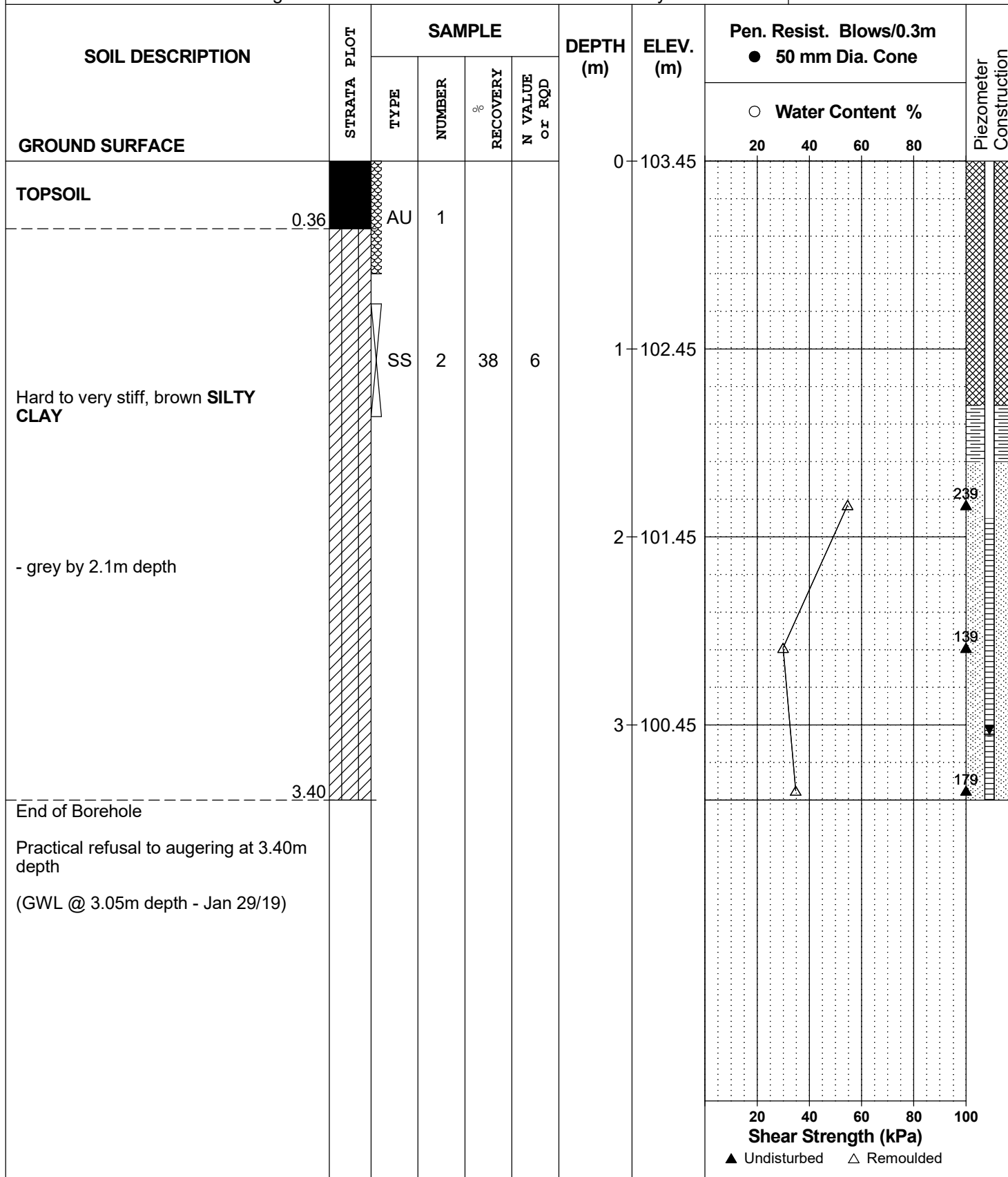
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HOLE NO. **BH 5**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



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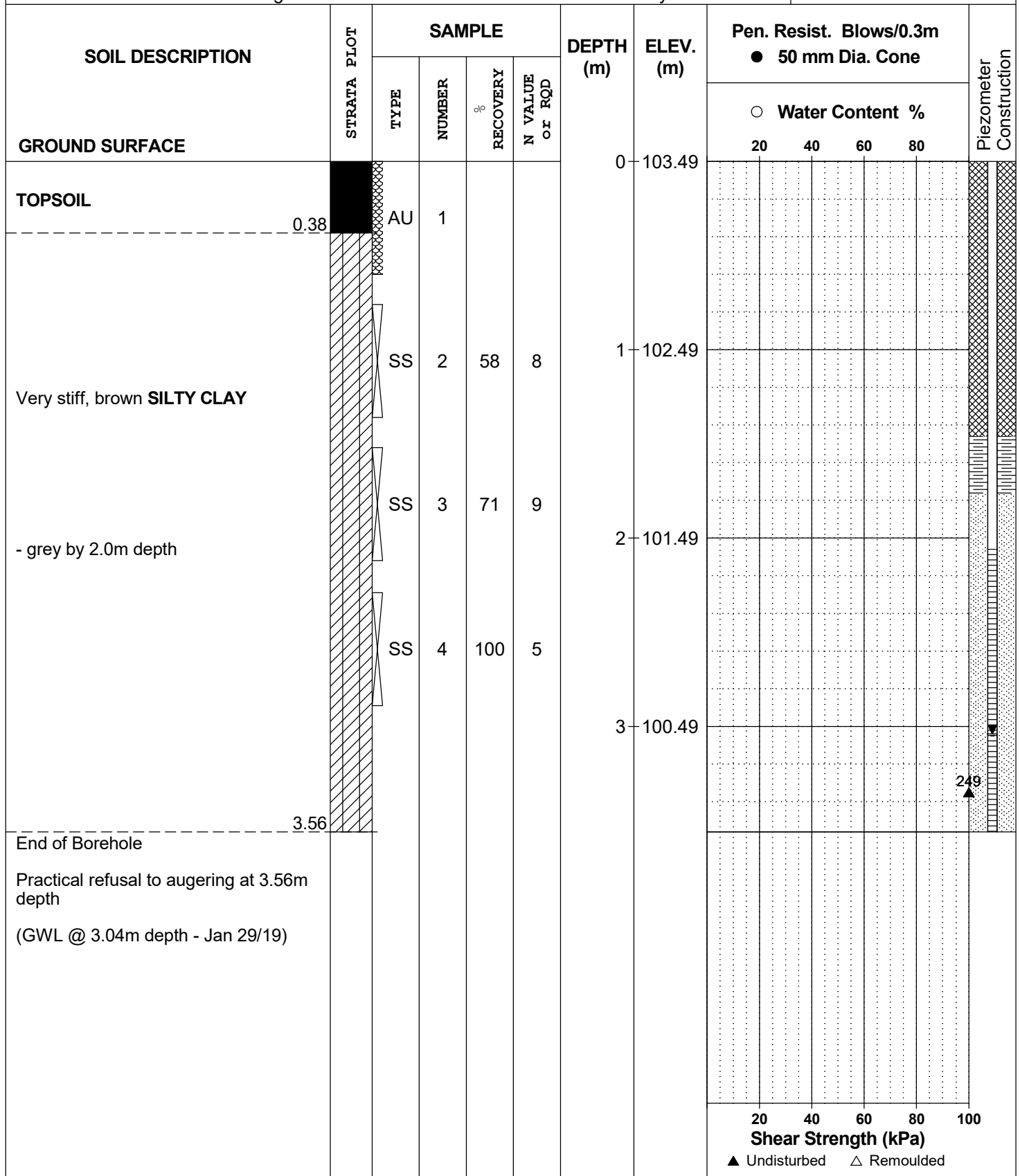
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REMARKS

HOLE NO. **BH 6**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

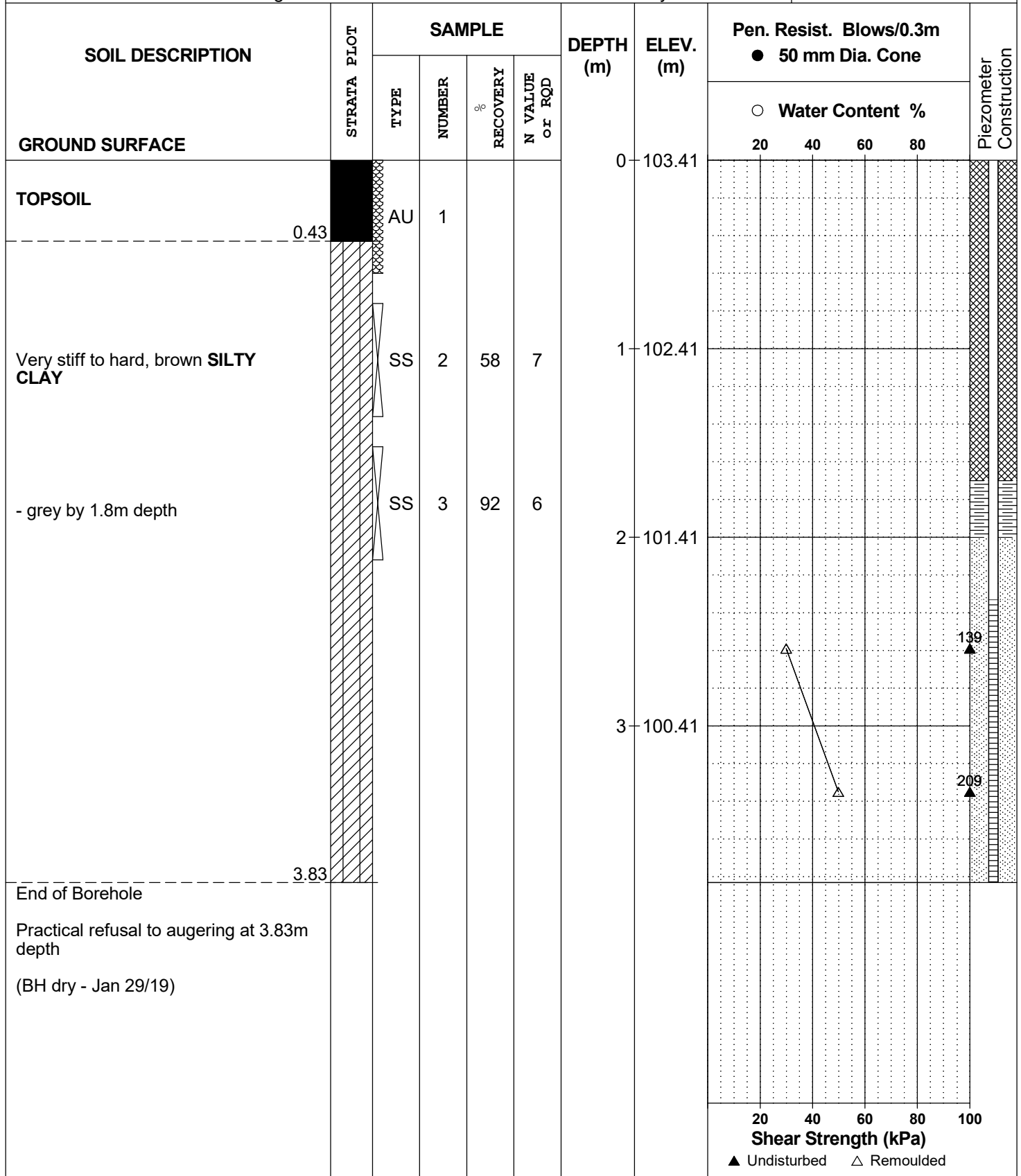
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REMARKS

HOLE NO. **BH 7**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

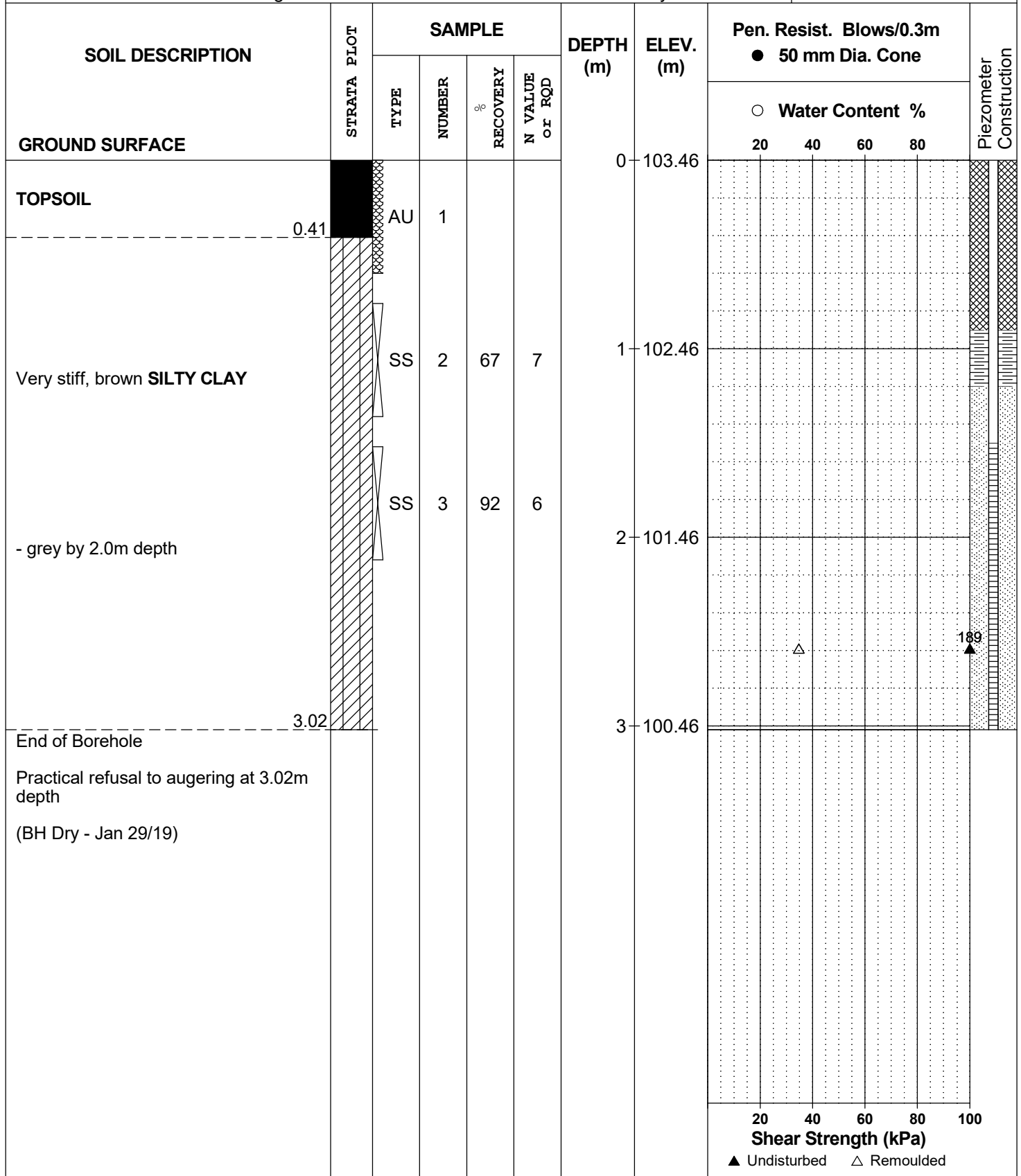
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REMARKS

HOLE NO. **BH 8**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

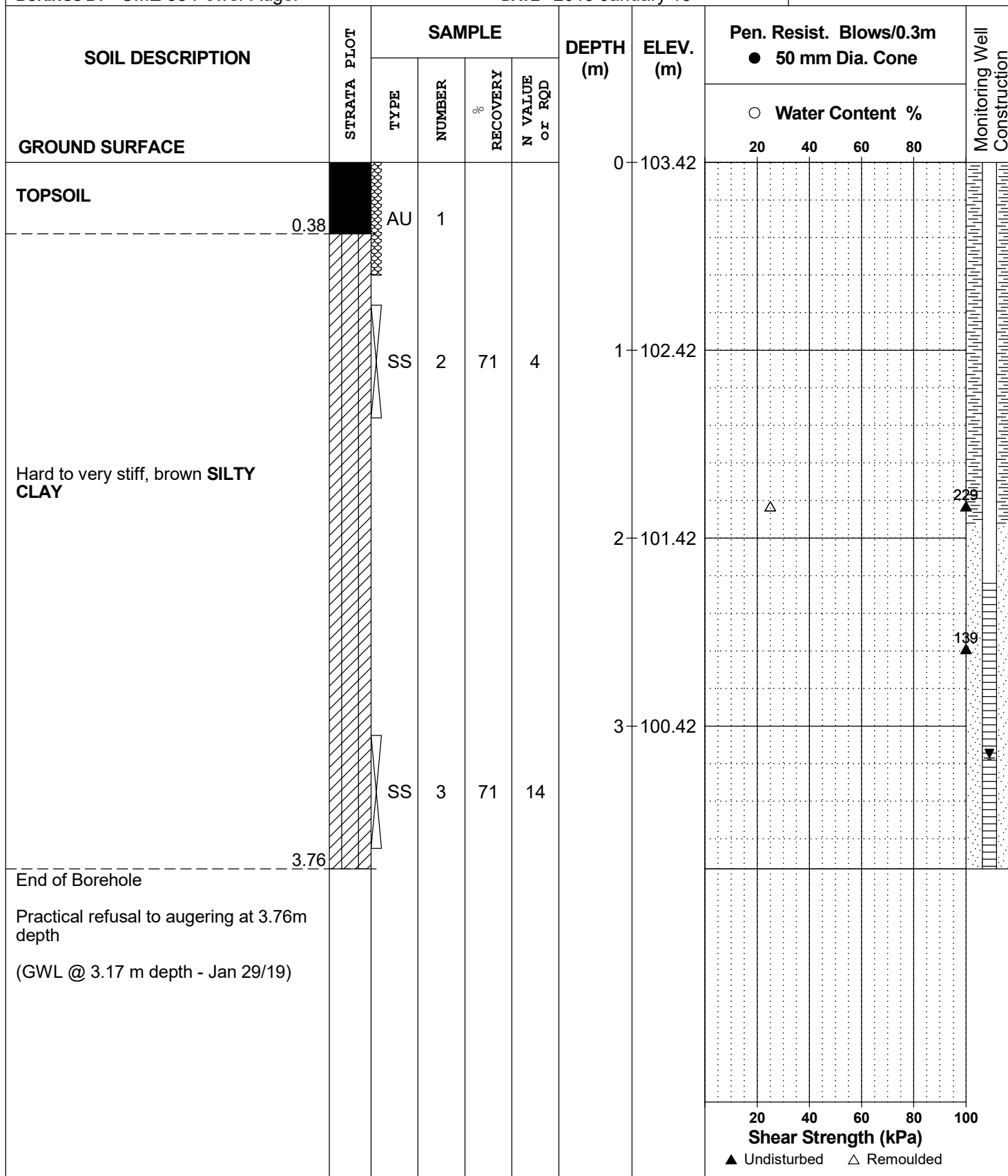
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH 9**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. PG4772

REMARKS

HOLE NO. BH10

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80	
GROUND SURFACE						0	103.31					
TOPSOIL		AU	1									
	0.41											
Very stiff, brown SILTY CLAY		SS	2	67	9	1	102.31					
		SS	3	75	6	2	101.31					
- grey by 2.1m depth												
	3.05											
GLACIAL TILL: Compact, brown sandy silt, trace clay and gravel, occasional cobbles and boulders		SS	4	83	19	3	100.31					169
	3.66											
End of Borehole												
Practical refusal to augering at 3.66m depth												
(GWL @ 2.18m depth - Jan 29/19)												

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed    △ Remoulded

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebek Ltd.

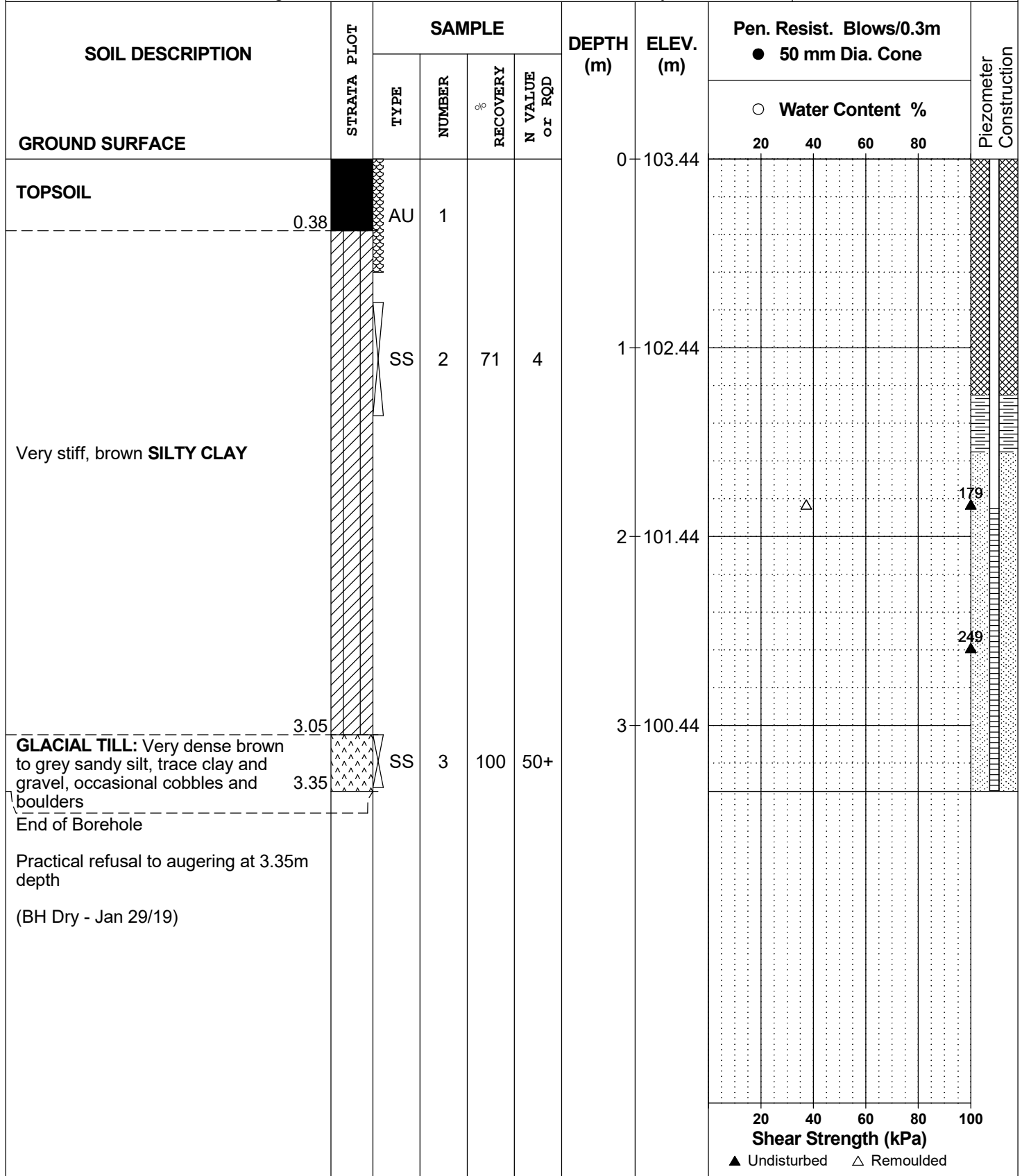
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REMARKS

HOLE NO. **BH11**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15





DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

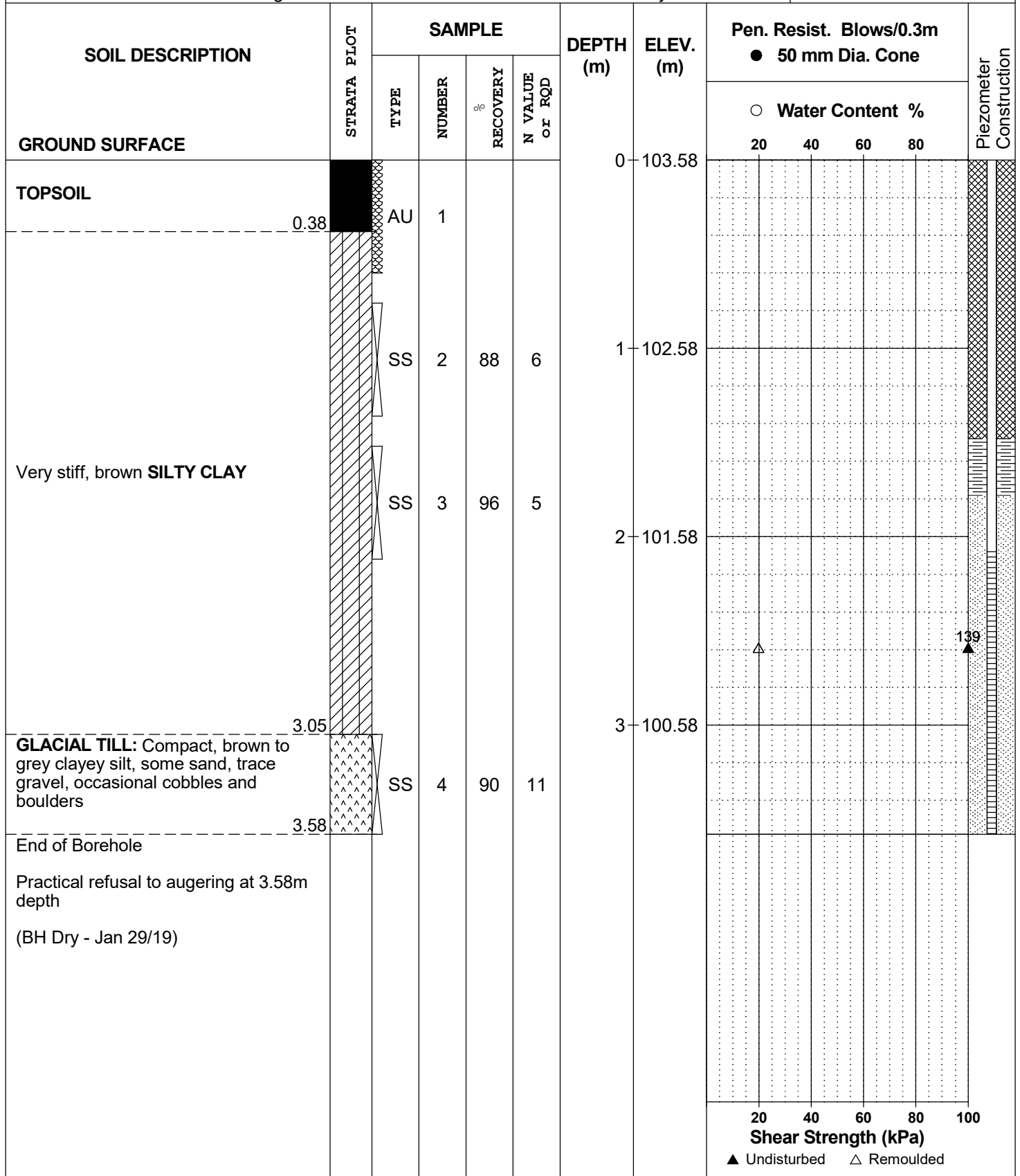
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REMARKS

HOLE NO. **BH12**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

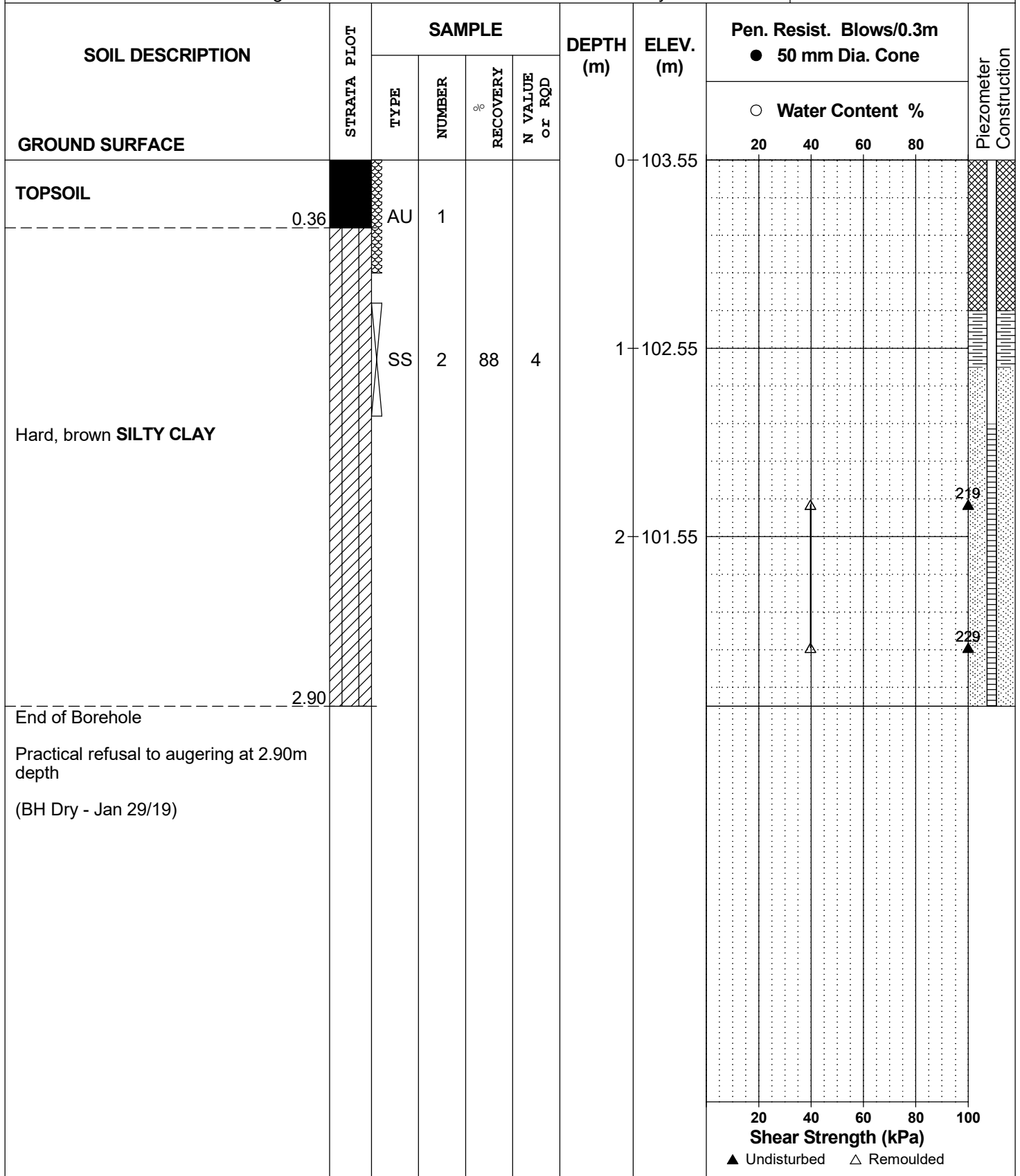
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REMARKS

HOLE NO. **BH13**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



## SOIL PROFILE AND TEST DATA

Geotechnical Investigation  
Proposed Mixed-Use Development - 20 Cedarow Ct.  
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

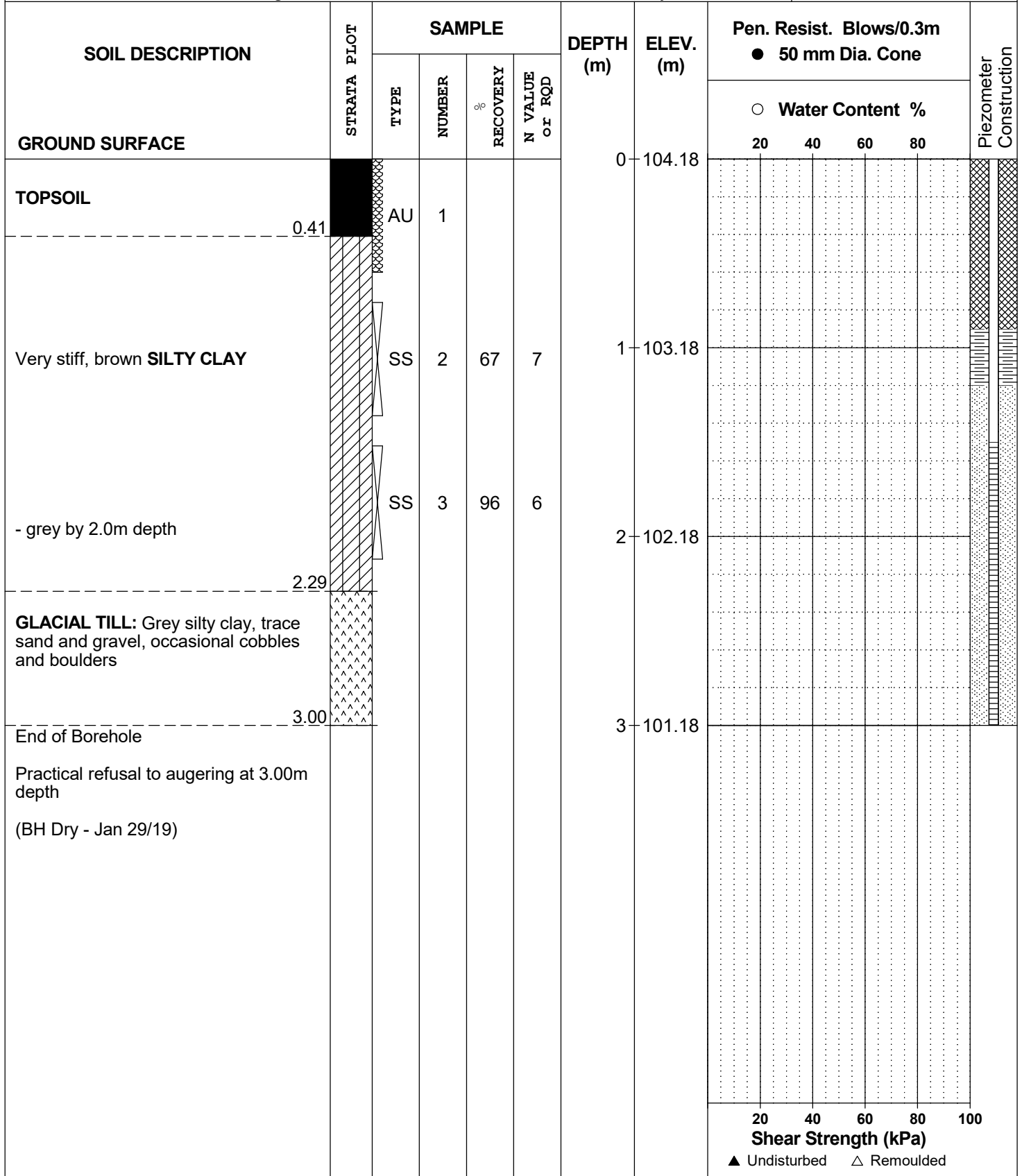
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REMARKS

HOLE NO. **BH14**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

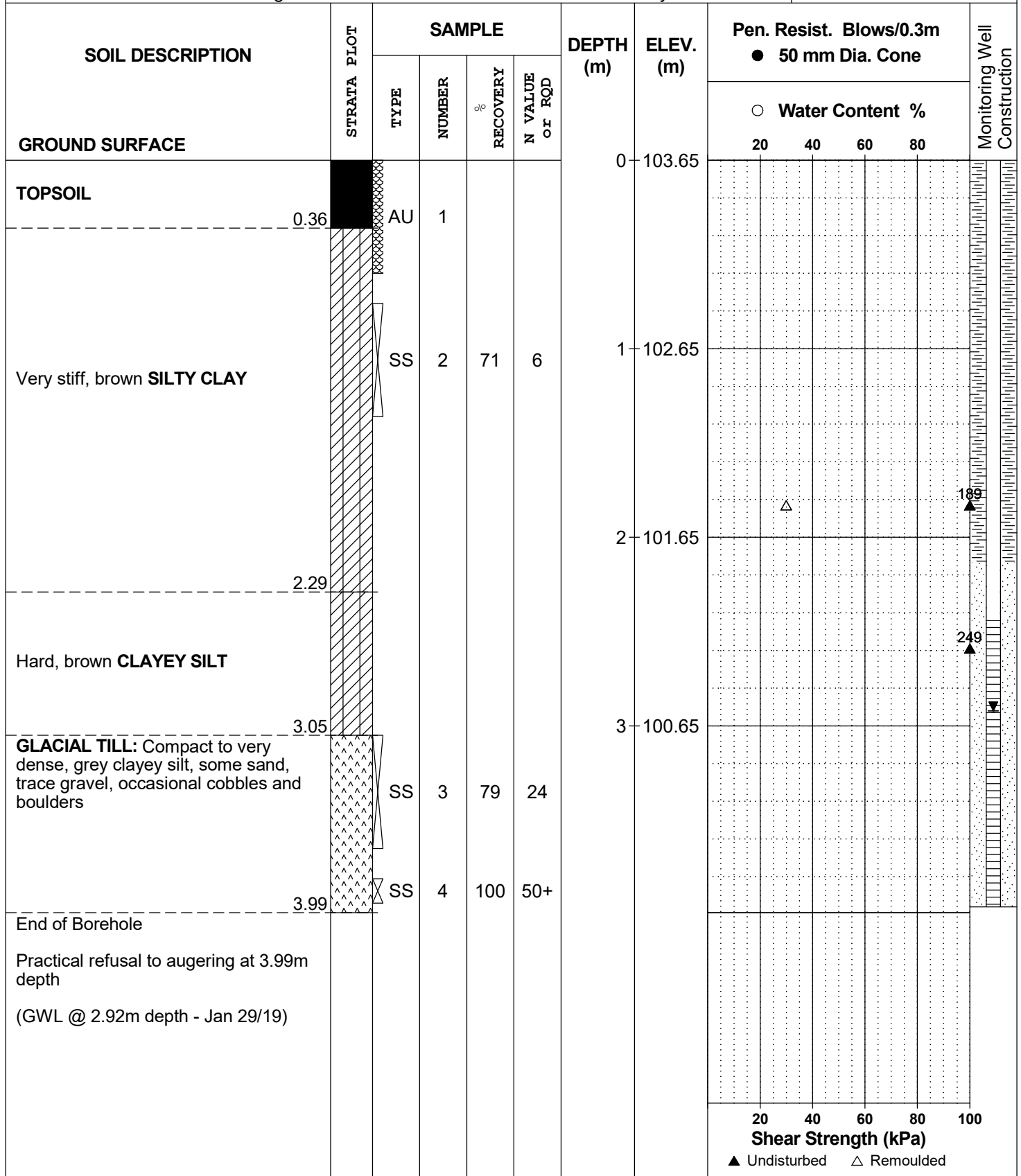
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REMARKS

HOLE NO. **BH15**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

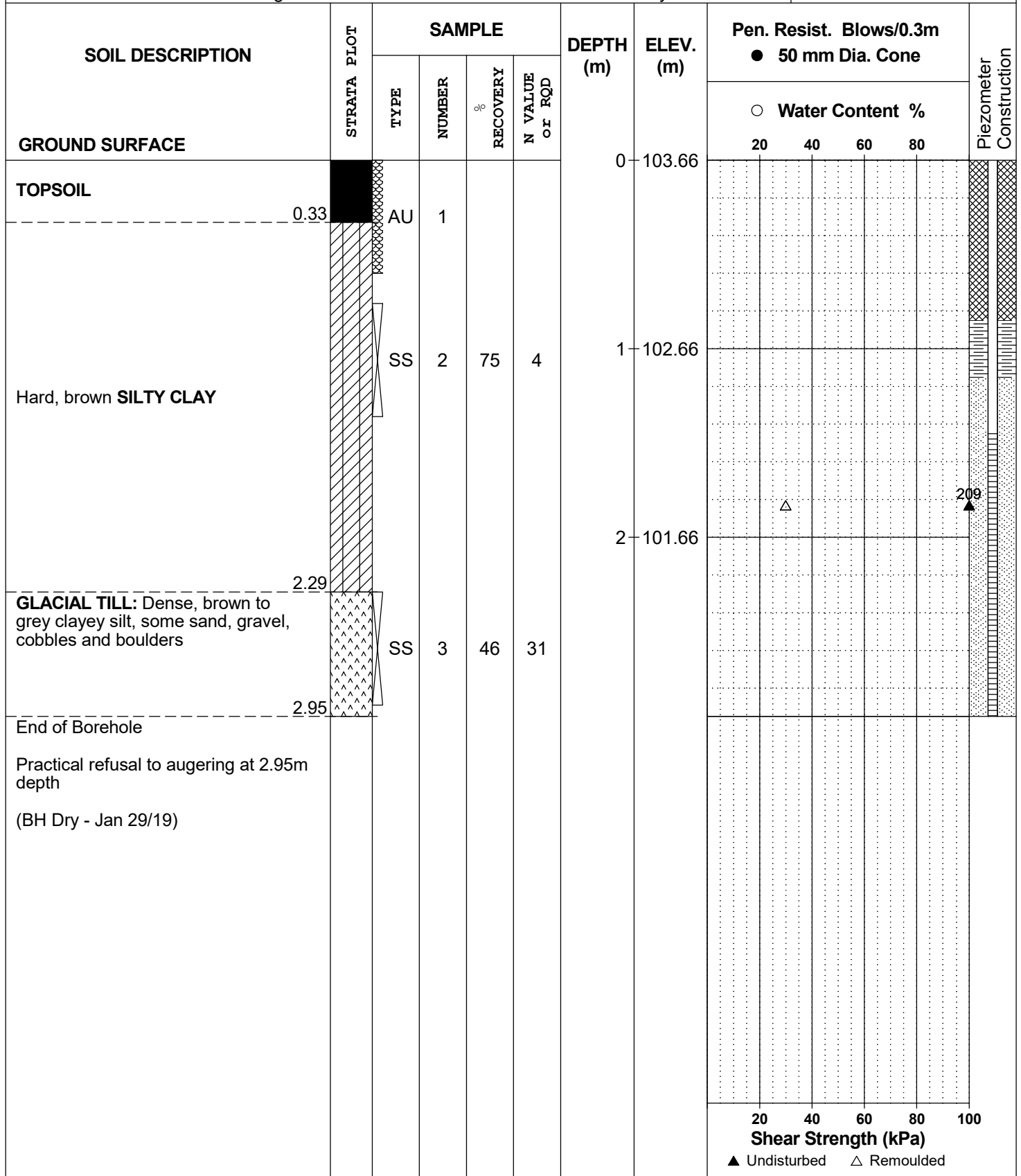
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REMARKS

HOLE NO. **BH16**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4772**

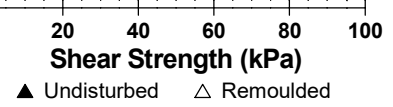
REMARKS

HOLE NO. **BH17**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %			N VALUE or RQD	20	40	60		80
GROUND SURFACE					0	104.19						
TOPSOIL	0.38	AU	1									
Very stiff to hard, brown <b>CLAYEY SILT</b>		SS	2	79	7	1	103.19					
- grey by 1.8m depth		SS	3	100	55	2	102.19					
End of Borehole	2.23											
Practical refusal to augering at 2.23m depth (BH Dry - Jan 29/19)												



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH18**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	104.15						
TOPSOIL	0.33	AU	1										
Hard, brown <b>CLAYEY SILT</b>	1.96	SS	2	88	11	1	103.15						
- grey by 1.8m depth		SS	3	88	50+								
End of Borehole Practical refusal to augering at 1.96m depth (BH Dry - Jan 29/19)													

○ Water Content %

▲ Undisturbed    △ Remoulded

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

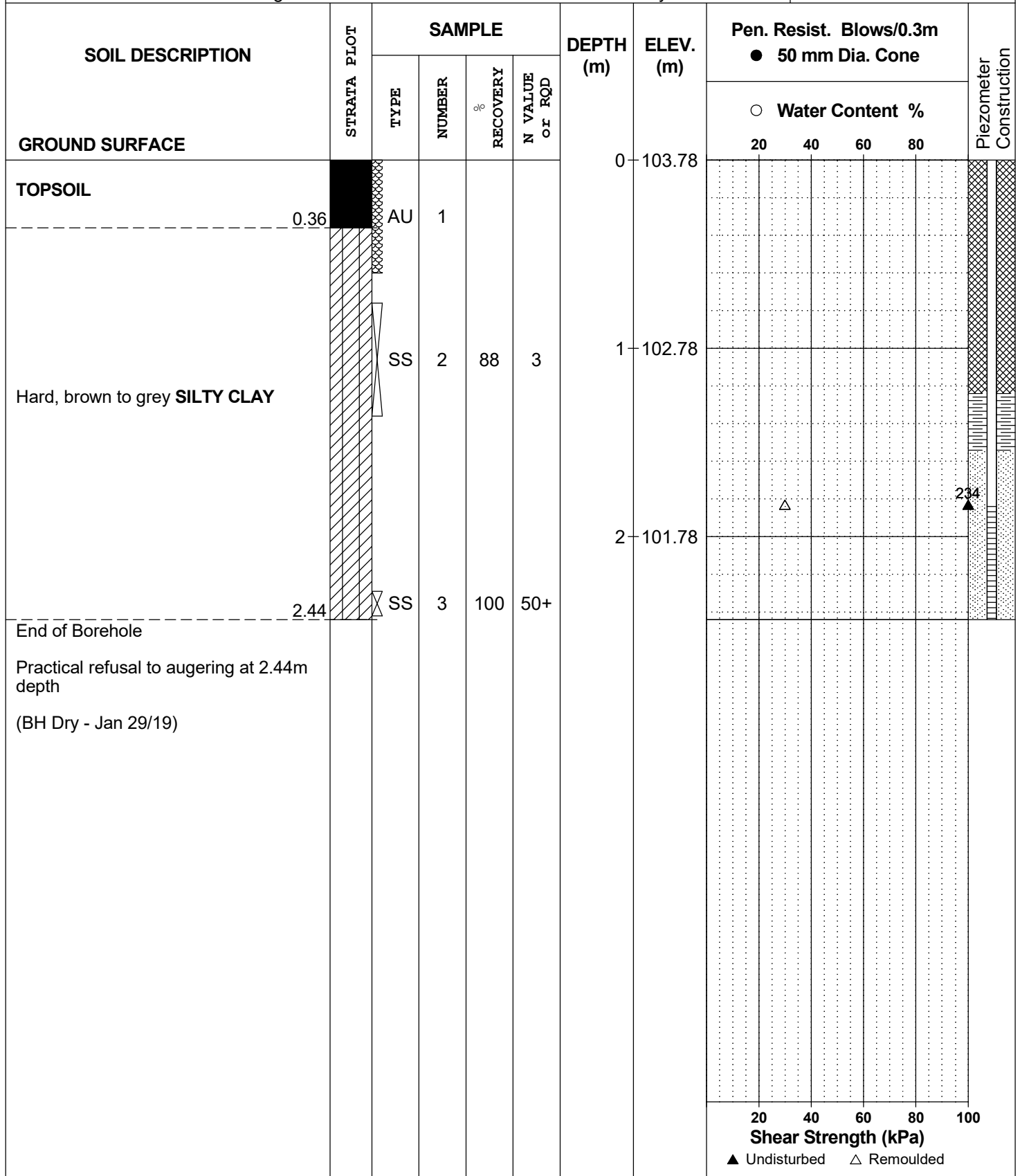
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH19**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16





**DATUM** Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

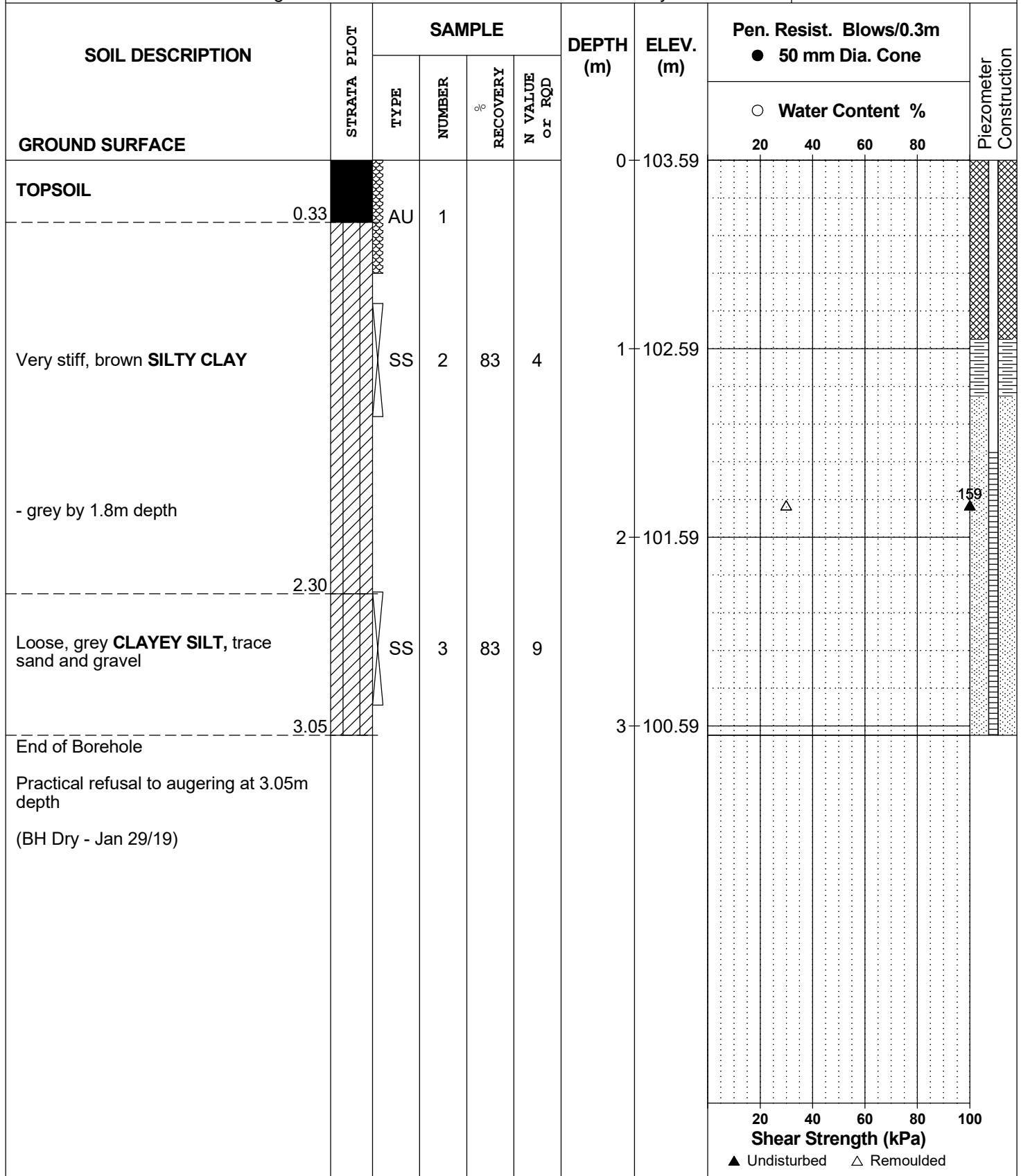
**FILE NO.**  
**PG4772**

**REMARKS**

**HOLE NO.**  
**BH20**

**BORINGS BY** CME 55 Power Auger

**DATE** 2019 January 16



## SOIL PROFILE AND TEST DATA

Geotechnical Investigation  
Proposed Mixed-Use Development - 20 Cedarow Ct.  
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebek Ltd.

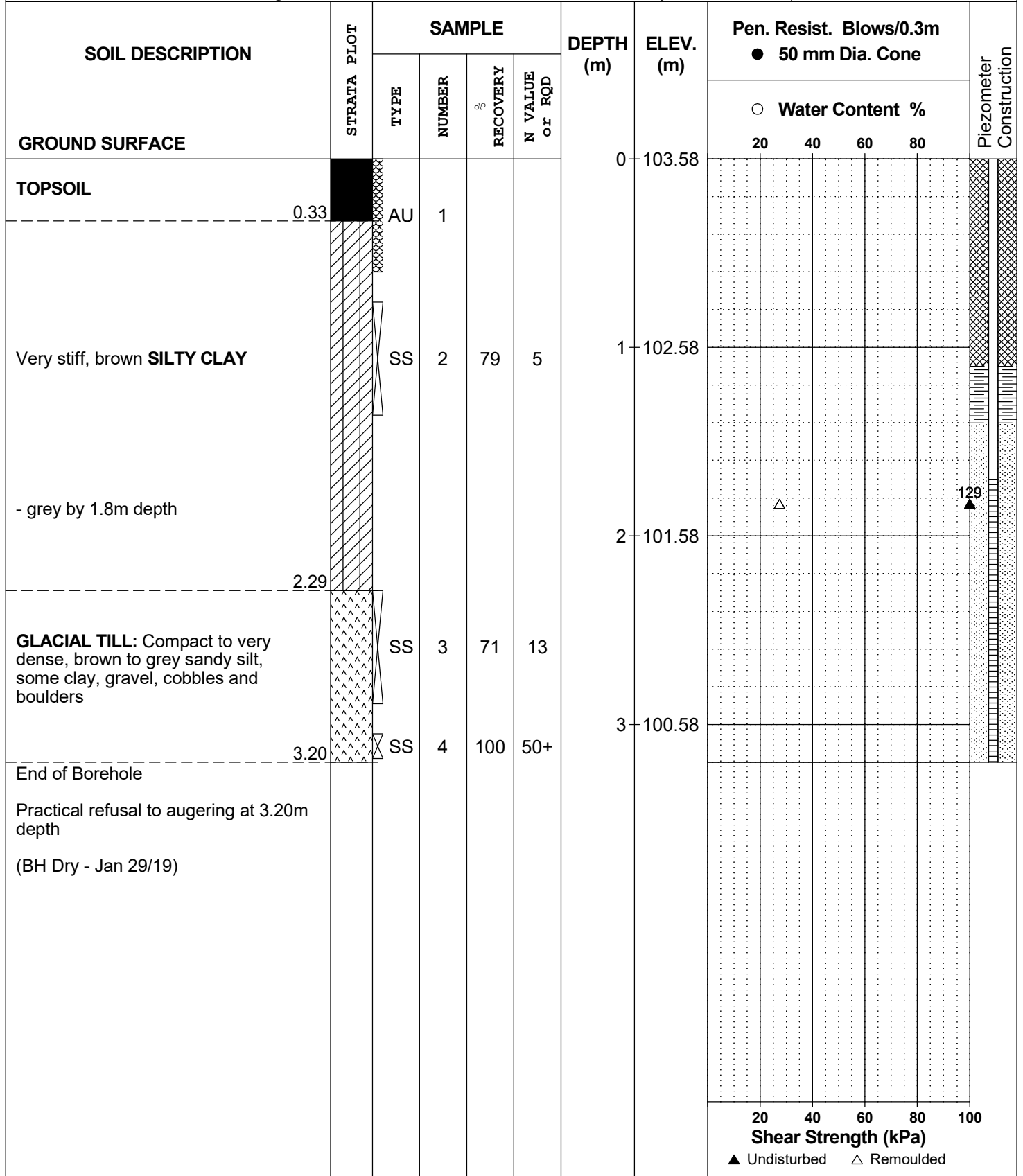
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH21**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

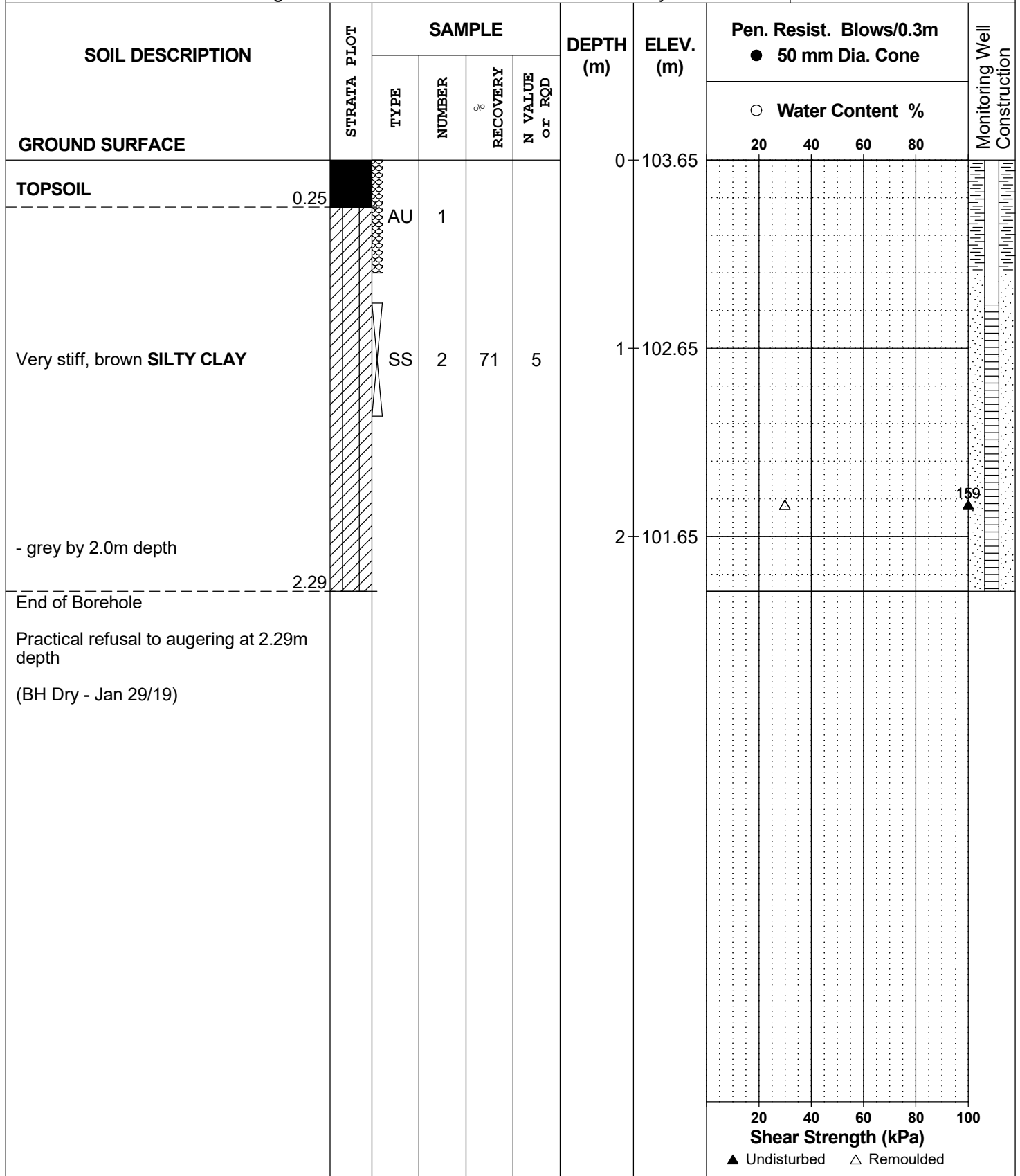
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH22**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

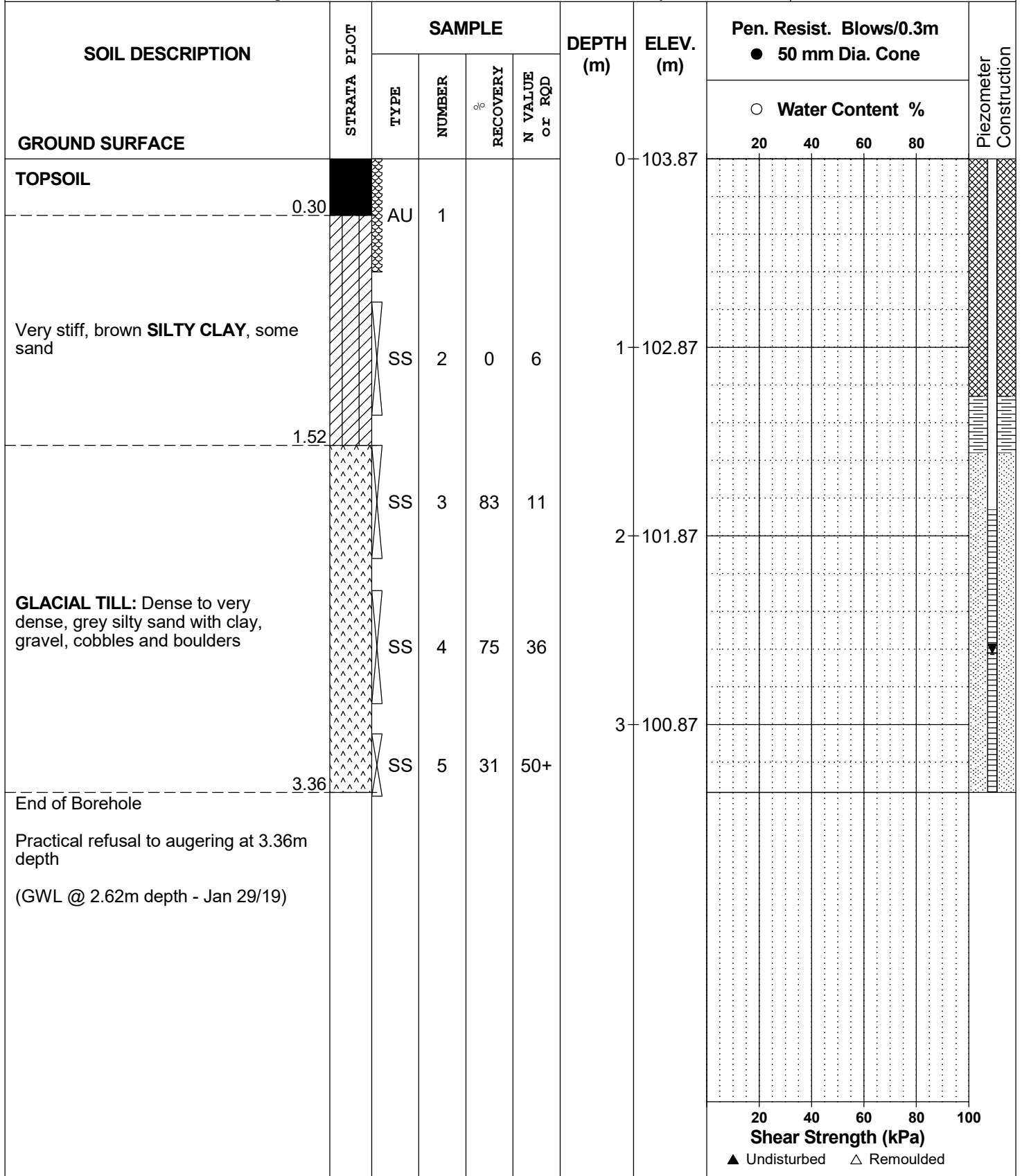
REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

FILE NO. **PG4772**

HOLE NO. **BH23**



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebek Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH24**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %			N VALUE or RQD	○ Water Content %				
								20	40	60		80
GROUND SURFACE					0	104.04						
TOPSOIL	0.36	AU	1									
Very stiff, brown to grey <b>CLAYEY SILT</b>	0.36 - 2.29	SS	2	67	10	1	103.04					
		SS	3	79	29	2	102.04					
<b>GLACIAL TILL:</b> Compact to very dense, brown clayey silt, some sand, gravel, cobbles and boulders	2.29 - 3.15	SS	4	58	23							
		SS	5	100	50+	3	101.04					
End of Borehole Practical refusal to augering at 3.15m depth (GWL @ 2.55m depth - Jan 29/19)	3.15											

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH25**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %			N VALUE or RQD	20	40	60		80
GROUND SURFACE					0	104.07						
TOPSOIL		AU	1									
Very stiff, brown <b>CLAYEY SILT</b>		SS	2	75	11	1	103.07					
<b>GLACIAL TILL:</b> Very dense, grey clayey silt with sand, gravel, cobbles, boulders		SS	3	75	50+							
End of Borehole												
Practical refusal to augering at 1.62m depth (GWL @ 1.68m depth - Jan 29/19)												

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

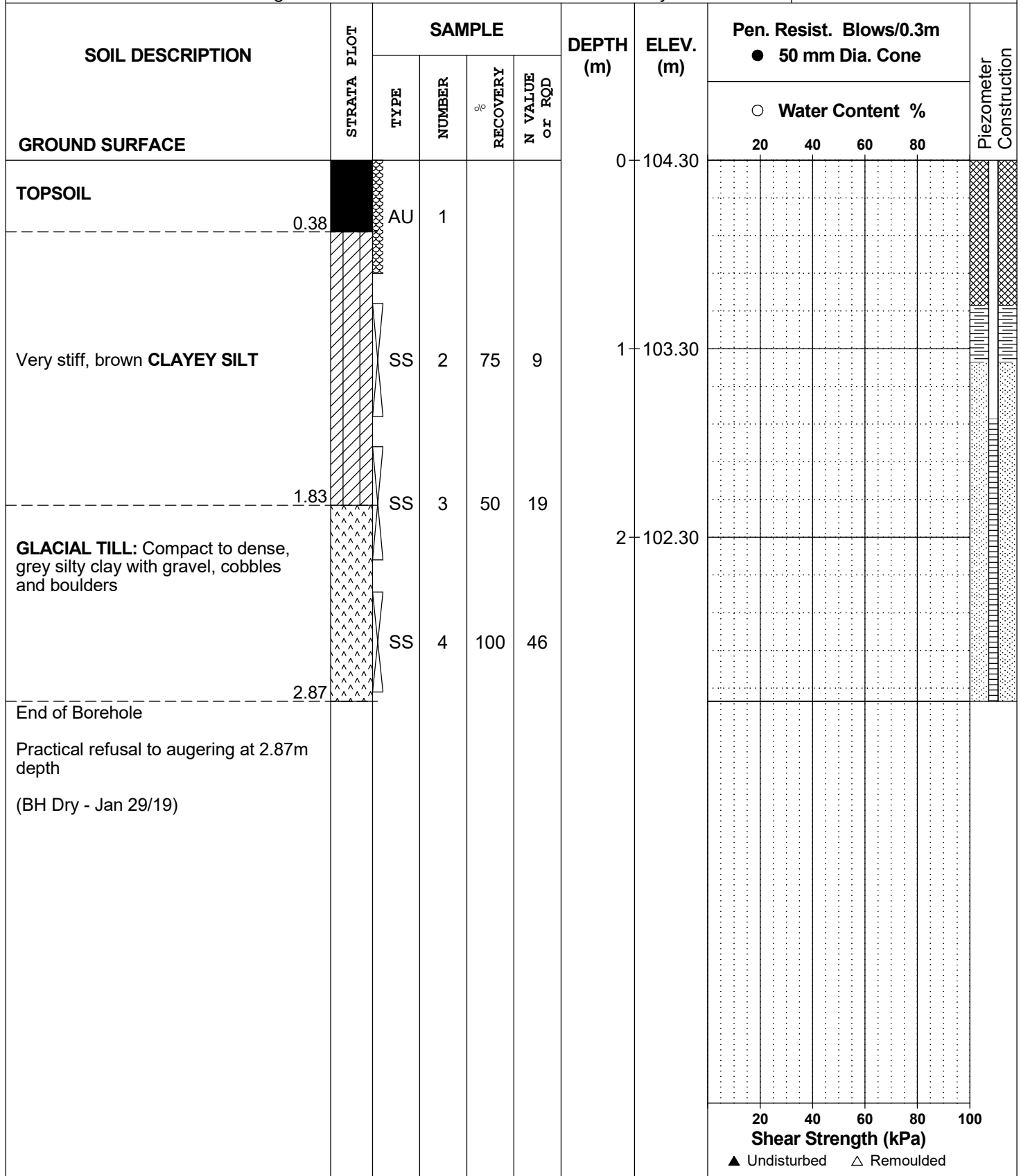
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH26**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17



## SOIL PROFILE AND TEST DATA

Geotechnical Investigation  
Proposed Mixed-Use Development - 20 Cedarow Ct.  
Ottawa, Ontario

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH27**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	103.97						
TOPSOIL	0.33	AU	1										
Very stiff, brown <b>CLAYEY SILT</b>		SS	2	71	8	1	102.97						
- grey by 1.7m depth		SS	3	88	50+								
End of Borehole	1.93												
Practical refusal to augering at 1.93m depth (BH Dry - Jan 29/19)													

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



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

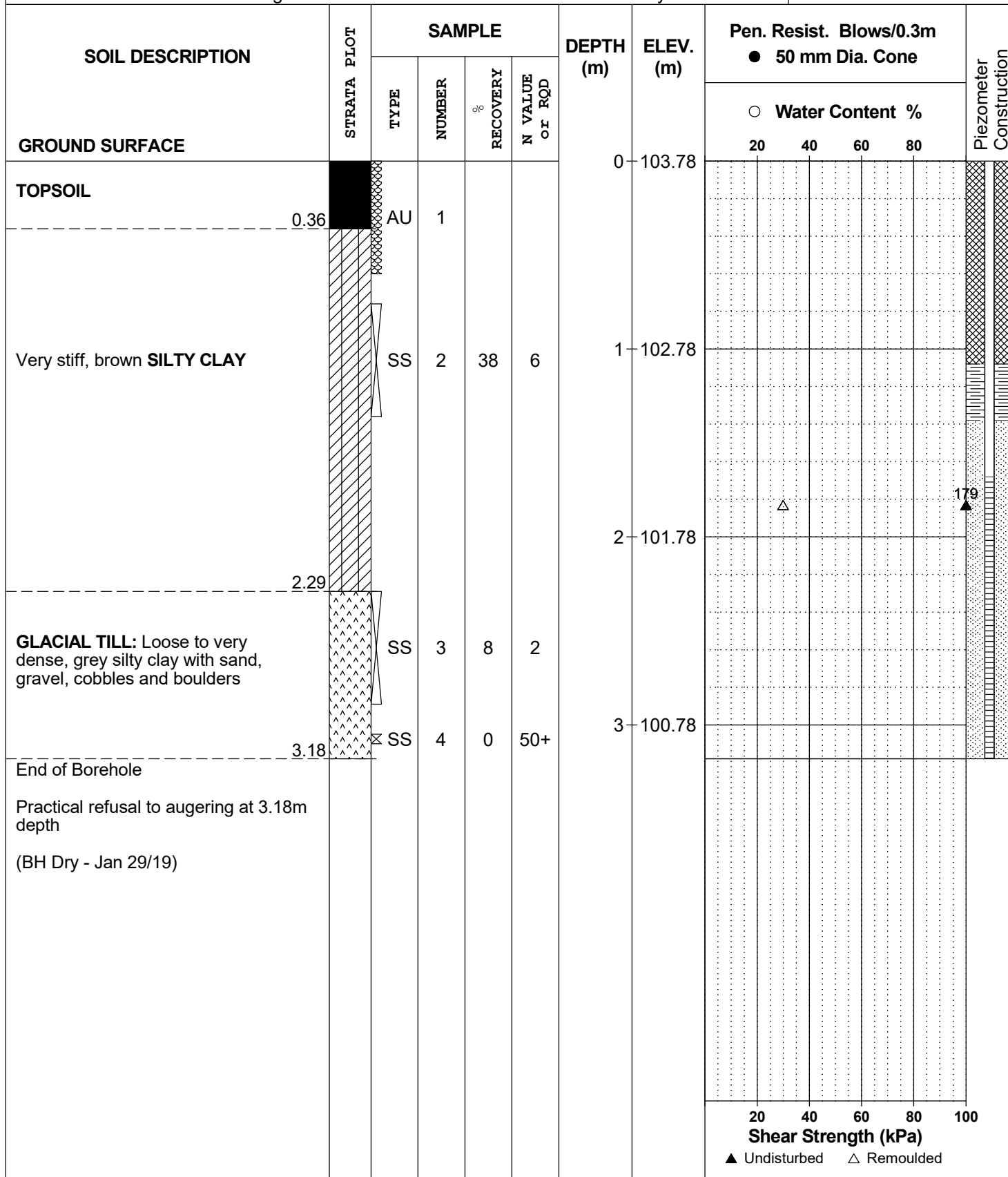
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH28**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH29**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	103.71						
TOPSOIL	0.38	AU	1										
Very stiff, brown <b>SILTY CLAY</b>	0.38	SS	2	50	7	1	102.71						
		SS	3	71	4	2	101.71						
<b>GLACIAL TILL:</b> Loose, grey silty clay with sand, gravel, cobbles and boulders	2.29	SS	4	17	7								
End of Borehole	2.95												
Practical refusal to augering at 2.95m depth (BH Dry - Jan 29/19)													
								20	40	60	80	100	
								<b>Shear Strength (kPa)</b>					
								▲ Undisturbed    △ Remoulded					

# SYMBOLS AND TERMS

## SOIL DESCRIPTION

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

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

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

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

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

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

## SYMBOLS AND TERMS (continued)

### SOIL DESCRIPTION (continued)

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

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

### ROCK DESCRIPTION

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

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

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

### SAMPLE TYPES

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

## SYMBOLS AND TERMS (continued)

### GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic limit, % (water content above which soil behaves plastically)
PI	-	Plasticity index, % (difference between LL and PL)
Dxx	-	Grain size which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D10	-	Grain size at which 10% of the soil is finer (effective grain size)
D60	-	Grain size at which 60% of the soil is finer
Cc	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
Cu	-	Uniformity coefficient = $D_{60} / D_{10}$

Cc and Cu are used to assess the grading of sands and gravels:

Well-graded gravels have:  $1 < Cc < 3$  and  $Cu > 4$

Well-graded sands have:  $1 < Cc < 3$  and  $Cu > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

Cc and Cu are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

### CONSOLIDATION TEST

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

### PERMEABILITY TEST

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

### STRATA PLOT



Topsoil



Asphalt



Fill



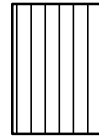
Peat



Sand



Silty Sand



Silt



Sandy Silt



Clay



Silty Clay



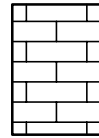
Clayey Silty Sand



Glacial Till



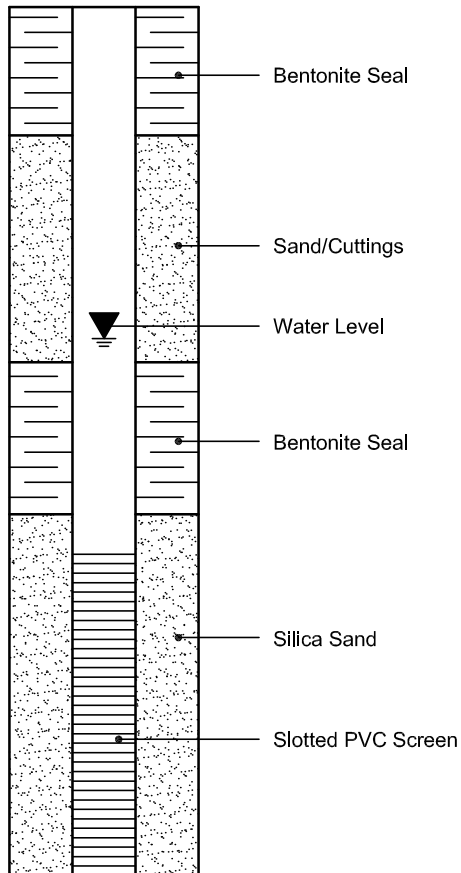
Shale



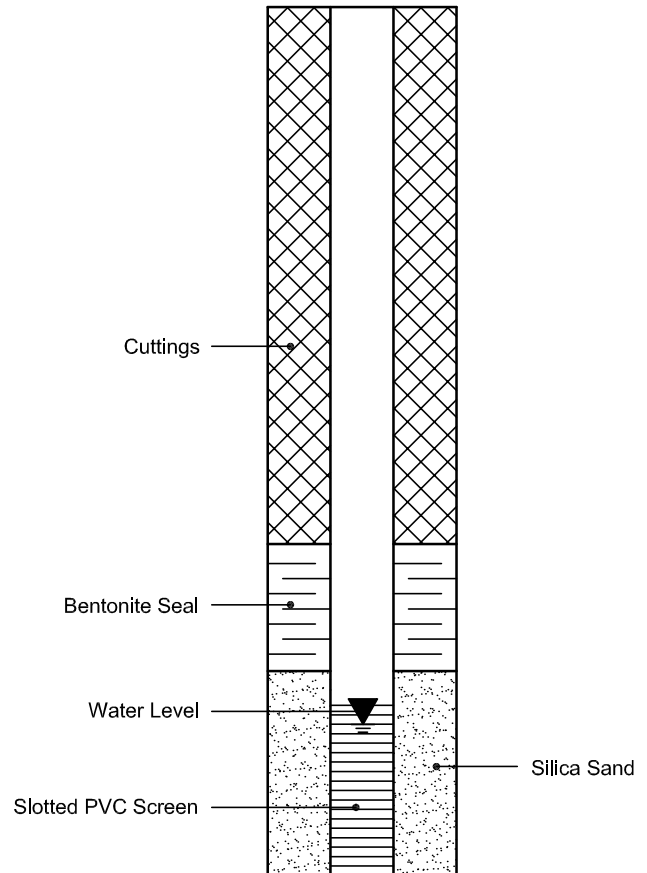
Bedrock

### MONITORING WELL AND PIEZOMETER CONSTRUCTION

#### MONITORING WELL CONSTRUCTION



#### PIEZOMETER CONSTRUCTION



Certificate of Analysis  
 Client: Paterson Group Consulting Engineers  
 Client PO: 25648

Report Date: 22-Jan-2019

Order Date: 16-Jan-2019

**Project Description: PG4772**

<b>Client ID:</b>	BH#16-19 SS#3	-	-	-
<b>Sample Date:</b>	01/15/2019 09:00	-	-	-
<b>Sample ID:</b>	1903309-01	-	-	-
<b>MDL/Units</b>	Soil	-	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	85.8	-	-	-
----------	--------------	------	---	---	---

**General Inorganics**

pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	76.2	-	-	-

**Anions**

Chloride	5 ug/g dry	6	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

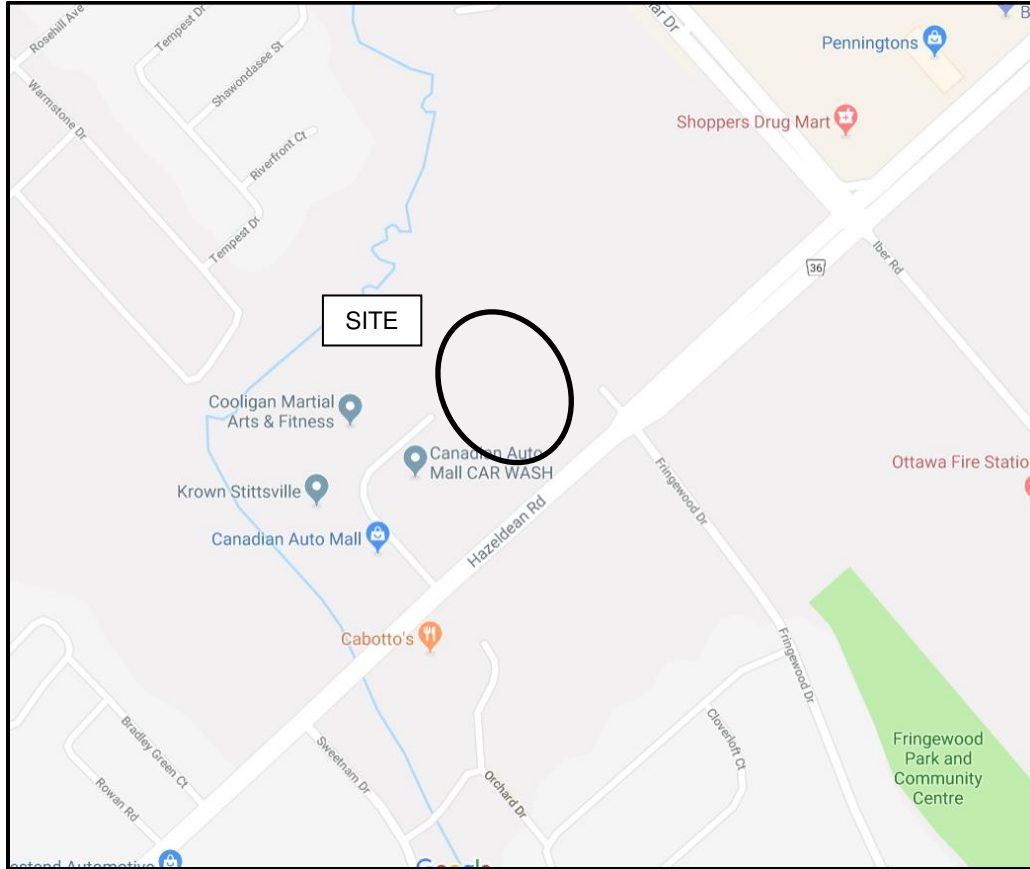
# **APPENDIX 2**

**FIGURE 1 - KEY PLAN**

**FIGURES 2 TO 4 - SLOPE STABILITY ANALYSIS SECTIONS**

**DRAWING PG4772-1 - TEST HOLE LOCATION PLAN**

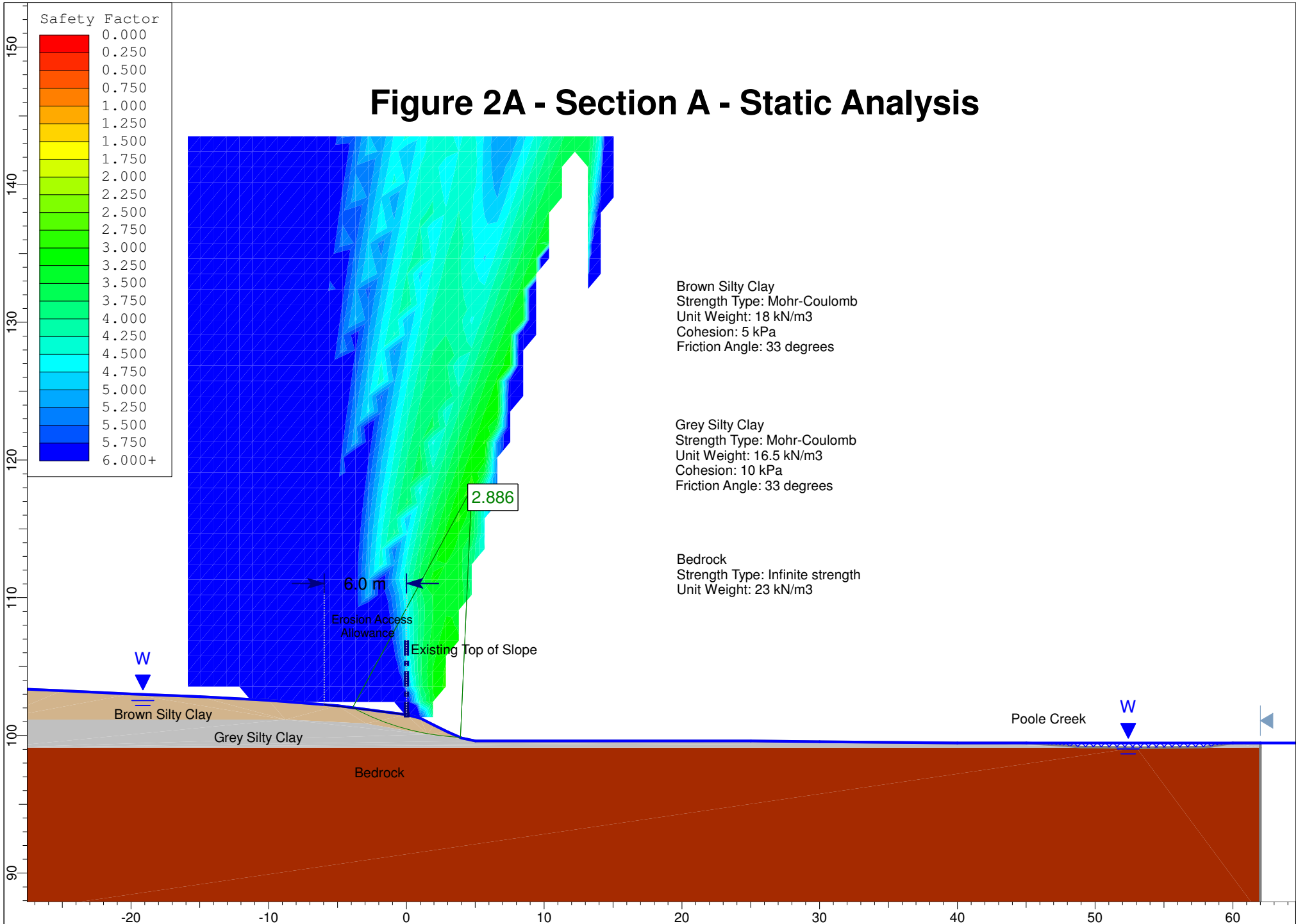


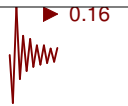


# FIGURE 1

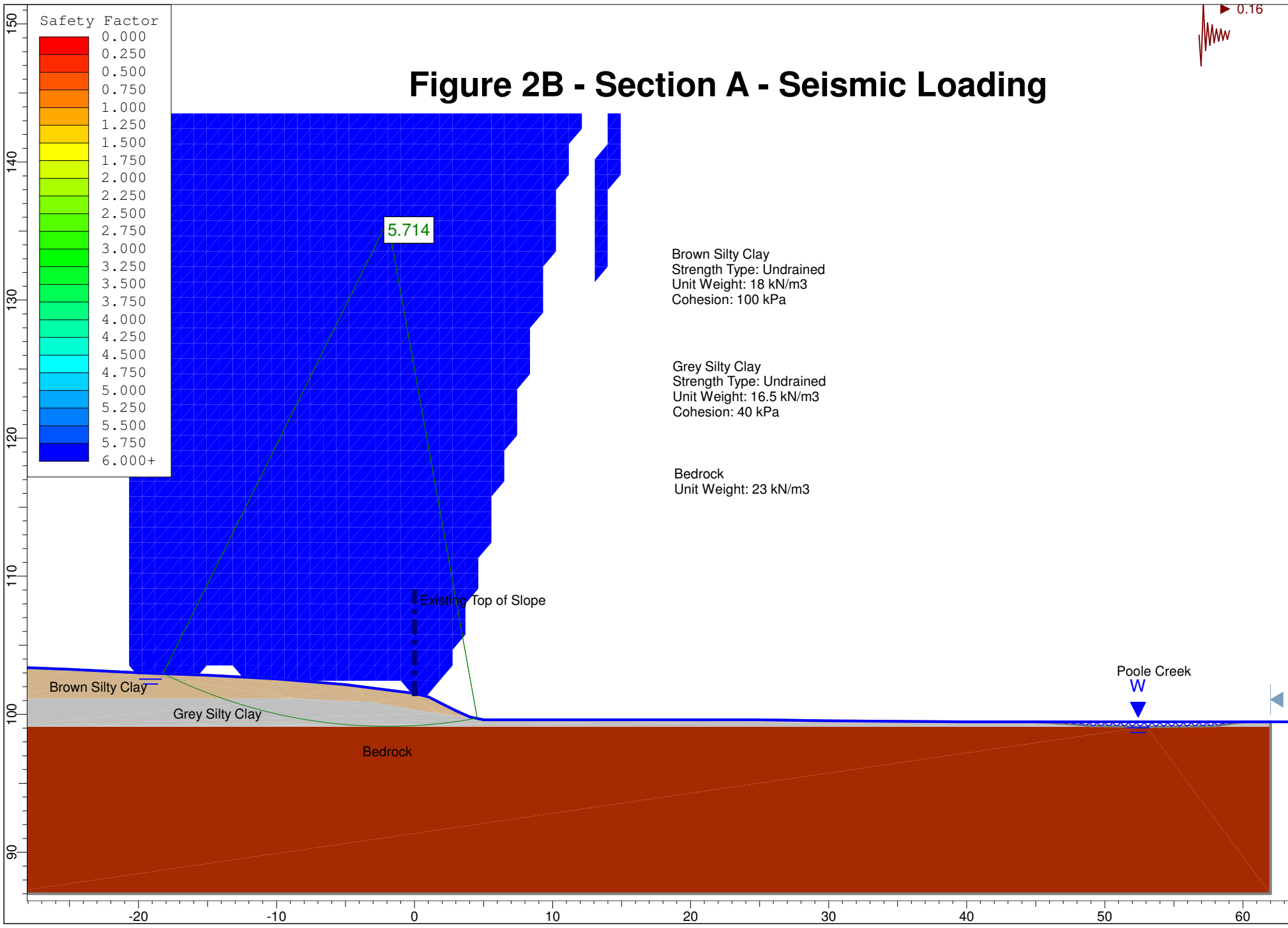
## KEY PLAN

# Figure 2A - Section A - Static Analysis

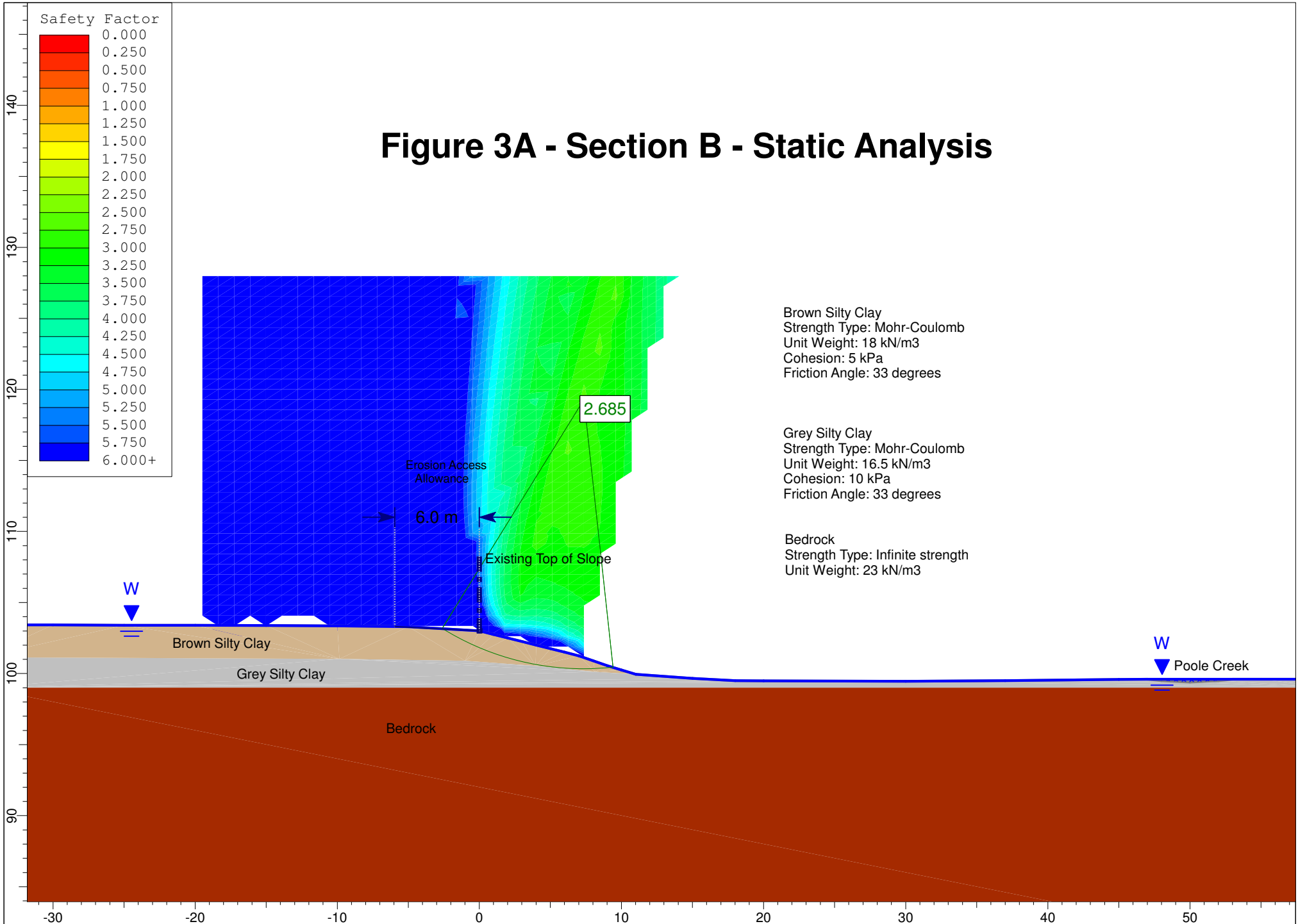




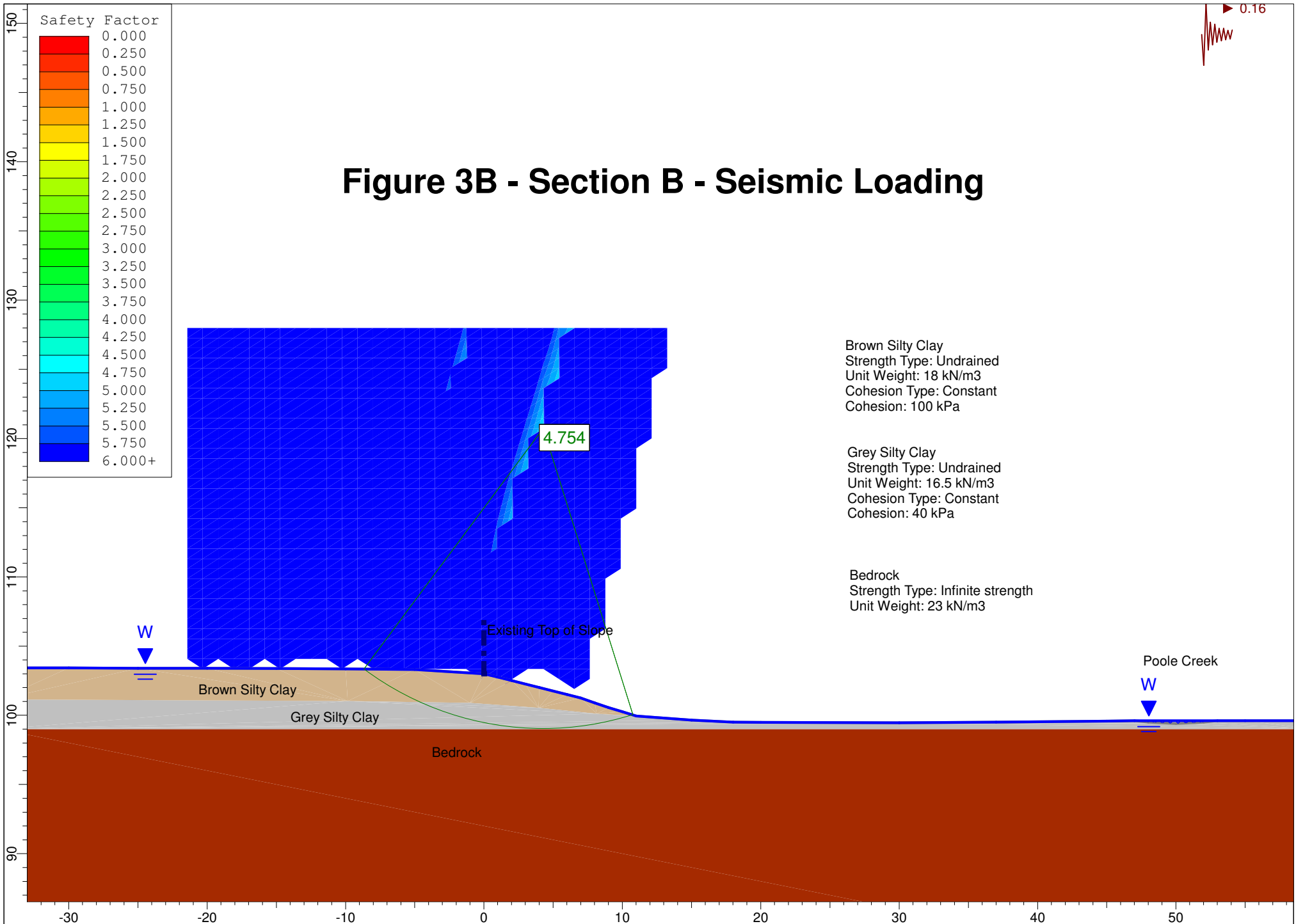
# Figure 2B - Section A - Seismic Loading



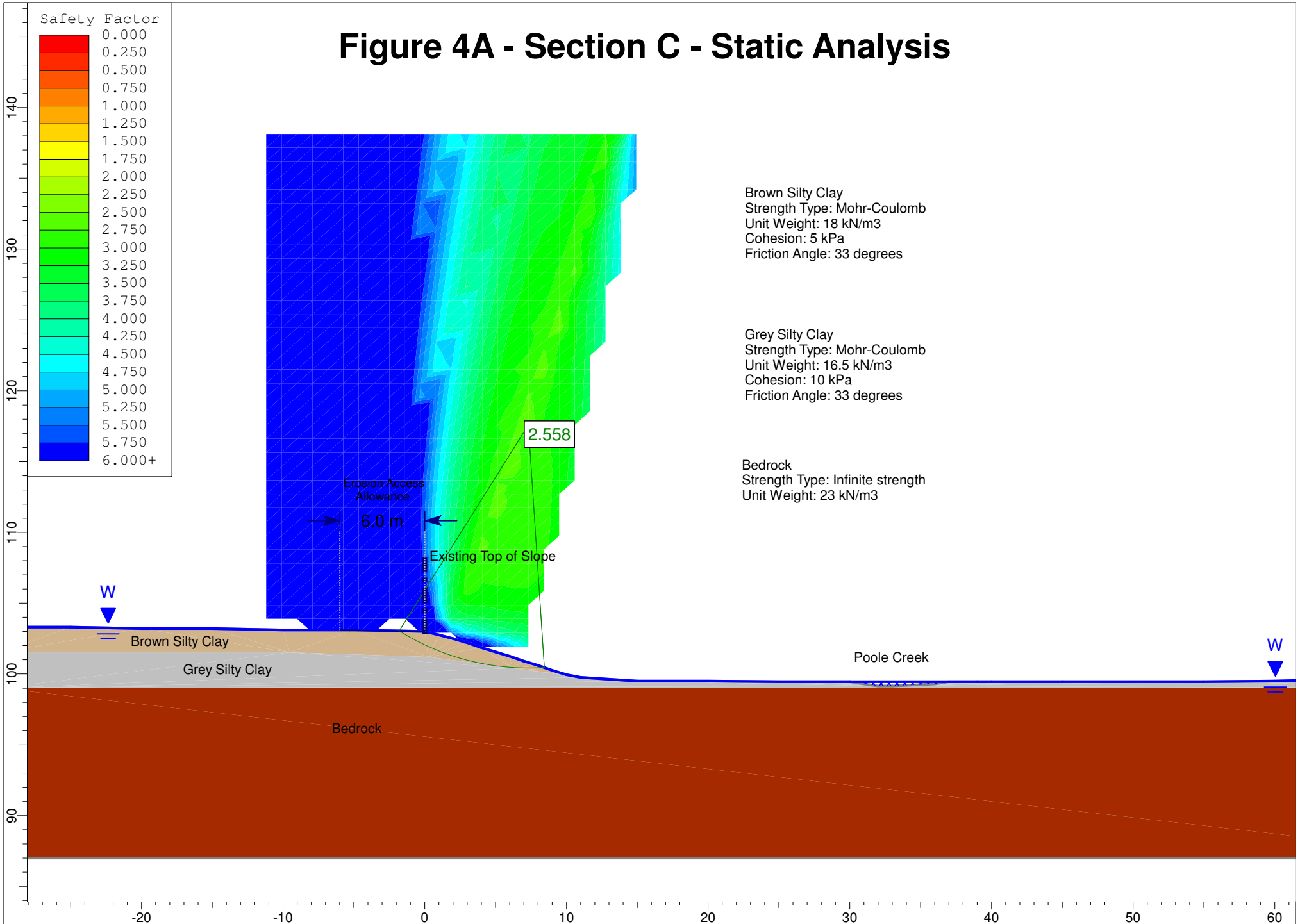
# Figure 3A - Section B - Static Analysis



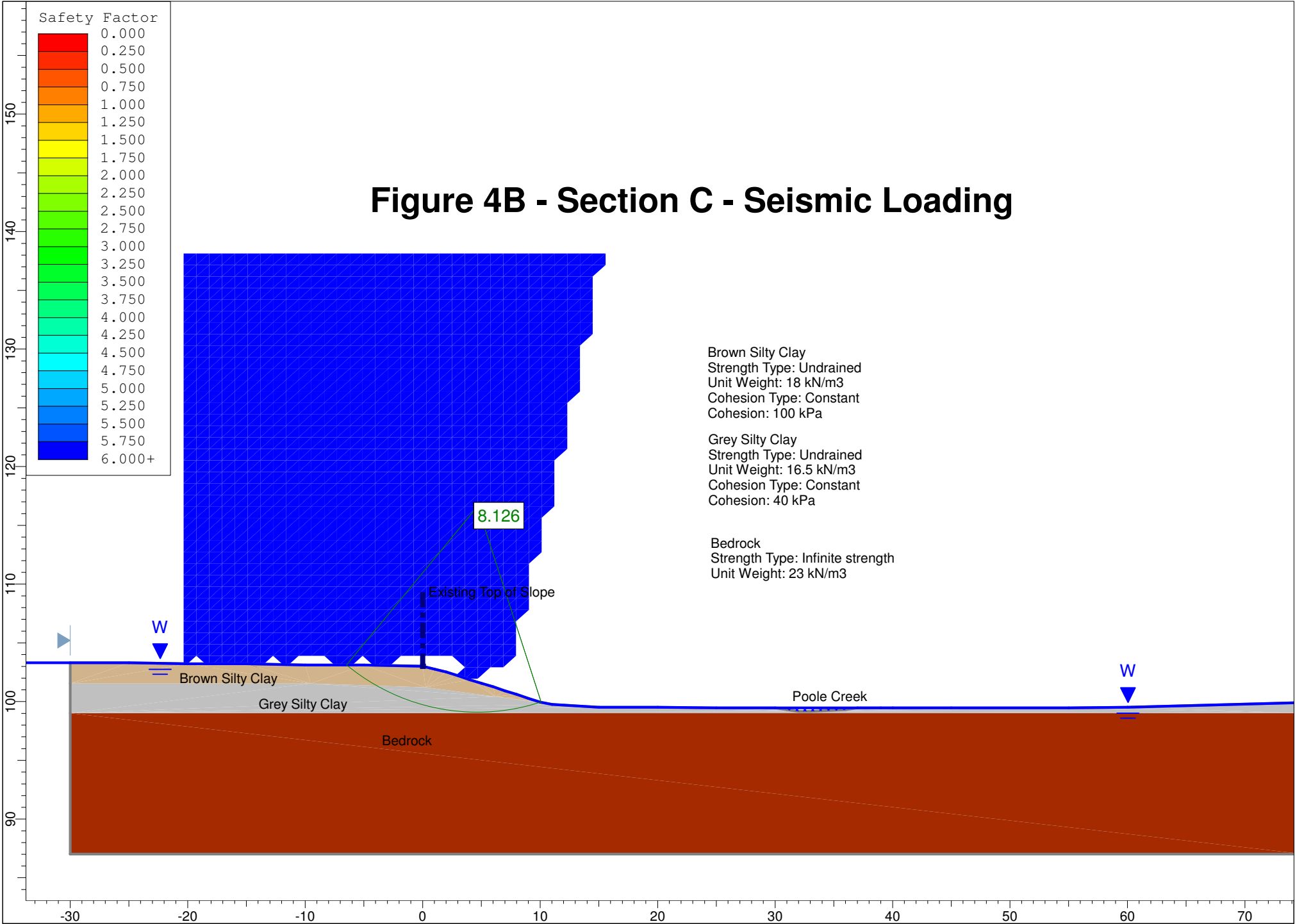
# Figure 3B - Section B - Seismic Loading

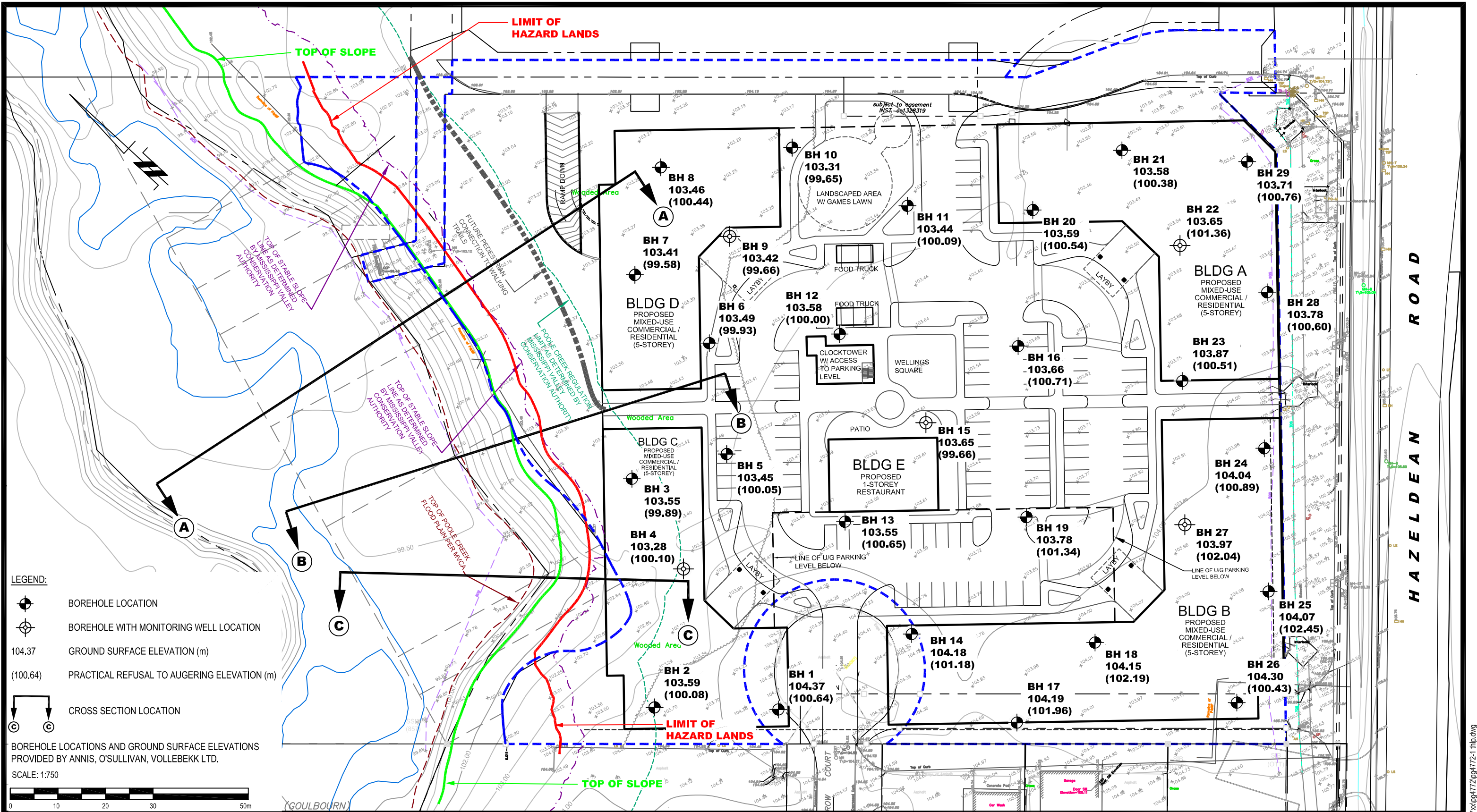


# Figure 4A - Section C - Static Analysis



# Figure 4B - Section C - Seismic Loading





**LEGEND:**

- BOREHOLE LOCATION
- BOREHOLE WITH MONITORING WELL LOCATION
- 104.37 GROUND SURFACE ELEVATION (m)
- (100.64) PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
- CROSS SECTION LOCATION

BOREHOLE LOCATIONS AND GROUND SURFACE ELEVATIONS PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.  
SCALE: 1:750

**patersongroup**  
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL
1	LIMIT OF HAZARD LANDS REVISED	28/02/2019	FA

**NAUTICAL LANDS GROUP**  
**GEOTECHNICAL INVESTIGATION**  
PROP. MIXED-USE DEVELOPMENT - WELLINGS OF STITTSVILLE PHASE 2 - 20 CEDAROW CT.  
OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale:	1:750	Date:	01/2019
Drawn by:	MPG	Report No.:	PG4772-1
Checked by:	FA	Dwg. No.:	<b>PG4772-1</b>
Approved by:	DJG	Revision No.:	1

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re: **Grading Plan Review**  
**Proposed Mixed-Use Development – Wellings of Stittsville Phase 2**  
**20 Cedarow Court - Ottawa**

to: Nautical Lands Group – **Mr. Mark Williams** – [mwilliams@nauticallandsgroup.com](mailto:mwilliams@nauticallandsgroup.com)

cc: Stantec – **Mr. Mike Sharp** – [Mike.Sharp@stantec.com](mailto:Mike.Sharp@stantec.com)

date: August 12, 2021

file: PG4772-MEMO.03

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Following your request and authorization, Paterson Group (Paterson) prepared the current memorandum to complete a grading plan review from a geotechnical perspective for Phase 2 of the mixed-use development to be constructed at the aforementioned site. The following memorandum should be read in conjunction with Paterson Group Report PG4772-1 Revision 1, dated September 29, 2020.

## Grading Plan Review

Paterson reviewed the following grading plan prepared by Stantec regarding the aforementioned development:

- ☐ Grading Plan - Wellings of Stittsville Phase 2 - Project No. 160401511 - Drawing No. GP-1 - Sheet No. 4 of 7 - Revision 2 - dated August 3, 2021.

Based on our review of the above noted grading plan, the proposed grades within Phase 2 of the aforementioned development are within the permissible grade raise restriction of 2 m provided throughout the subject site in the aforementioned geotechnical investigation report. Therefore, the proposed grading is considered acceptable from a geotechnical perspective. No exceedances of the grade raise restriction were noted, therefore lightweight fill or other considerations to accommodate the proposed grades are not required at this time.

We trust that this information satisfies your immediate requirements.

Best Regards,

**Paterson Group Inc.**



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## Geotechnical Investigation

Proposed Mixed-Use Development  
Wellings of Stittsville - Phase 2, 3 and 4  
20 Cedarow Court  
Ottawa, Ontario

Prepared For

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February 17, 2022

Report PG4772-1 Revision 2

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## Appendices

- Appendix 1      Soil Profile and Test Data Sheets  
                     Symbols and Terms  
                     Analytical Testing Results
- Appendix 2      Figure 1 - Key Plan  
                     Figures 2 to 4 - Slope Stability Analysis Sections  
                     Drawing PG4772-1 - Test Hole Location Plan

## 1.0 Introduction

Paterson Group (Paterson) was commissioned by Nautical Lands Group to conduct a geotechnical investigation for the proposed mixed-use development to be located at 20 Cedarow Court in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

- ❑ Determine the subsurface conditions by means of boreholes.
- ❑ Provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design.

The following report has been prepared specifically and solely for the aforementioned project. This report contains geotechnical findings and includes recommendations pertaining to the design and construction of the proposed development as understood at the time of writing this report.

## 2.0 Proposed Development

Based on the available drawings, it is our understanding that the proposed development will consist of four, six (6) storey mixed-use buildings with a shared underground parking level occupying the majority of the footprint of the subject site. The buildings are understood to include retail, office space and residential units. Associate at-grade parking areas, access lanes, amenity and landscaped areas are also anticipated a part of the development. It is also anticipated that the proposed development will be municipally serviced.

## **3.0 Method of Investigation**

### **3.1 Field Investigation**

#### **Field Program**

The field program for the current investigation was carried out from January 14, 2019 to January 18, 2019. At that time, 29 boreholes were drilled to a maximum depth of 4 m below existing grade.

A supplemental field investigation was conducted on February 2, 2022. At that time, 3 boreholes were advanced to the bedrock surface and cored a maximum depth of 3.2 m into the bedrock surface.

The borehole locations were distributed in a manner to provide general coverage of the proposed development. The locations of the boreholes are shown on Drawing PG4772-1 - Test Hole Location Plan included in Appendix 2.

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

#### **Sampling and In Situ Testing**

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

Standard Penetration Tests (SPT) were conducted and recorded as “N” values on the Soil Profile and Test Data sheets. The “N” value is the number of blows required to drive the split-spoon sample 300 mm into the soil after the initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm.

Undrained shear strength tests were conducted in cohesive soils with a field vane apparatus.

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Rock core samples were recovered from boreholes BH 1-22, BH 2-22 and BH 3-22 drilled during the supplemental investigation using a core barrel and diamond drilling techniques.

The bedrock samples were classified on site, placed in hard cardboard core boxes and transported to Paterson's laboratory. The depths at which rock core samples were recovered from the boreholes are presented as RC on the Soil Profile and Test Data sheets in Appendix 1.

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

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

### **Groundwater**

Flexible polyethylene standpipes were installed in the majority of the boreholes to permit groundwater results subsequent to the sampling program completion. Monitoring wells were installed in BH 4, BH 9, BH 15, BH 22, and BH 27 to provide general site coverage as part of our hydrogeological study. The groundwater observations are discussed in Subsection 4.3 and presented in the Soil Profile and Test Data Sheets in Appendix 1.

### **Sample Storage**

All rock core samples from the supplemental investigation will be stored in the laboratory for a period of one month after issuance of this report at which time the samples will be discarded unless otherwise directed.

## **3.2 Field Survey**

The borehole locations were selected by Paterson taking in consideration site features. The ground surface at the test pit locations was located and surveyed by Annis, O'Sullivan, Vollebakk LTD. It is understood that the ground surface elevations at the borehole locations were referenced to a geodetic datum. The locations and ground surface elevation at the boreholes are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

### **3.3 Laboratory Testing**

Soil samples recovered from the subject site were visually examined in our laboratory to review the field logs.

### **3.4 Analytical Testing**

One (1) soil sample was submitted for analytical testing to assess the potential for exposed ferrous metals and the sulphate potential against subsurface concrete structures. The results are discussed further in Subsection 6.7.



## **4.0 Observations**

### **4.1 Surface Conditions**

The subject site is currently undeveloped and grass covered with a tree-line located along the west boundary line of Cedarow Court. The ground surface across the site is relatively flat and approximately 1 m lower than adjacent properties and Hazeldean Road. Poole Creek ravine runs along the western border of the subject site approximately 3 m below the subject site.

The subject site is bordered by an active construction site for Phase 1 of the Wellings of Stittsville development along the north, Hazeldean Road along the east, and commercial buildings at the edge of Cedarow Court along the south.

### **4.2 Subsurface Profile**

#### **Overburden**

The subsurface profile at the borehole locations consists of topsoil overlying a hard to very stiff silty clay crust followed by a grey, very stiff to stiff silty clay layer. Glacial till was encountered below the silty clay layer consisting of compact silty sand to sandy silt with clay, gravel, cobbles and boulders. A deposit of very stiff to hard clayey silt was encountered below the topsoil in BH 17, BH 18, BH 24, BH 25, BH 26, and BH 27. Practical refusal to augering on inferred bedrock was encountered in all boreholes at depths ranging between 1.6 to 4.0 m. Specific details of the soil profile at each test hole location are presented on the Soil Profile and Test Data sheets provided in Appendix 1.

#### **Bedrock**

Bedrock was cored in 3 boreholes to a maximum depth of 3.2 m below the bedrock surface. The bedrock in borehole BH 1-22 was observed to have an RQD value of 100%. This is indicative of a fair to excellent quality bedrock. The average RQD value in boreholes BH 2-21 was generally between 82 and 100% which is an indicative of good to excellent quality bedrock. The upper portion of the bedrock in borehole BH 3-21 had an RQD value of 64%, indicative of fair quality bedrock whereas the remainder of the bedrock was found to be in good to excellent quality. Reference should be made to the Soil Profile and Test Data sheets in Appendix 1 for the details of the soil profile encountered at borehole location.

Based on available geological mapping, the subject site consists of interbedded dolostone and limestone of the Gull River formation and an approximate drift thickness of 2 to 15 m.

### **4.3 Groundwater**

The measured groundwater levels at the borehole locations are presented in Table 1. Groundwater readings recorded in flexible piezometers could be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater level can also be estimated based on observations of the recovered soil samples, such as the moisture level, soil consistency and colouring. Based on these observations, the long-term groundwater level is anticipated at a depth ranging between 2.5 to 3.5 m below existing grade. Groundwater levels are subject to seasonal fluctuations and could vary at the time of construction.

<b>Table 1 - Groundwater Readings Summary</b>				
<b>Test Hole Number</b>	<b>Ground Elevation (m)</b>	<b>Groundwater Levels (m)</b>		<b>Recording Date</b>
		<b>Depth</b>	<b>Elevation</b>	
BH 1	104.37	DRY	n/a	January 29, 2019
BH 2	103.59	3.05	100.54	January 29, 2019
BH 3	103.55	1.81	101.74	January 29, 2019
BH 4	103.28	3.05	100.23	January 29, 2019
BH 5	103.45	3.05	100.40	January 29, 2019
BH 6	103.49	3.04	100.45	January 29, 2019
BH 7	103.41	DRY	n/a	January 29, 2019
BH 8	103.46	DRY	n/a	January 29, 2019
BH 9	103.42	3.17	100.25	January 29, 2019
BH 10	103.31	2.18	101.13	January 29, 2019
BH 11	103.44	DRY	n/a	January 29, 2019
BH 12	103.58	DRY	n/a	January 29, 2019
BH 13	103.55	DRY	n/a	January 29, 2019
BH 14	104.18	DRY	n/a	January 29, 2019
BH 15	103.65	2.92	100.73	January 29, 2019
BH 16	103.66	DRY	n/a	January 29, 2019
BH 17	104.19	DRY	n/a	January 29, 2019
BH 18	104.15	DRY	n/a	January 29, 2019
BH 19	103.78	DRY	n/a	January 29, 2019
BH 20	103.59	DRY	n/a	January 29, 2019
BH 21	103.58	DRY	n/a	January 29, 2019
BH 22	103.65	DRY	n/a	January 29, 2019
BH 23	103.87	2.62	101.25	January 29, 2019
BH 24	104.04	2.55	101.49	January 29, 2019
BH 25	104.07	1.68	102.39	January 29, 2019
BH 26	104.30	DRY	n/a	January 29, 2019
BH 27	103.97	DRY	n/a	January 29, 2019
BH 28	103.78	DRY	n/a	January 29, 2019
BH 29	103.71	DRY	n/a	January 29, 2019

**Note:** The ground surface elevation at the borehole locations was provided by Annis, O’Sullivan, Vollebekk Ltd.

## 5.0 Discussion

### 5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is suitable for the proposed development. The proposed structures will be founded on conventional shallow foundations placed on an undisturbed, hard to very stiff silty clay, compact to dense glacial till and/or clean, surface sounded bedrock bearing surface. Alternatively, conventional shallow footings can be placed over a near vertical, zero entry, concrete in-filled trenches extending to a clean, surface sounded bedrock bearing surface.

Permissible grade raise restriction areas are also required due to the silty clay deposit. A permissible grade raise restriction of **2 m** is recommended for areas where settlement sensitive structures are founded over the silty clay deposit.

Depending on the extent of the underground parking garage and potential grade raise, the bedrock may be encountered during excavation and construction. All contractors should be prepared for bedrock removal within the subject site.

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

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

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

### 5.2 Site Grading and Preparation

#### Stripping Depth

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

## **Bedrock Removal**

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

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

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

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

Excavation side slopes in sound bedrock can be excavated almost vertical side walls. A minimum 1 m horizontal ledge, should remain between the overburden excavation and the bedrock surface. The ledge will provide an area to allow for potential sloughing or a stable base for the overburden shoring system.

## **Vibration Considerations**

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

The following construction equipments could be the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters are used to determine the permissible vibrations, namely, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity is less than that for high frequency vibrations. As a guideline, the peak particle velocity should be less than 15 mm/s between frequencies of 4 to 12 Hz, and 50 mm/s above a frequency of 40 Hz (interpolate between 12 and 40 Hz). The guidelines are for current construction standards.

Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended be completed to minimize the risks of claims during or following the construction of the proposed buildings.

### **Fill Placement**

Fill placed for grading beneath the structure(s) or other settlement sensitive areas should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The engineered fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material’s standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be placed as general landscaping fill where surface settlement is a minor concern. The backfill materials should be spread in thin lifts and at a minimum compacted by the tracks of the spreading equipment to minimize voids. If the non-specified backfill is to be placed to increase the subgrade level for areas to be paved, the fill should be compacted in maximum 300 mm lifts and compacted to 95% of the material’s SPMDD. Non-specified existing fill and site-excavated soils are not suitable for placement as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

## **5.3 Foundation Design**

### **Bearing Resistance Values (Shallow Foundation)**

Footings for the proposed buildings can be designed with the following bearing resistance values presented in Table 2.

<b>Table 2 - Bearing Resistance Values</b>		
<b>Bearing Surface</b>	<b>Bearing Resistance Value at SLS (kPa)</b>	<b>Factored Bearing Resistance Value at ULS (kPa)</b>
Very stiff to hard silty clay	150	250
Compact to dense glacial till	200	300
Weathered Limestone Bedrock	-	1500
Clean, Surface Sounded Limestone Bedrock	-	2000

<b>Table 2 - Bearing Resistance Values</b>		
<b>Bearing Surface</b>	<b>Bearing Resistance Value at SLS (kPa)</b>	<b>Factored Bearing Resistance Value at ULS (kPa)</b>
Lean Concrete In-filled Trenches	-	2000
<p>Note: Strip footings, up to 3 m wide, and pad footings, up to 8 m wide, placed over an undisturbed, silty clay bearing surface can be designed using the abovenoted bearing resistance values. - A geotechnical resistance factor of 0.5 was applied to the above noted bearing resistance value at ULS.</p>		

The above-noted bearing resistance values at SLS for soil bearing surfaces will be subjected to potential post-construction total and differential settlements of 25 and 20 mm, respectively. Footings bearing on an acceptable bedrock bearing surface and designed for the bearing resistance values provided herein will be subjected to negligible potential post-construction total and differential settlements.

The bearing resistance values are provided on the assumption that the footings are placed on undisturbed soil bearing surfaces. An undisturbed soil bearing surface consists of one from which all topsoil and deleterious materials, such as loose, frozen or disturbed soil, whether in situ or not, have been removed, in the dry, prior to the placement of concrete for footings.

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

### **Lean Concrete Filled Trenches**

Where bedrock is encountered below the design underside of footing elevation, consideration should be given to excavating vertical trenches to expose the underlying bedrock surface and backfilling with lean concrete (**15 MPa** 28-day compressive strength). Typically, the excavation sidewalls will be used as the form to support the concrete. The additional width of the concrete poured against an undisturbed trench sidewall will suffice in providing a direct transfer of the footing load to the underlying bedrock.

The effectiveness of this operation will depend on the ability of maintaining vertical trenches until the lean concrete can be poured. It is suggested that once the bottom of the excavation is exposed, an assessment should be completed to determine the water infiltration and stability of the excavation sidewalls extending to the bedrock surface.

The trench excavation should be at least 300 mm wider than all sides of the footing at the base of the excavation. The excavation bottom should be relatively clean using the hydraulic shovel only (workers will not be permitted in the excavation below a 1.5 m depth). Once approved by the geotechnical engineer, lean concrete can be poured up to the proposed founding elevation.

### **Bedrock/Soil Transition**

Where a building is founded partly on bedrock and partly on soil, it is recommended to decrease the soil bearing resistance value by 25% for the footings placed on soil bearing media to reduce the potential long term total and differential settlements. Also, at the soil/bedrock and bedrock/soil transitions, it is recommended that the upper 0.5 m of the bedrock be removed for a minimum length of 2 m (on the bedrock side) and replaced with nominally compacted OPSS Granular A or Granular B Type II material. The width of the sub-excavation should be at least the proposed footing width plus 0.5 m. Steel reinforcement, extending at least 3 m on both sides of the 2 m long transition, should be placed in the top part of the footings and foundation walls.

### **Lateral Support**

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to an engineered fill, stiff silty clay or glacial till above the groundwater table when a plane extending horizontally and vertically from the underside of the footing at a minimum of 1.5H:1V passing through in situ soil of the same or higher bearing capacity as the bearing medium soil.

### **Permissible Grade Raise Restriction**

Based on the current borehole information, a **permissible grade raise restriction of 2 m** is recommended for the proposed buildings and settlement sensitive structures where founded over a silty clay deposit. A post-development groundwater lowering of 0.5 m was assumed for our calculations.

## **5.4 Design for Earthquakes**

The site class for seismic site response can be taken as **Class C** for the foundations considered at this site. However, a higher site class, such as Class A or B can be provided if a site specific shear wave velocity test is completed to confirm the seismic site classification. The soils underlying the subject site are not susceptible to liquefaction. Refer to the latest revision of the Ontario Building Code for a full discussion of the earthquake design requirements.



## 5.5 Basement Slab

The basement area for the proposed project will be mostly parking and the recommended pavement structure noted in Subsection 5.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be constructed, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. The upper 200 mm of sub-slab fill is recommended to consist of OPSS Granular A crushed stone for slab on grade construction. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

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

A subfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor (discussed in Subsection 6.1).

## 5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the proposed structure's basement walls. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m<sup>3</sup>.

The foundation wall is anticipated to be provided with a perimeter drainage system; therefore, the retained soils should be considered drained. For the undrained conditions, the applicable effective unit weight of the retained soil can be designed with 13 kN/m<sup>3</sup>. A hydrostatic pressure should be added to the total static earth pressure when calculating the effective unit weight. The total earth pressure ( $P_{AE}$ ) includes both the static earth pressure component ( $P_o$ ) and the seismic component ( $\Delta P_{AE}$ ).

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

### Static Conditions

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

- $K_o$  = at-rest earth pressure coefficient of the applicable retained soil, 0.5  
 $\gamma$  = unit weight of fill of the applicable retained soil (kN/m<sup>3</sup>)  
 $H$  = height of the wall (m)

An additional pressure with a magnitude equal to  $K_o \cdot q$  and acting on the entire height of the wall should be added to the above formula for any surcharge loading,  $q$  (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure should only be applicable for static analyses and not be calculated in conjunction with the seismic loading case. Actual earth pressures could be higher than the “at-rest” case if care is not exercised during the compaction of the backfill materials to maintain a minimum separation of 0.3 m from the walls with the compaction equipment.

### Seismic Conditions

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

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

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

The peak ground acceleration, ( $a_{max}$ ), for the Ottawa area is 0.32g according to OBC 2012. The vertical seismic coefficient is assumed to be zero. The earth force component ( $P_o$ ) under seismic conditions could be calculated using  $P_o = 0.5 K_o \gamma H^2$ , where  $K_o = 0.5$  for the soil conditions presented above.

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

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

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

## 5.7 Pavement Structure

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

<b>Table 3 - Recommended Flexible Pavement Structure - At-Grade Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
50	<b>Wear Course</b> - HL-3 or Superpave 12.5 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
300	<b>SUBBASE</b> - OPSS Granular B Type II
	<b>SUBGRADE</b> - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

<b>Table 4 - Recommended Flexible Pavement Structure - Access Lanes and Heavy Truck Parking Areas</b>	
<b>Thickness (mm)</b>	<b>Material Description</b>
40	<b>Wear Course</b> - HL-3 or Superpave 12.5 Asphaltic Concrete
50	<b>Binder Course</b> - HL-8 or Superpave 19.0 Asphaltic Concrete
150	<b>BASE</b> - OPSS Granular A Crushed Stone
450	<b>SUBBASE</b> - OPSS Granular B Type II
	<b>SUBGRADE</b> - In situ soil, or OPSS Granular B Type I or II material placed over in situ soil

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

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the SPMDD.

## **6.0 Design and Construction Precautions**

### **6.1 Foundation Drainage and Backfill**

#### **Foundation Drainage**

A perimeter foundation drainage system is recommended to be provided for the proposed structures. The composite drainage system (such as Miradrain G100N, Delta Drain 6000 or an approved equivalent) is recommended to extend to the footing level. Sleeves, 150 mm diameter, at 3 m centres are recommended to be placed in the footing or at the foundation wall/footing interface for blind sided pours to allow the infiltration of water to flow to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

#### **Underfloor Drainage**

Underfloor drainage is recommend to control water infiltration for the proposed structures. For design purposes, Paterson recommends 150 mm diameter PVC, corrugated, perforated pipes be placed at 3 to 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

#### **Adverse Effects of Dewatering on Adjacent Properties**

Due to the low permeability of the subsoils profile, any dewatering will be considered relatively minor as a result of the proposed construction. Therefore, adverse effects to the surrounding buildings or properties are not expected with respect to any groundwater lowering.

#### **Foundation Backfill**

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls where frost heave sensitive structures, such as a concrete sidewalk, will be placed. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material may be used for this purpose. A composite drainage system, such as Delta Drain 6000, Miradrain G100 or an approved equivalent, should be placed against the foundation wall to promote drainage toward the perimeter drainage pipe.

## **6.2 Protection of Footings Against Frost Action**

Perimeter footings of heated structures are recommended to be protected against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a combination of soil cover and foundation insulation should be provided.

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

The parking garage should not require protection against frost action due to the founding depth. Unheated structures, such as the access ramp wall footings, may be required to be insulated against the deleterious effect of frost action. A minimum of 2.1 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with foundation insulation, should be provided.

## **6.3 Excavation Side Slopes**

### **Temporary Side Slopes**

The temporary excavation side slopes should either be excavated to acceptable slopes or retained by shoring systems from the beginning of the excavation until the structure is backfilled.

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

Excavated soil should not be stockpiled directly at the top of excavations and heavy equipment should maintain safe working distance from the excavation sides.

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

A trench box is recommended to be installed at all times to protect personnel working in trenches with steep or vertical sides. Services are expected to be installed by “cut and cover” methods and excavations should not be remain exposed for extended periods of time.

## Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements designed by a structural engineer specializing in those works will depend on the excavation depths, the proximity of the adjacent structures and the elevation of the adjacent building foundations and underground services. The design and implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's structural designer prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be included to the earth pressures described below. These systems could be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the shoring system may be calculated with the following parameters.

<b>Table 6 - Soil Parameters</b>	
<b>Parameters</b>	<b>Values</b>
Active Earth Pressure Coefficient ( $K_a$ )	0.33
Passive Earth Pressure Coefficient ( $K_p$ )	3
At-Rest Earth Pressure Coefficient ( $K_o$ )	0.5
Dry Unit Weight ( $\gamma$ ), kN/m <sup>3</sup>	20
Effective Unit Weight ( $\gamma$ ), kN/m <sup>3</sup>	13

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

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

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

## **6.4 Pipe Bedding and Backfill**

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

A minimum of a 150 mm layer of OPSS Granular A crushed stone should be placed for pipe bedding for sewer and water pipes for a soil subgrade. The bedding thickness should be increased to 300 mm for areas where the subgrade consists of bedrock. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 300 mm above the obvert of the pipe should consist of OPSS Granular A. The bedding and cover materials should be placed in maximum 300 mm thick lifts compacted to a minimum of 95% of the SPMDD.

The site excavated material may be placed above cover material if the excavation operations are completed in dry weather conditions and the site excavated material is approved by the geotechnical consultant. All cobbles greater than 200 mm in the longest dimension should be removed prior to the site materials being reused.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the SPMDD. Within the frost zone (1.8 m below finished grade), non frost susceptible materials should be used when backfilling trenches below the original bedrock level.

Clay seals are recommended for the subject site. The seals should be a minimum of 1.5 m long (in the trench direction) and should extend from trench wall to trench wall. Generally, the seals should extend from the frost line and fully penetrate the bedding, subbedding and cover material. The barriers should consist of relatively dry and compactable brown silty clay placed in maximum 225 mm thick loose layers and compacted to a minimum of 95% of the SPMDD. The clay seals should be placed at the site boundaries, roadway intersections and at a maximum distance of every 50 m in the service trenches.

## **6.5 Groundwater Control**

### **Groundwater Control for Building Construction**

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

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

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



## **Long-term Groundwater Control**

Any groundwater encountered along the buildings' perimeter or sub-slab drainage system will be directed to the proposed buildings' cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the expected long-term groundwater flow should be low (i.e. less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The long-term groundwater flow is anticipated to be controllable using conventional open sumps.

## **Impacts on Neighbouring Properties**

A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed buildings. It should be noted that the neighbouring multi-storey buildings are expected to be founded over the bedrock surface and would not be affected by the short-term groundwater lowering during construction. The water table is located within the glacial till layer and/or bedrock surface. Based on the existing groundwater level, the extent of any significant groundwater lowering will take place within a limited range of the proposed building. Based on the proximity of neighbouring buildings and minimal zone impacted by the groundwater lowering, the proposed development will not negatively impact the neighbouring structures or City infrastructures.

## **6.6 Winter Construction**

Precautions must be provided if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions in the contract documents should be provided to protect the excavation walls from freezing, if applicable.

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

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

## **6.7 Corrosion Potential and Sulphate**

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

## **6.8 Limit of Hazard Lands**

### **Field Observations**

Paterson conducted a site visit on January 13, 2019 to review the slope located along the west boundary of the subject site, assess the current slope conditions and confirm the grades provided in the existing topographic mapping. A section of Poole Creek is located within the west portion of the site and shown in Drawing PG4772-1 - Test Hole Location Plan.

Three (3) slope cross-sections were reviewed in the field as the worst case scenarios. The cross section locations are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2. Generally, the riverbanks along both sides of Poole Creek are currently well vegetated and were observed in an acceptable condition. Poole Creek was observed within a 20 to 40 m wide flood plain. The slope along the east side of Poole Creek ranged in height between 3 and 5 m with an inclination ranging between 2.3H:1V and 3.3H:1V. The upper slope was observed to be well vegetated with little to no signs of active surficial erosion.

### **Slope Stability Analysis**

#### *Limit of Hazard Lands*

The slope condition was reviewed based on available topographic mapping along the east side slopes of Poole Creek within the west portion of the subject development. A total of 3 slope cross-sections were assessed as the worst case scenarios. The cross section locations are presented on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

A slope stability assessment was carried out to determine the required stable slope allowance setback from the top of slope based on a factor of safety of 1.5. A toe erosion and 6 m erosion access allowances were also included in the determination of limits of hazard lands and are discussed below. The proposed limit of hazard lands (as shown on Drawing PG4772-1 - Test Hole Location Plan) includes:

- a geotechnical slope stability allowance with a factor of safety of 1.5
- a toe erosion allowance
- a 6 m erosion access allowance and top of slope

### Slope Stability Analysis

The analysis of the stability of the slope sections was carried out using SLIDE, a computer program which permits a two-dimensional slope stability analysis using several methods including the Bishop's method, which is a widely used and accepted analysis method. The program calculates a factor of safety, which represents the ratio of the forces resisting failure to those favoring failure. Theoretically, a factor of safety of 1.0 represents a condition where the slope is stable. However, due to intrinsic limitations of the calculation methods and the variability of the subsoil and groundwater conditions, a factor of safety greater than one is usually required to ascertain that the risks of failure are acceptable. A minimum factor of safety of 1.5 is generally recommended for conditions where the failure of the slope would endanger permanent structures.

An analysis considering seismic loading was also completed. A horizontal acceleration of 0.16G was considered for the sections for the seismic loading condition. A factor of safety of 1.1 is considered to be satisfactory for stability analyses including seismic loading.

The cross-sections were analysed taking into account a groundwater level at ground surface, which represents a worse-case scenario that can be reasonably expected to occur in cohesive soils. The stability analysis assumes full saturation of the soil with groundwater flow parallel to the slope face. Subsoil conditions at the cross-sections were inferred based on the findings at borehole locations along the top of slope and general knowledge of the area's geology.

### Stable Slope Allowance

The results of the stability analysis for static conditions at Sections A through C are presented in Figures 2A to 4A in Appendix 2. All the reviewed slope sections along the subject creek were noted to be shaped to at least a 2.3H:1V. Based on the soil conditions observed and the results of the slope stability analysis, the slope stability factor of safety was calculated to be 1.5 or greater for all the slope sections which indicates that a stable slope allowance is not required for the subject slope.

The results of the analyses including seismic loading are shown in Figures 2B to 4B for the slope sections. The results indicate that the factor of safety for the sections are greater than 1.1.

It should be noted that the existing vegetation on the slope face should not be removed as it contributes to the stability of the slope and reduces erosion. If the existing vegetation needs to be removed, it is recommended that a 100 to 150 mm of topsoil mixed with a hardy seed and/or topped with an erosion control blanket be which can be placed across the exposed slope face.

#### Toe Erosion and Erosion Access Allowance

The toe erosion allowance for the valley corridor wall slope was based on the cohesive nature of the top layers of the subsoils, the observed current erosional activities and the width and location of the current watercourse. It should be noted that if the flood plain is measured to be greater than 20 m, no toe erosion will be required. Therefore, based on the above factors, no toe erosion allowance is considered for the subject slope.

An erosion access allowance of 6 m is required from the top of slope to ensure access is provided should future maintenance to the slope face is required. The limit of hazard lands, which includes these allowances, is indicated on Drawing PG4772-1 - Test Hole Location Plan in Appendix 2.

#### Proposed Conditions

An analysis was conducted following a review of the proposed grade raise and development. It is understood that storm water storage tanks are proposed on the north portion of the site. The proposed conditions are presented in Figure 2C, 3C and 4C in Appendix 1. Following a review of the proposed conditions, the slope will not be impacted by the proposed development.

## **6.9 Landscaping Considerations**

### **Tree Planting Restrictions**

According to the City of Ottawa Guidelines for tree planting, where a sensitive silty clay deposit is present within the vicinity of the site, tree planting restrictions should be determined. However, for this site, based on the founding medium of the underground parking level which will occupy the majority of the site, tree planting restrictions are not required from a geotechnical perspective.

## **6.10 Storm water storage tanks**

Based on existing servicing drawings, it is understood that storm water storage tanks are proposed along the north portion of the site. The tanks are approximately 3.6 m wide, 3.6 m deep and 9.1 m long. The tanks are expected to be fully buried with an invert level of approximately 99.7 m with a soil cover of approximately 1 m above the top of the tanks. Frost protection will not be required due to the founding depth.

Due to the depth of the storm tanks and the estimated depth of the groundwater table, the tanks should be waterproofed up to 1 m above the estimated groundwater table. It should be noted that the fill placed around and above the water tanks will provide sufficient resistance to the expected buoyancy forces resulting from the long-term groundwater table. Therefore, the proposed water storage tanks are considered acceptable from a geotechnical perspective.

Due to the expected founding depth, the tanks should be installed on OPSS Granular A or Granular B Type II extending to the bedrock surface and compacted to a minimum 98% of the material's SPMDD.

Reference should be made to subsection 6.8 for slope stability analysis and limit of hazard lands setback for the development. The setback from the top of slope and the tanks is sufficient, therefore, the slope will not be negatively impacted by the proposed storm water storage tanks.

## 7.0 Recommendations

A materials testing and observation services program is a requirement for the provided foundation design data to be applicable. The following aspects of the program should be performed by the geotechnical consultant:

- Review detailed grading plan(s) from a geotechnical perspective.
- Review groundwater conditions at the time of construction to determine if waterproofing is required.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials used.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- Observation of all subgrades prior to backfilling.
- Field density tests to determine the level of compaction achieved.
- Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that the construction work has been conducted in general accordance with the above recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

## 8.0 Statement of Limitations

The recommendations provided in the report are in accordance with Paterson's present understanding of the project. Paterson request permission to review the recommendations when the drawings and specifications are completed.

A geotechnical investigation is a limited sampling of a site. Should any conditions encountered during construction differ from the borehole locations, Paterson requests immediate notification to permit reassessment of the recommendations provided herein.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Nautical Lands Group or their agent(s) is not authorized without review by Paterson for the applicability of our recommendations to the altered use of the report.

### Paterson Group Inc.



Owen Canton, E.I.T.



Faisal I. Abou-Seido, P.Eng.

### Report Distribution:

- Nautical Lands Group (Digital copy)
- Paterson Group (1 copy)

# **APPENDIX 1**

**SOIL PROFILE AND TEST DATA SHEETS**

**SYMBOLS AND TERMS**

**ANALYTICAL TESTING RESULTS**



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH1-22**

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
OVERBURDEN						0	103.32					
						1	102.32					
						2	101.32					
BEDROCK						3	100.32					
						4	99.32					
			RC	1	100	100	5	98.32				
			RC	2	100	100	6	97.32				
End of Borehole												

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

FILE NO. **PG4772**

HOLE NO. **BH2-22**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			○ Water Content %				
GROUND SURFACE								20	40	60	80	
OVERBURDEN						0	103.34					
						1	102.34					
						2	101.34					
BEDROCK						3	100.34					
						4	99.34					
						5	98.34					
End of Borehole												

3.51

5.72

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH3-22**

BORINGS BY CME 55 Power Auger

DATE February 2, 2022

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	103.81						
OVERBURDEN						1	102.81						
						2	101.81						
	2.29					3	100.81						
BEDROCK		RC	1	100	64								
		RC	2	100	95								
		RC	3	97	97								
	4.90												
End of Borehole													

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed    △ Remoulded

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

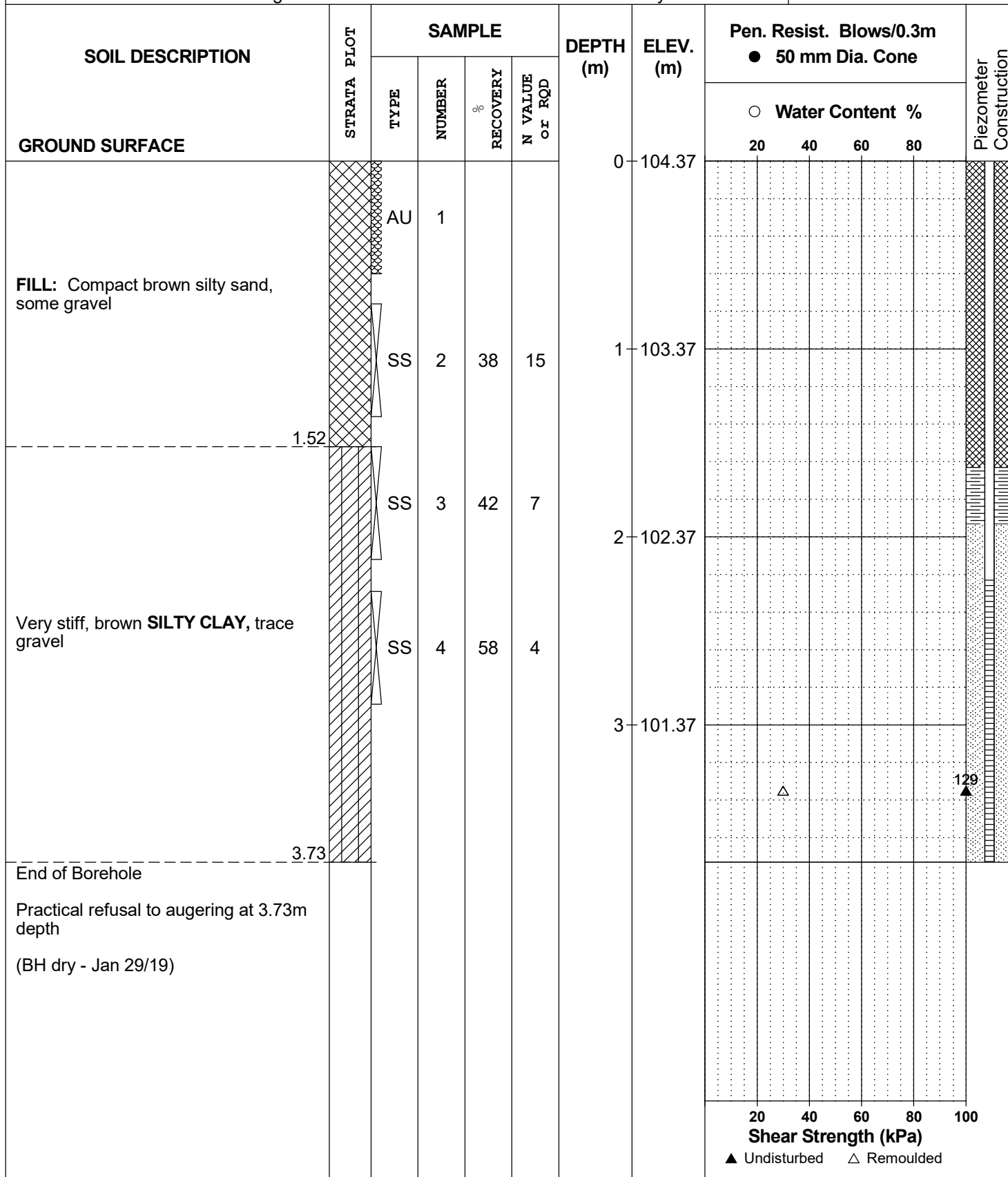
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REMARKS

HOLE NO. **BH 1**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



## SOIL PROFILE AND TEST DATA

**Geotechnical Investigation**  
Proposed Mixed-Use Development - 20 Cedarow Ct.  
Ottawa, Ontario

**DATUM** Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

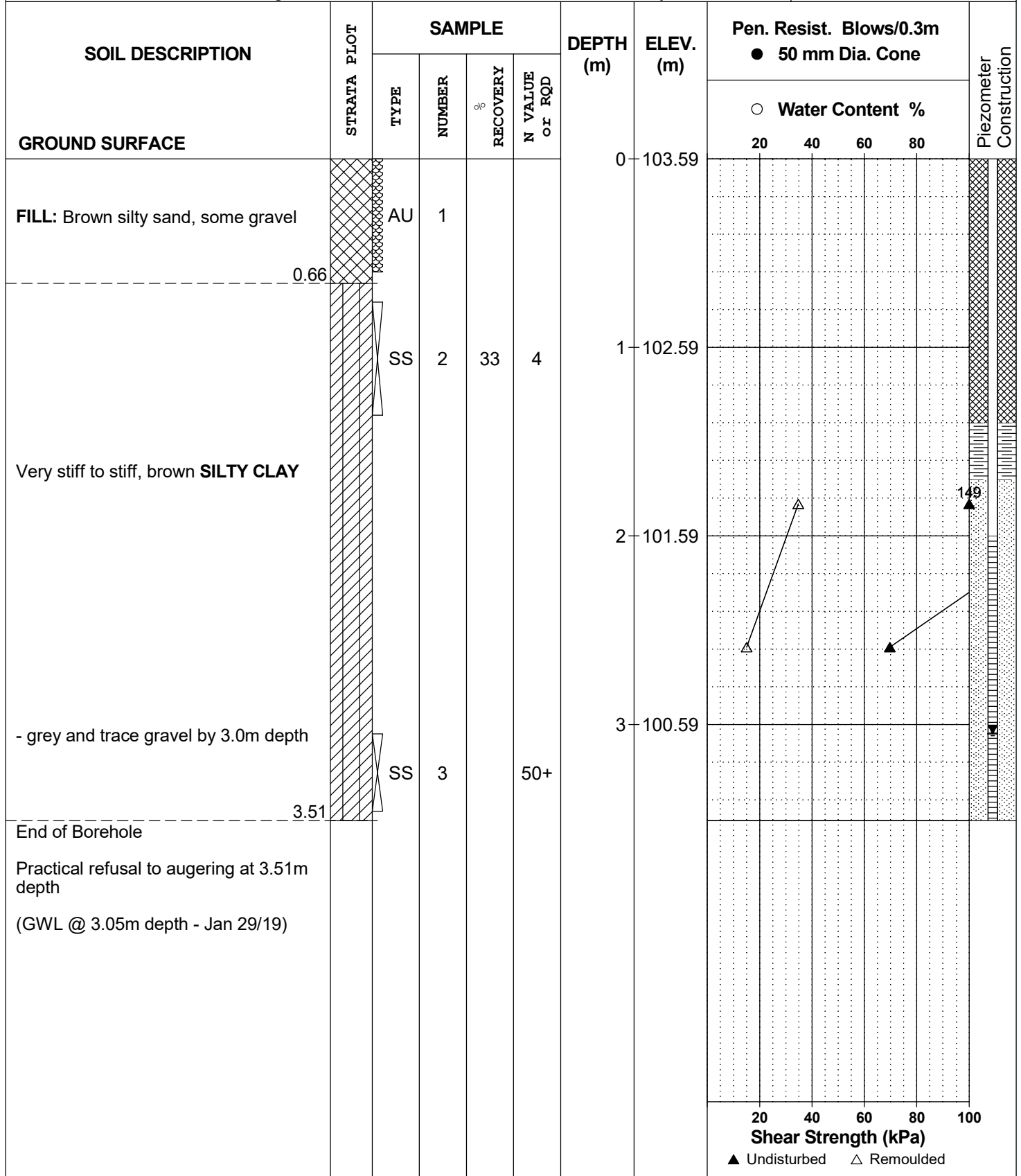
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**PG4772**

**REMARKS**

**HOLE NO.**  
**BH 2**

**BORINGS BY** CME 55 Power Auger

**DATE** 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

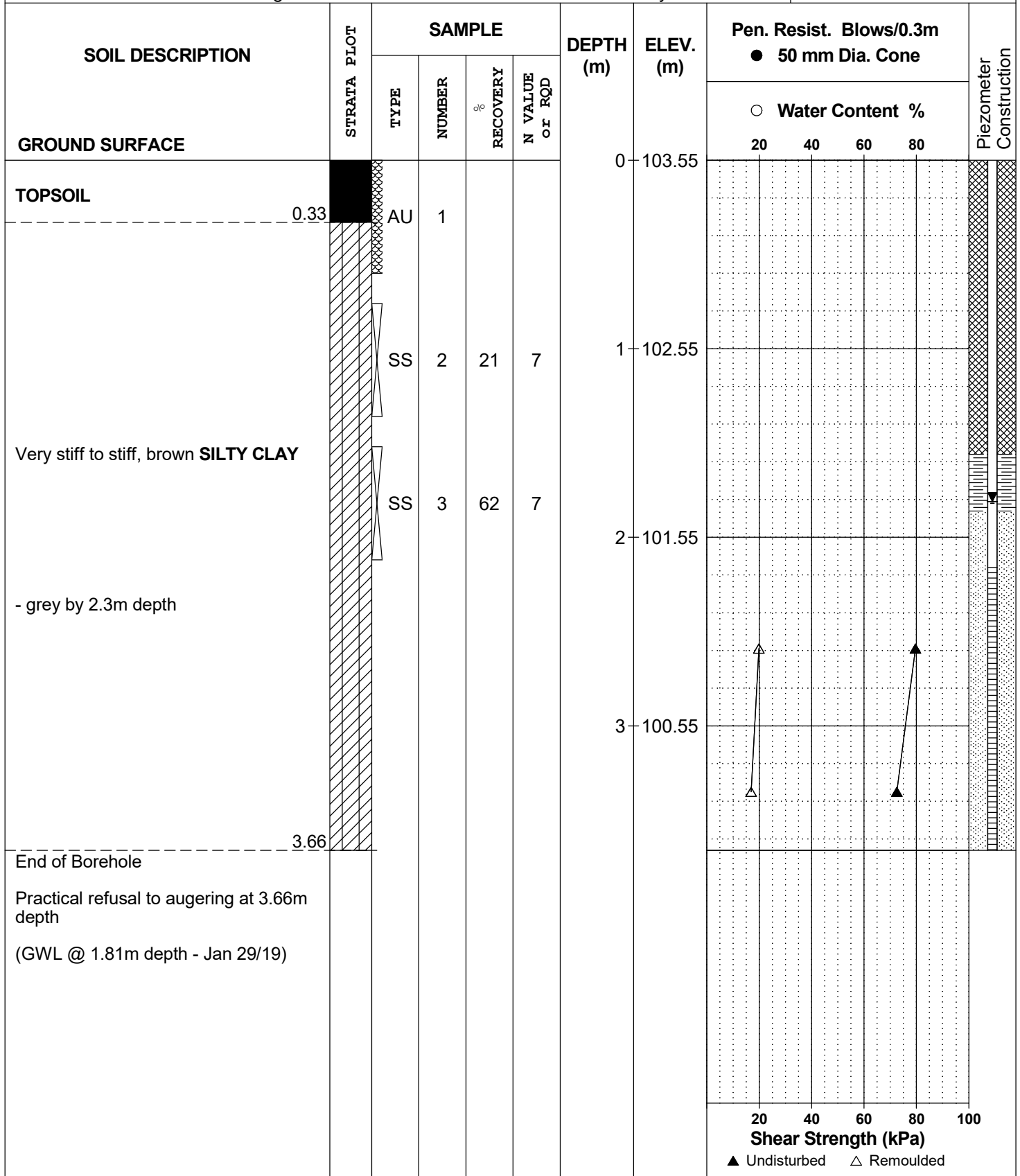
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REMARKS

HOLE NO. **BH 3**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

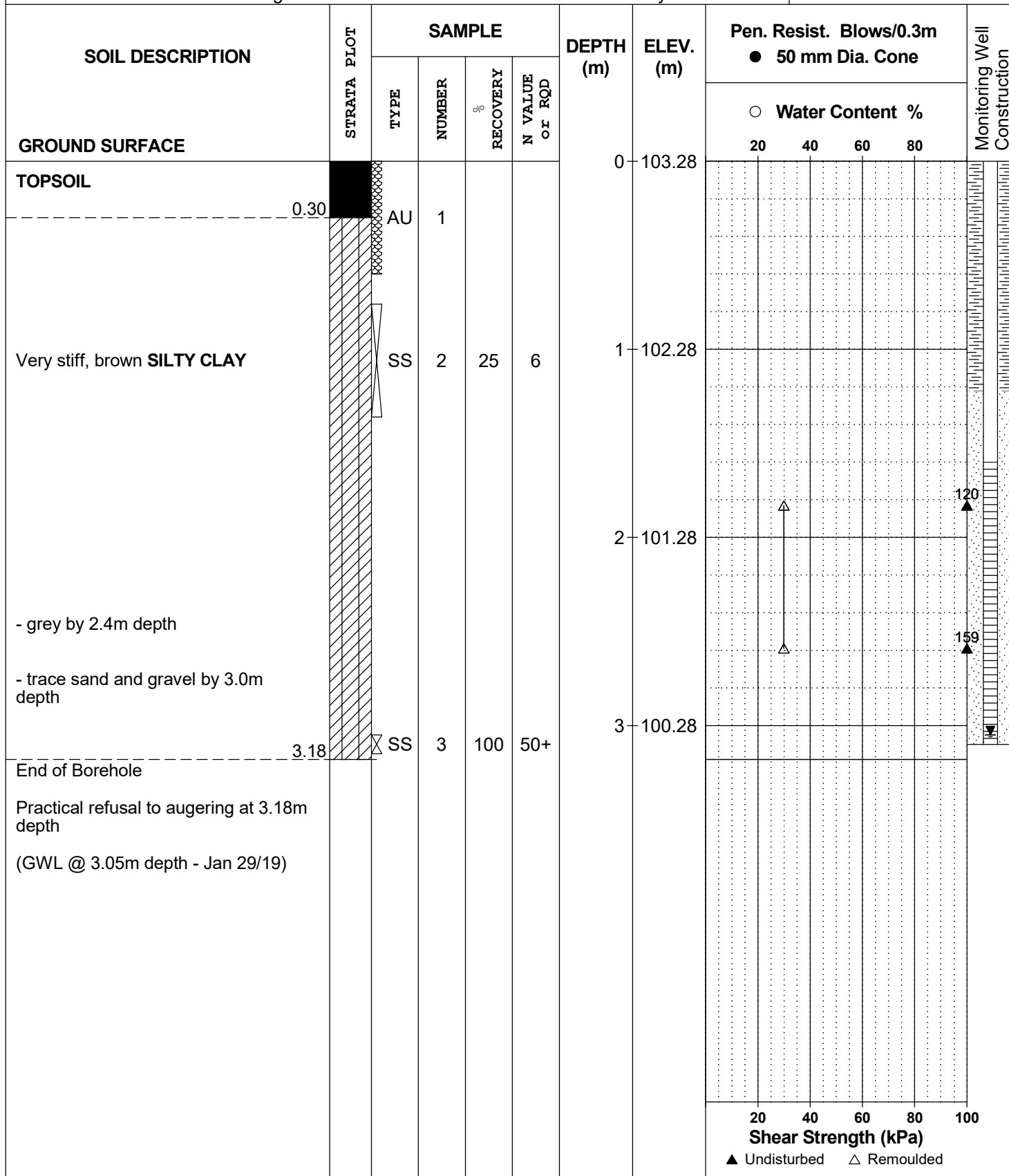
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REMARKS

HOLE NO. **BH 4**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

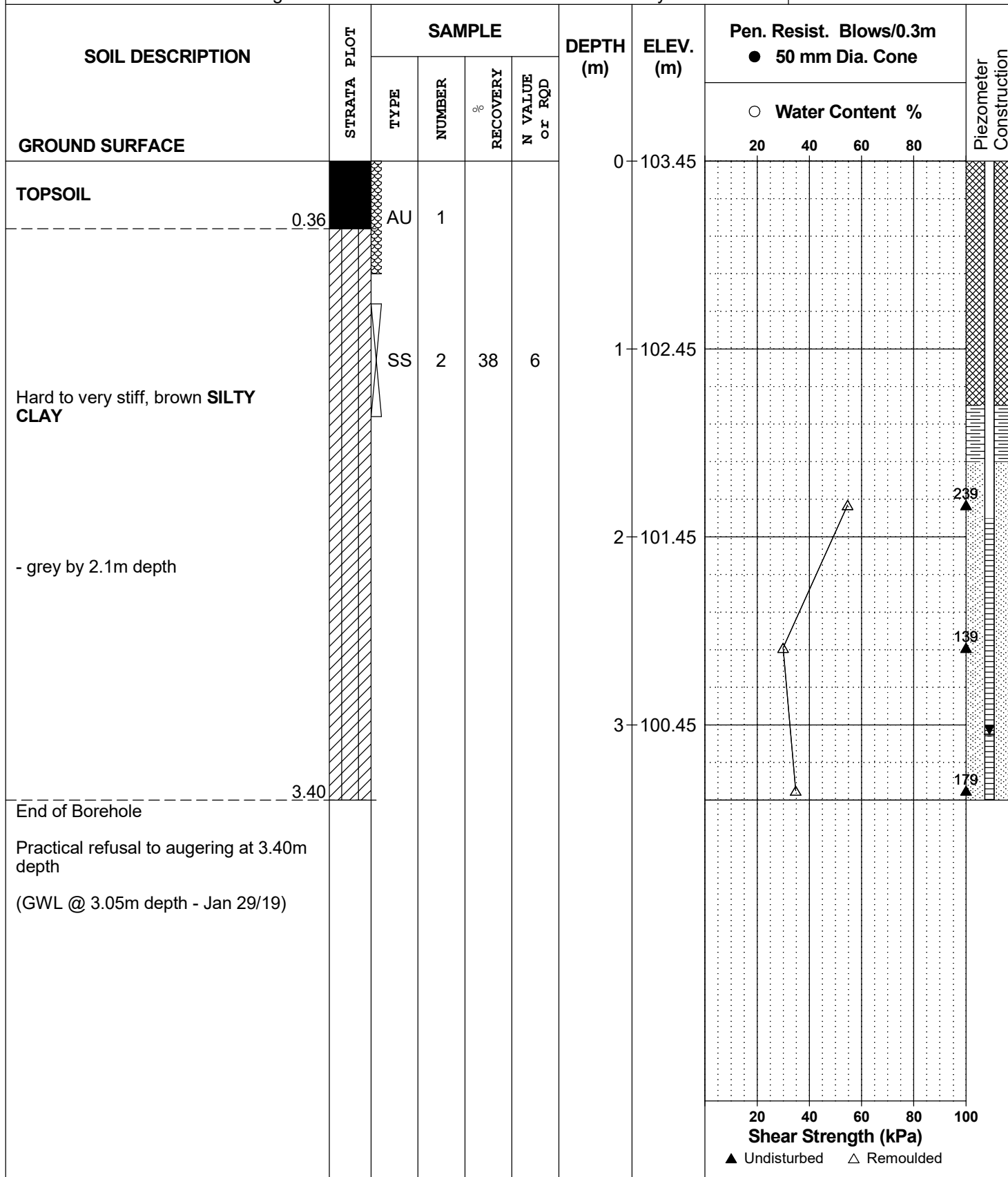
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REMARKS

HOLE NO. **BH 5**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14





DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

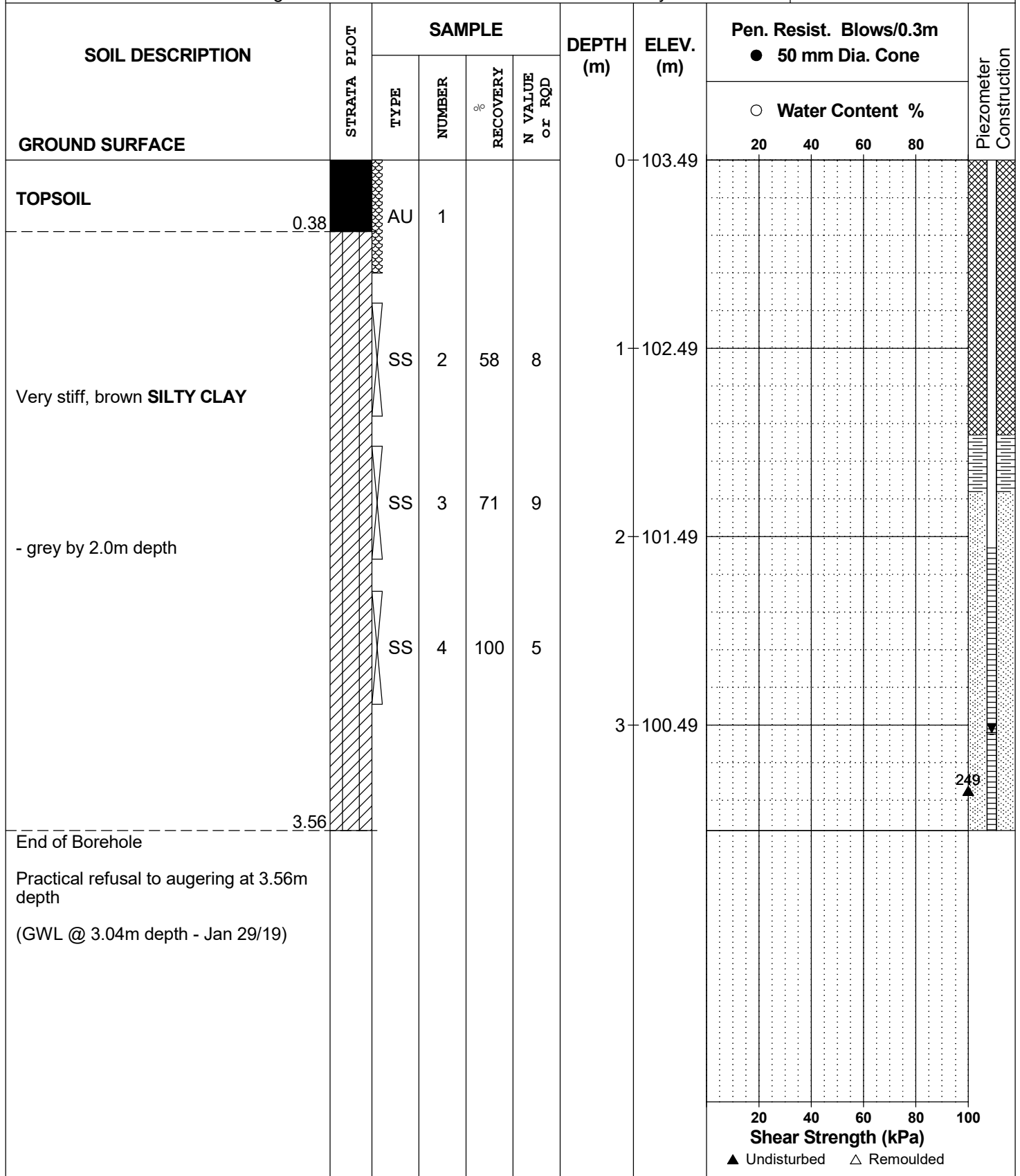
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REMARKS

HOLE NO. **BH 6**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

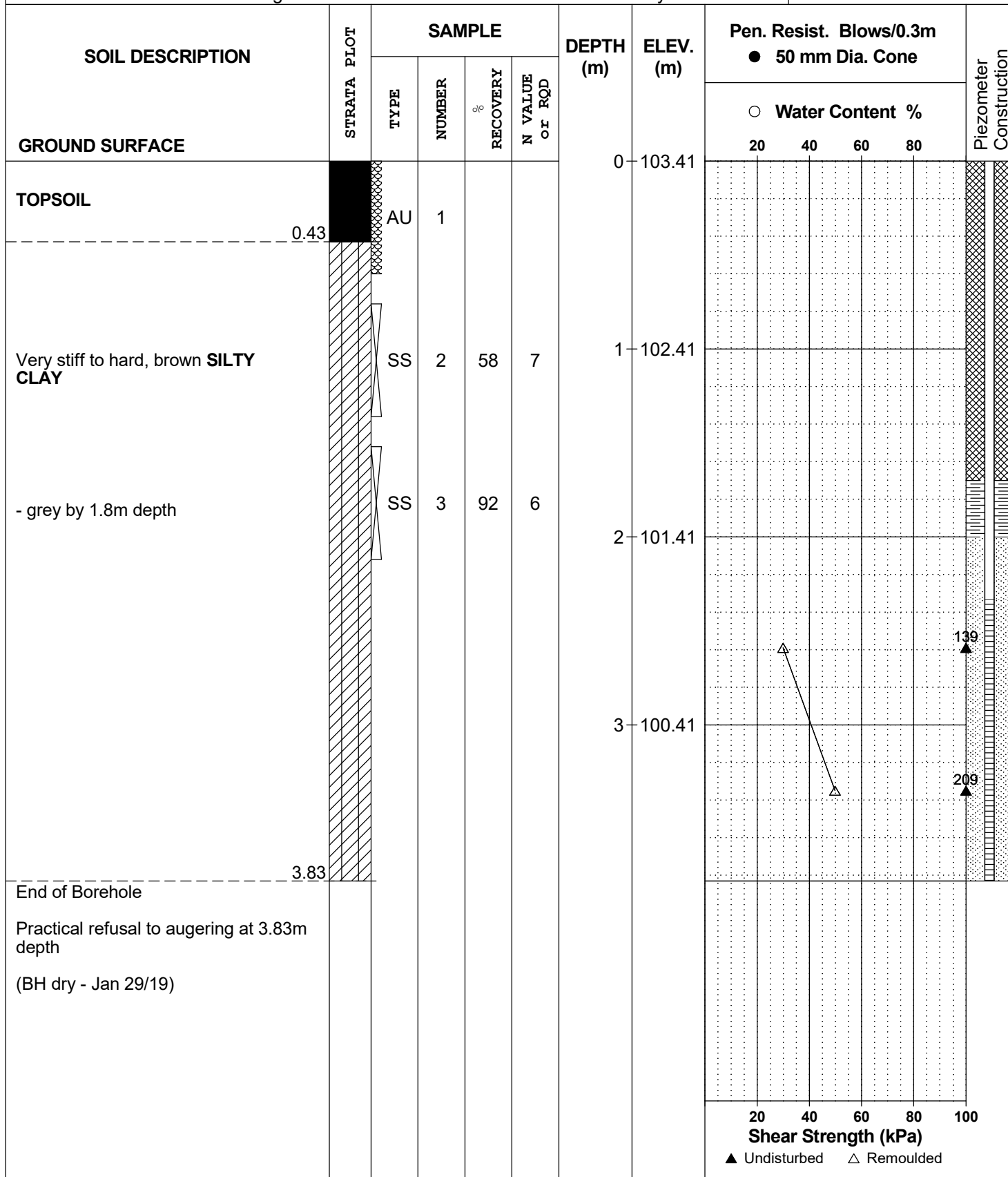
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REMARKS

HOLE NO. **BH 7**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

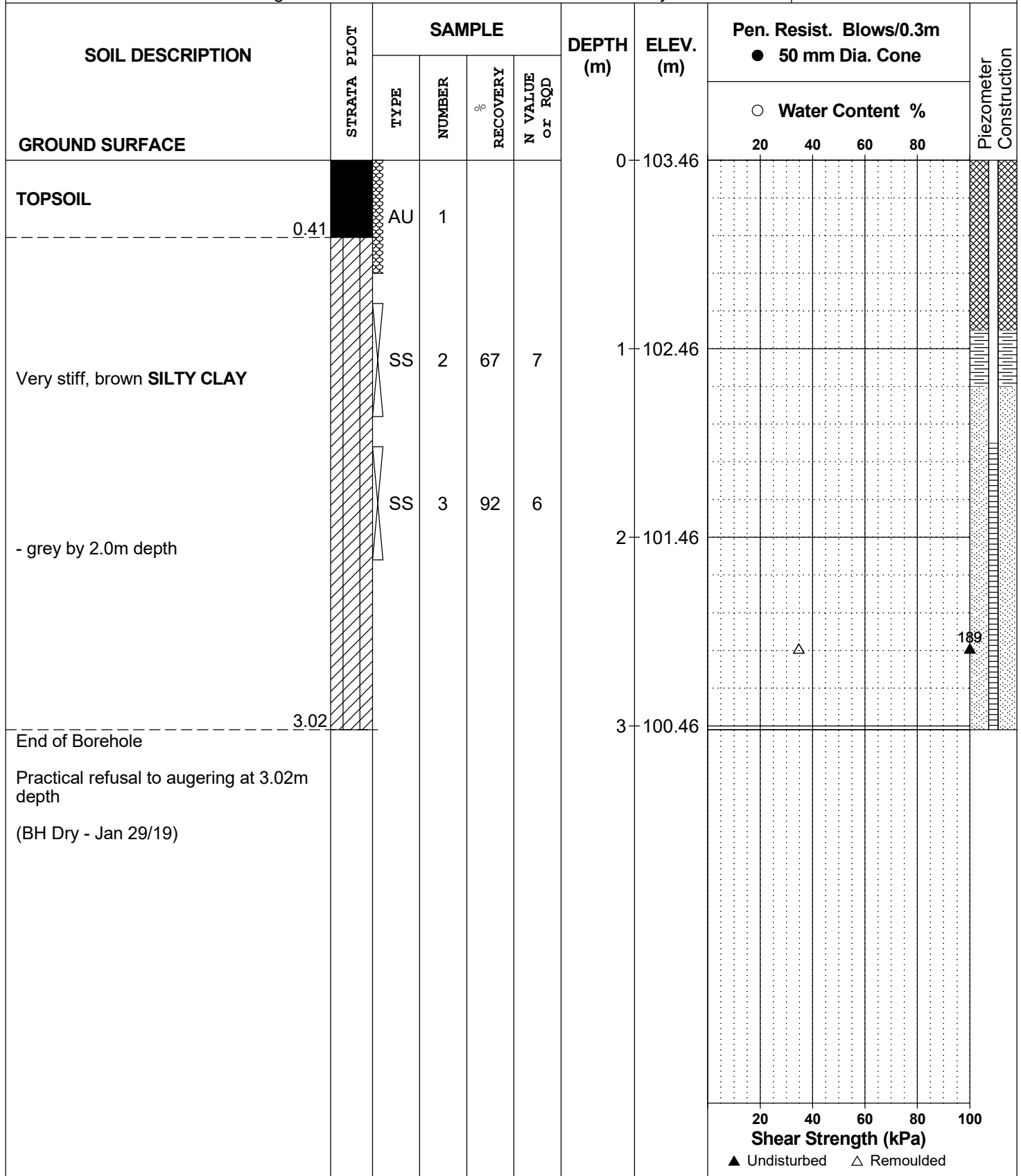
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REMARKS

HOLE NO. **BH 8**

BORINGS BY CME 55 Power Auger

DATE 2019 January 14



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

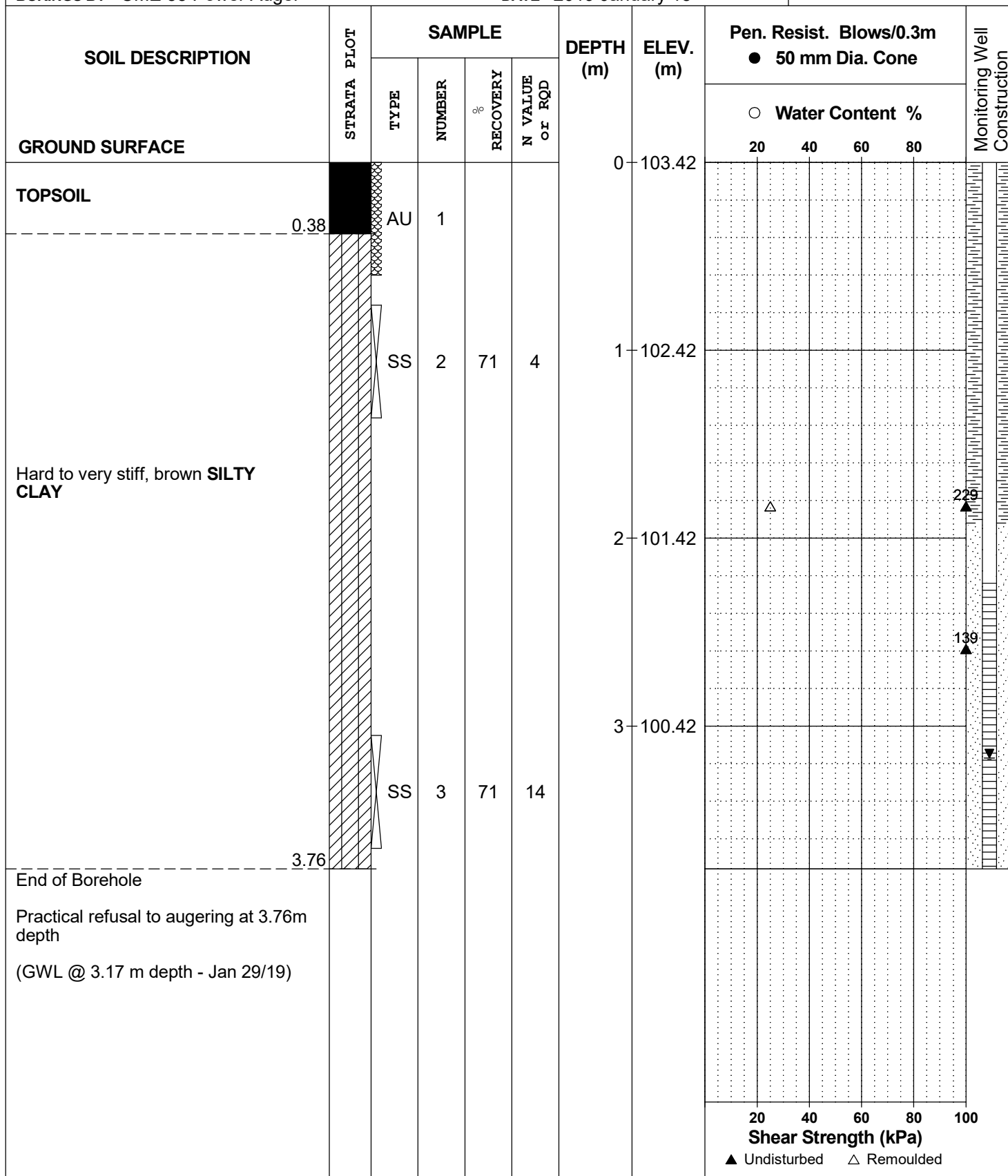
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REMARKS

HOLE NO. **BH 9**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

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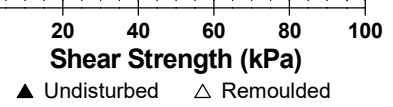
REMARKS

HOLE NO. **BH10**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	103.31						
TOPSOIL		AU	1										
	0.41												
Very stiff, brown SILTY CLAY		SS	2	67	9	1	102.31						
		SS	3	75	6								
- grey by 2.1m depth						2	101.31						
	3.05												
GLACIAL TILL: Compact, brown sandy silt, trace clay and gravel, occasional cobbles and boulders		SS	4	83	19								
	3.66					3	100.31						
End of Borehole													
Practical refusal to augering at 3.66m depth (GWL @ 2.18m depth - Jan 29/19)													



**DATUM** Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

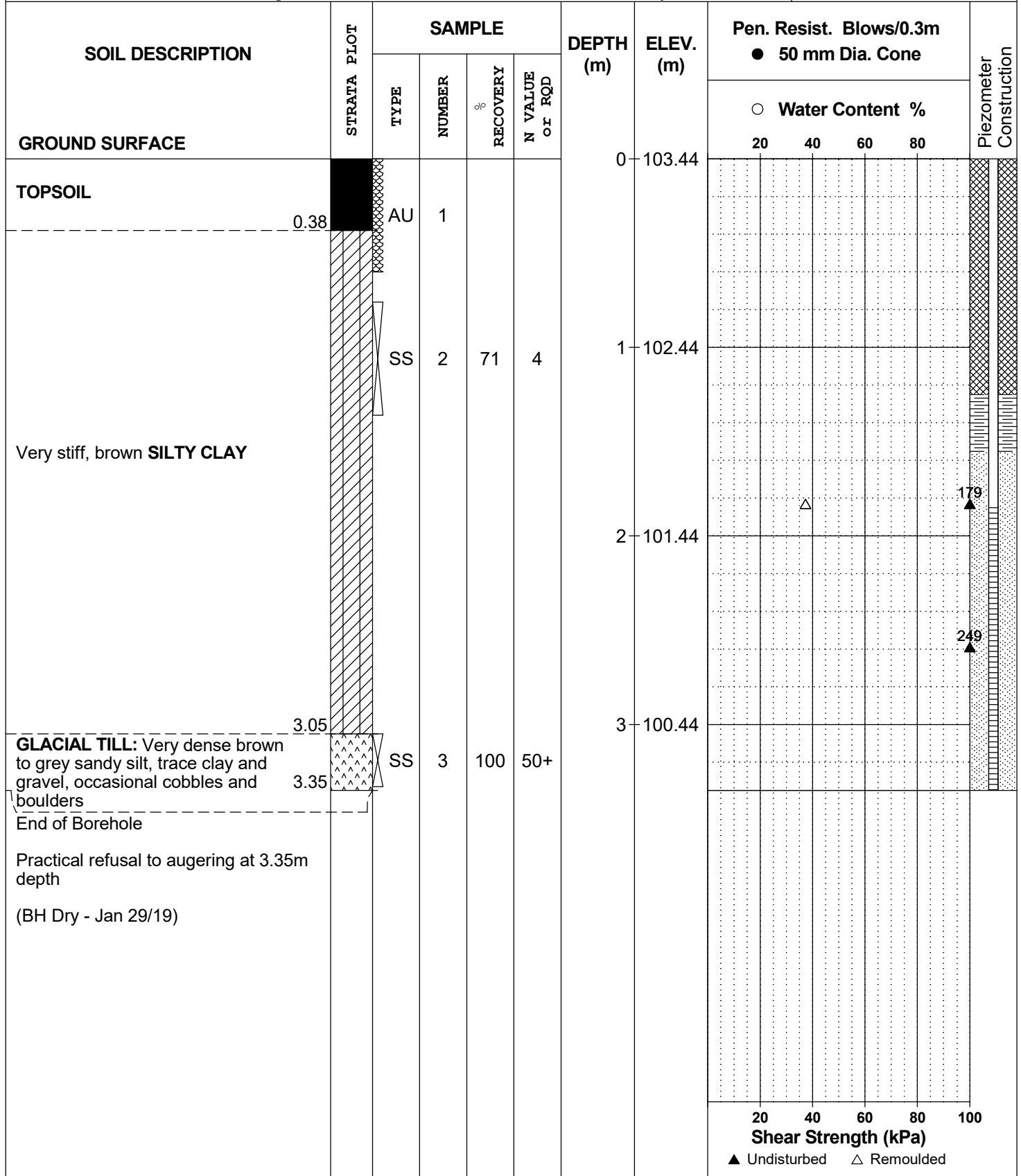
**FILE NO.**  
**PG4772**

**REMARKS**

**HOLE NO.**  
**BH11**

**BORINGS BY** CME 55 Power Auger

**DATE** 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

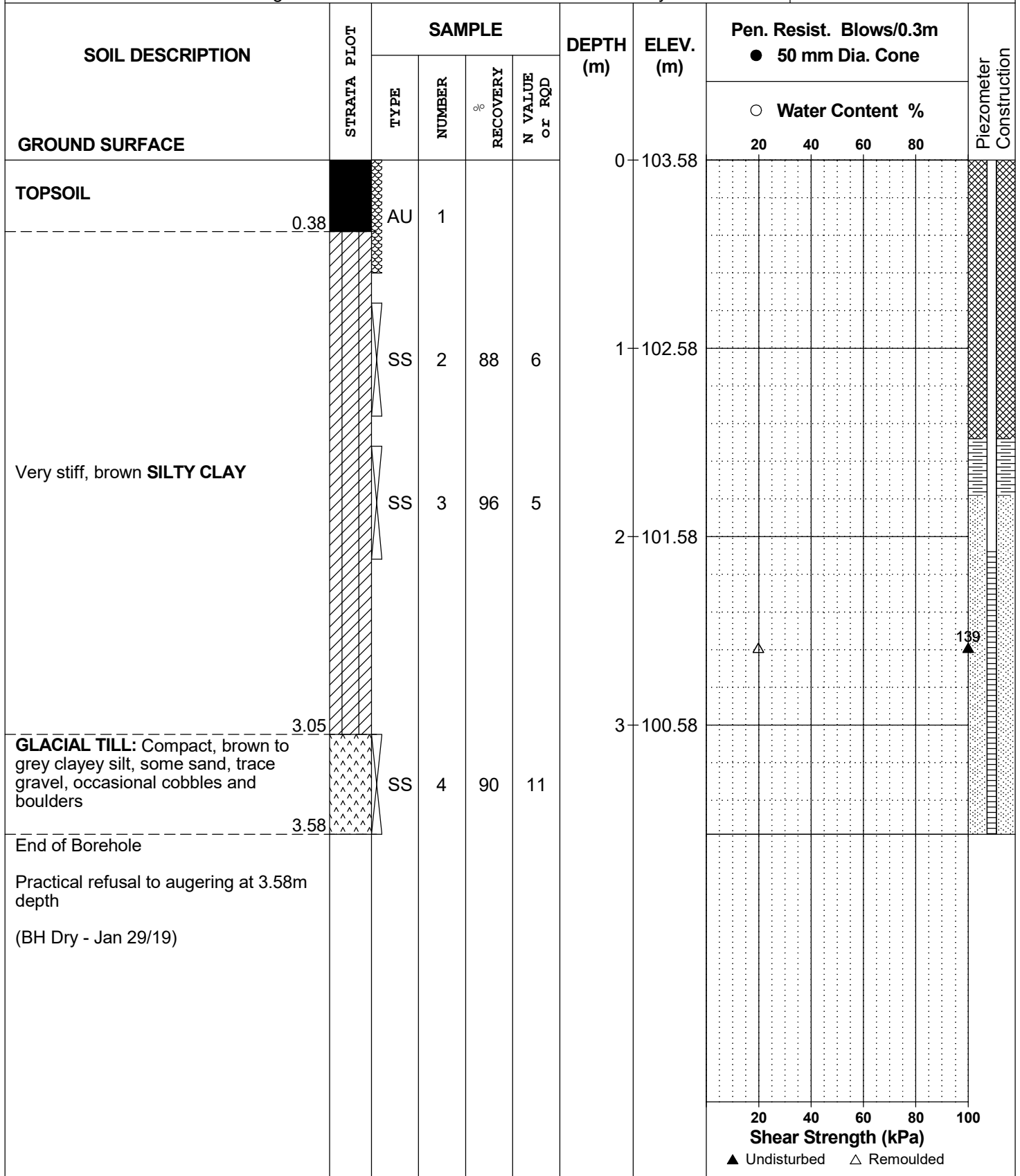
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REMARKS

HOLE NO. **BH12**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

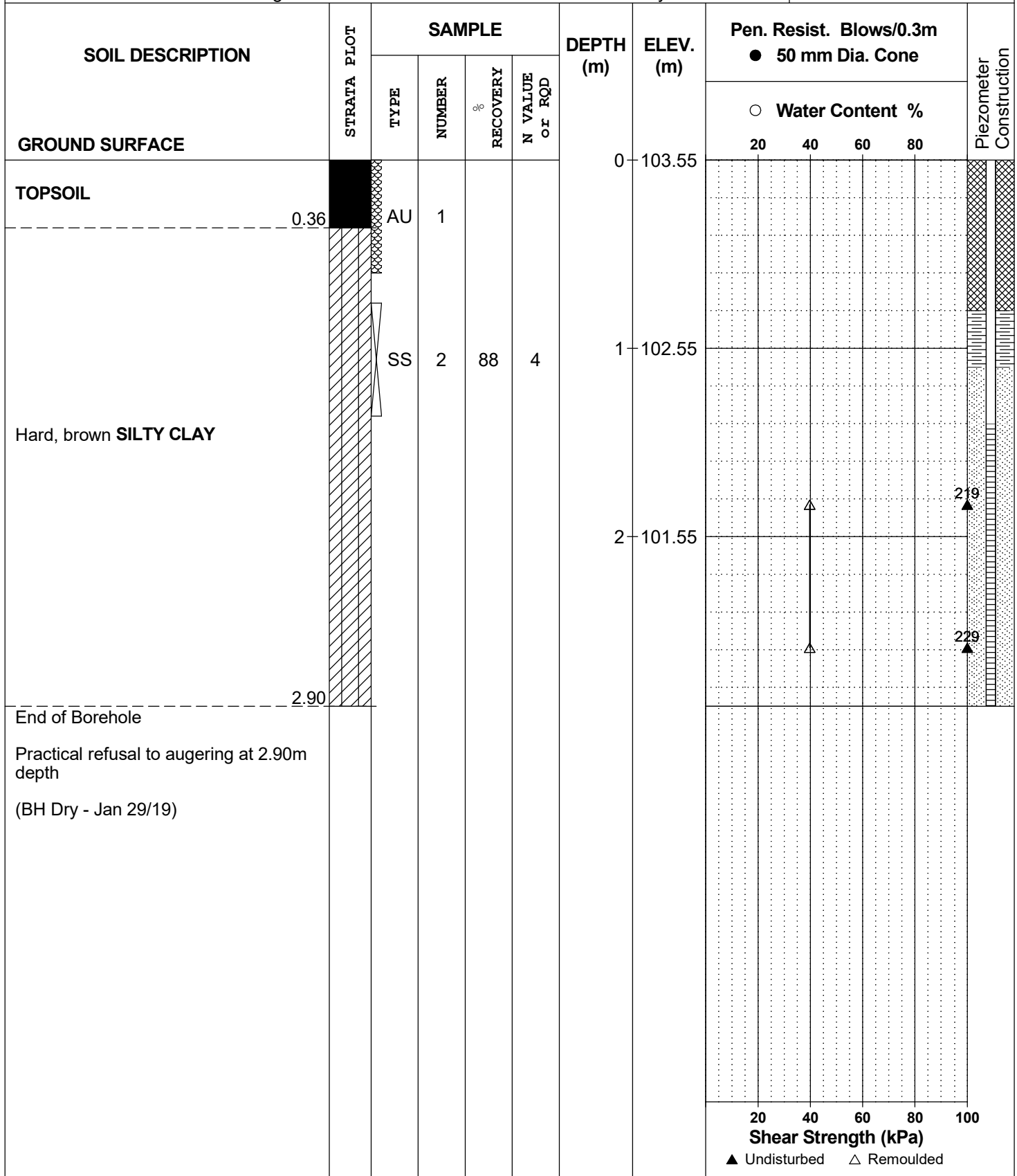
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH13**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15





**DATUM** Ground surface elevations provided by Annis, O'Sullivan, Vollebek Ltd.





**REMARKS**

**BORINGS BY** CME 55 Power Auger

**DATE** 2019 January 15

**FILE NO.**  
**PG4772**

**HOLE NO.**  
**BH14**

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction		
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80			
<b>GROUND SURFACE</b>						0	104.18							
<b>TOPSOIL</b>		AU	1											
	0.41													
Very stiff, brown <b>SILTY CLAY</b>		SS	2	67	7	1	103.18							
- grey by 2.0m depth		SS	3	96	6	2	102.18							
	2.29													
<b>GLACIAL TILL:</b> Grey silty clay, trace sand and gravel, occasional cobbles and boulders						3	101.18							
	3.00													
End of Borehole														
Practical refusal to augering at 3.00m depth (BH Dry - Jan 29/19)														

20 40 60 80 100

**Shear Strength (kPa)**

▲ Undisturbed    △ Remoulded

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

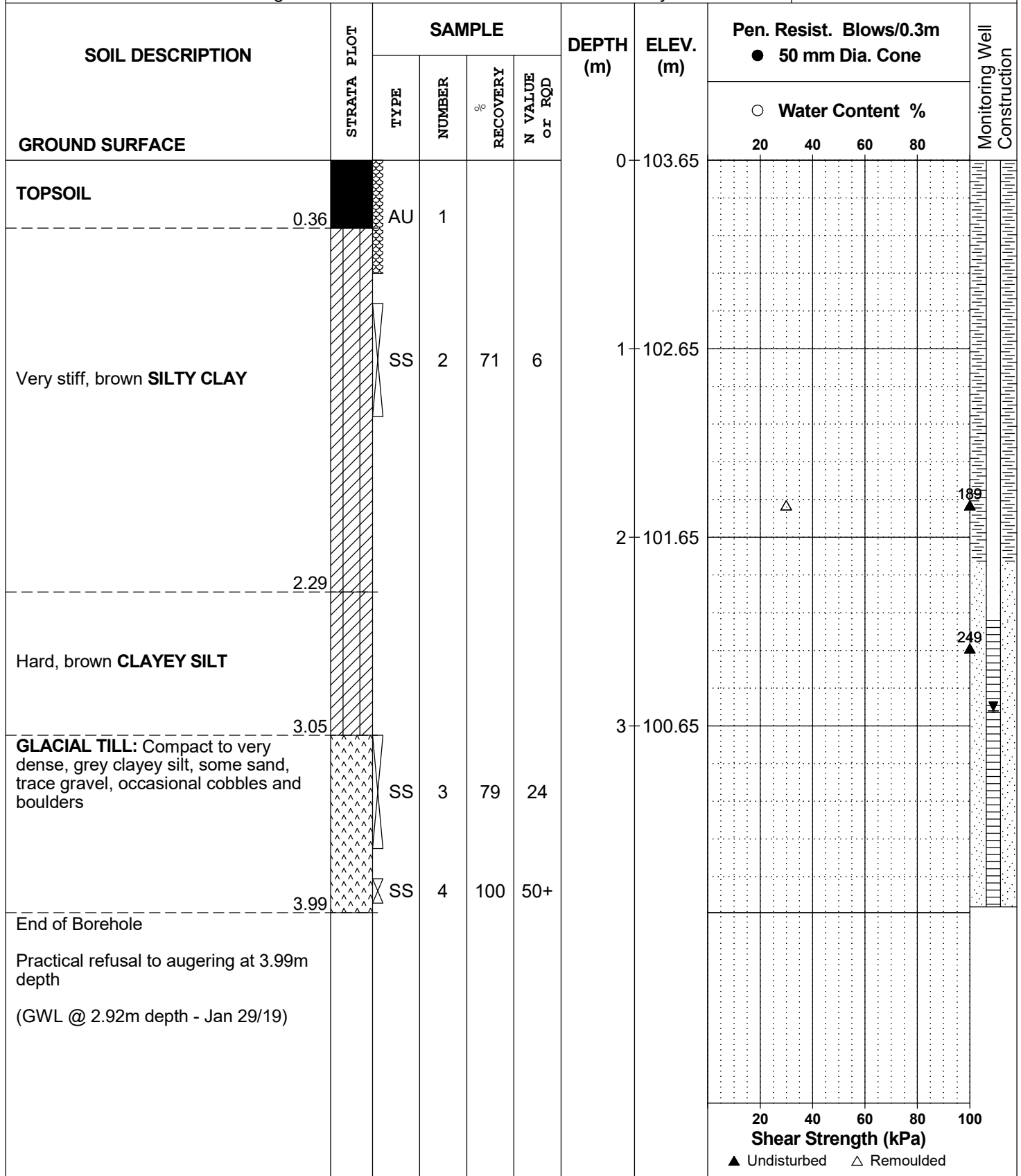
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REMARKS

HOLE NO. **BH15**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

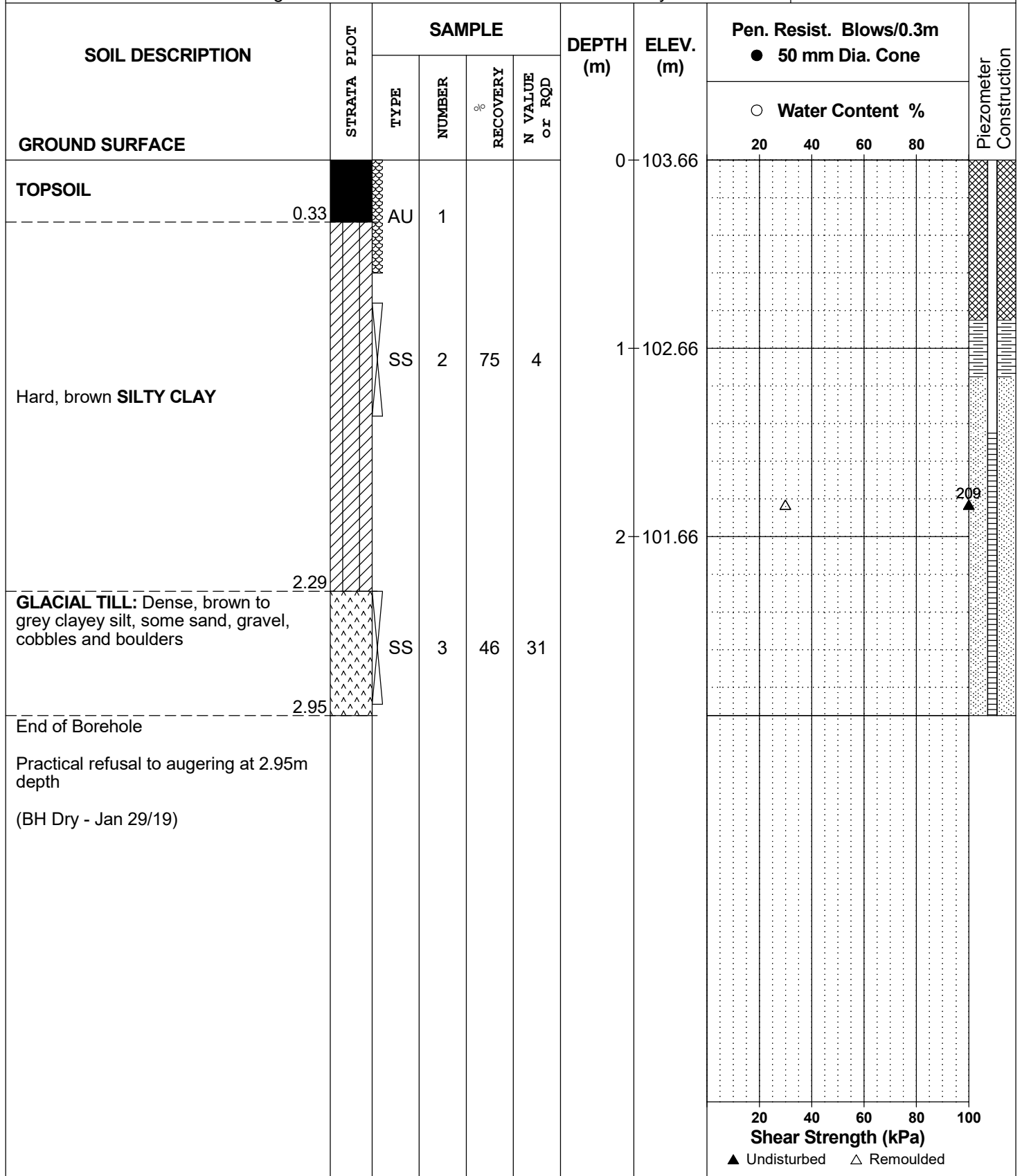
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH16**

BORINGS BY CME 55 Power Auger

DATE 2019 January 15



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH17**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %			N VALUE or RQD	20	40	60		80
GROUND SURFACE					0	104.19						
TOPSOIL		AU	1									
Very stiff to hard, brown <b>CLAYEY SILT</b>		SS	2	79	7	1	103.19					
- grey by 1.8m depth		SS	3	100	55	2	102.19					
End of Borehole Practical refusal to augering at 2.23m depth (BH Dry - Jan 29/19)												

○ Water Content %

20 40 60 80 100  
Shear Strength (kPa)

▲ Undisturbed    △ Remoulded

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH18**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	104.15						
TOPSOIL	0.33	AU	1										
Hard, brown <b>CLAYEY SILT</b>	1.96	SS	2	88	11	1	103.15						
- grey by 1.8m depth		SS	3	88	50+								
End of Borehole Practical refusal to augering at 1.96m depth (BH Dry - Jan 29/19)													

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

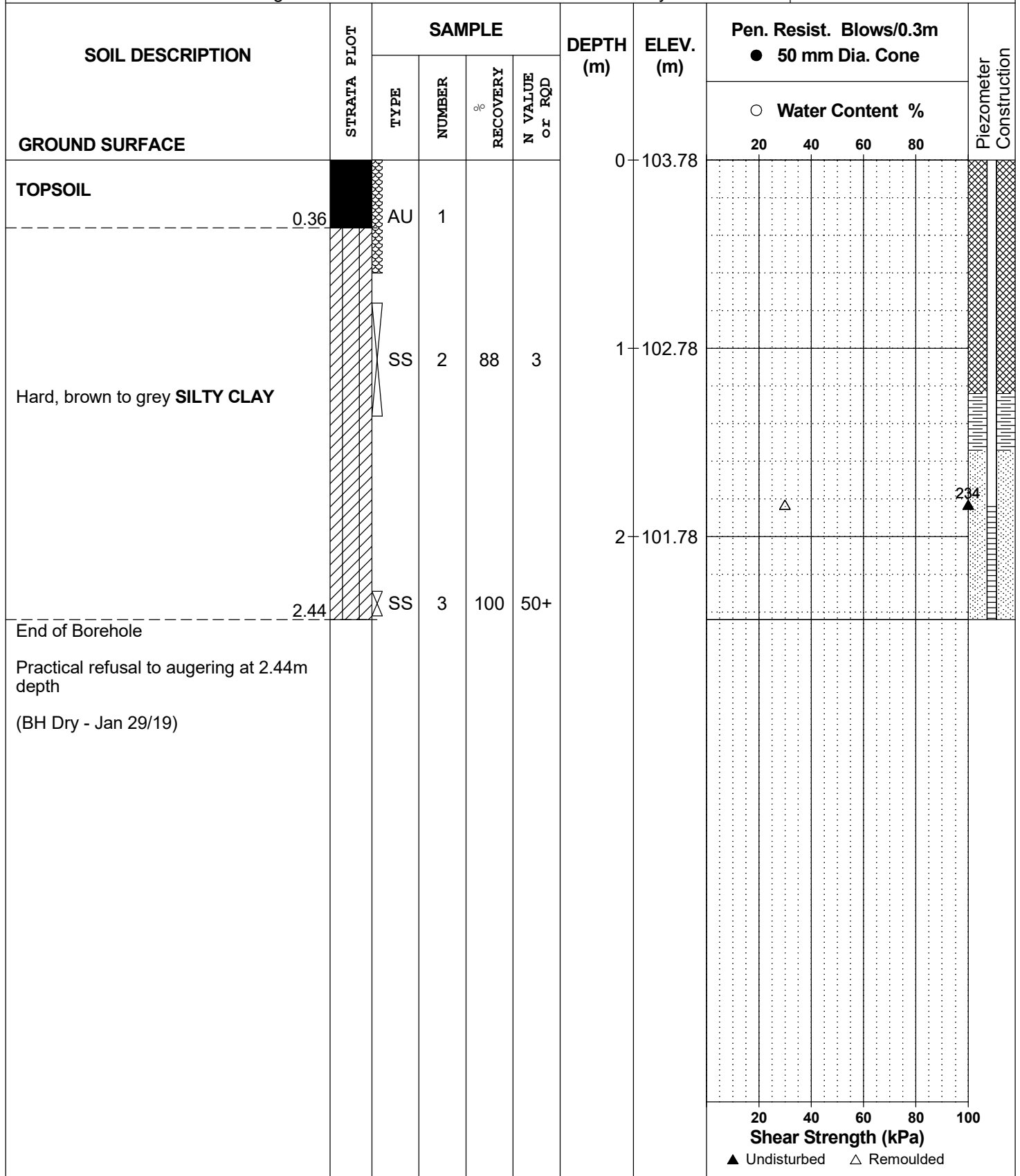
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH19**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

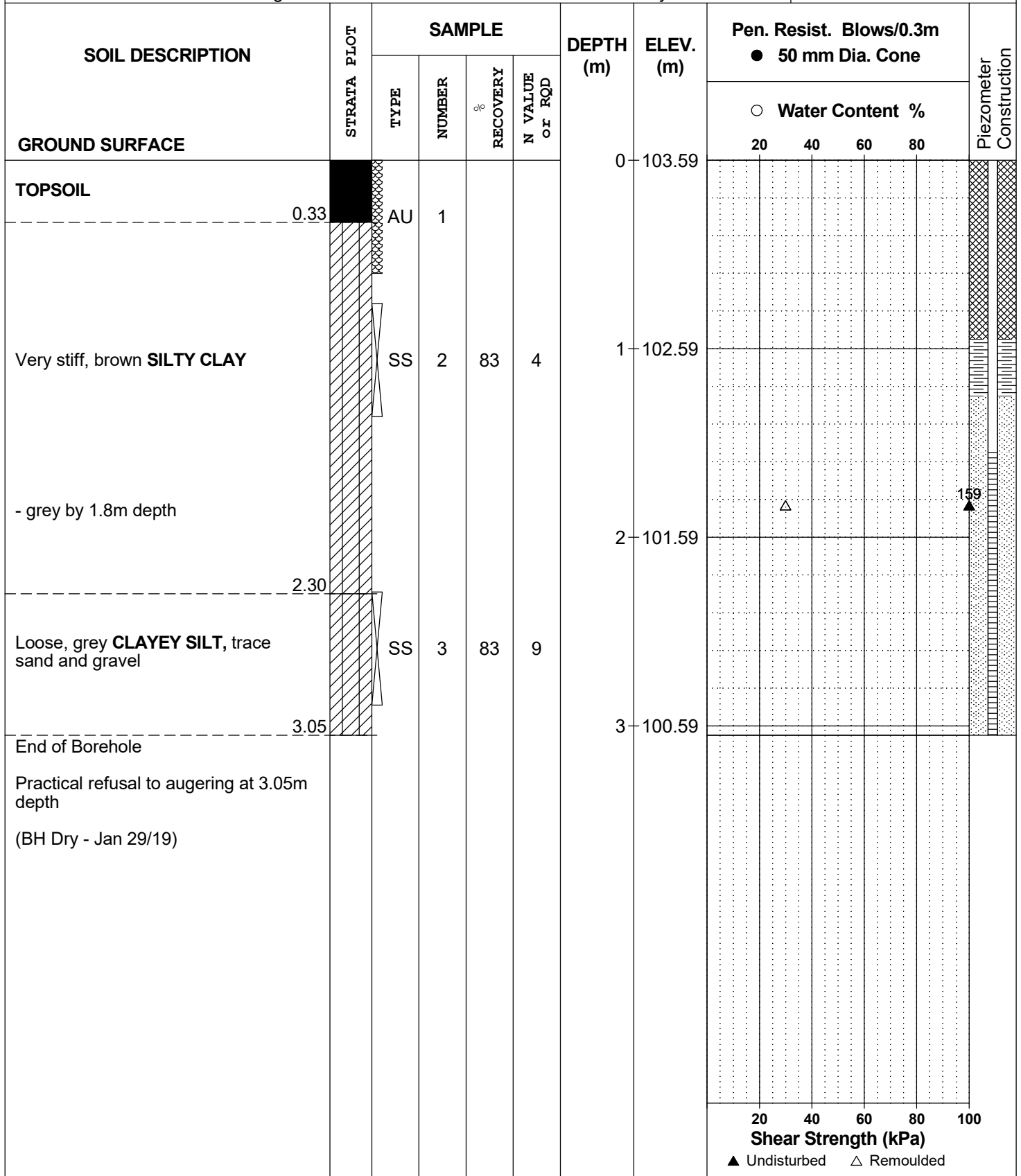
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH20**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

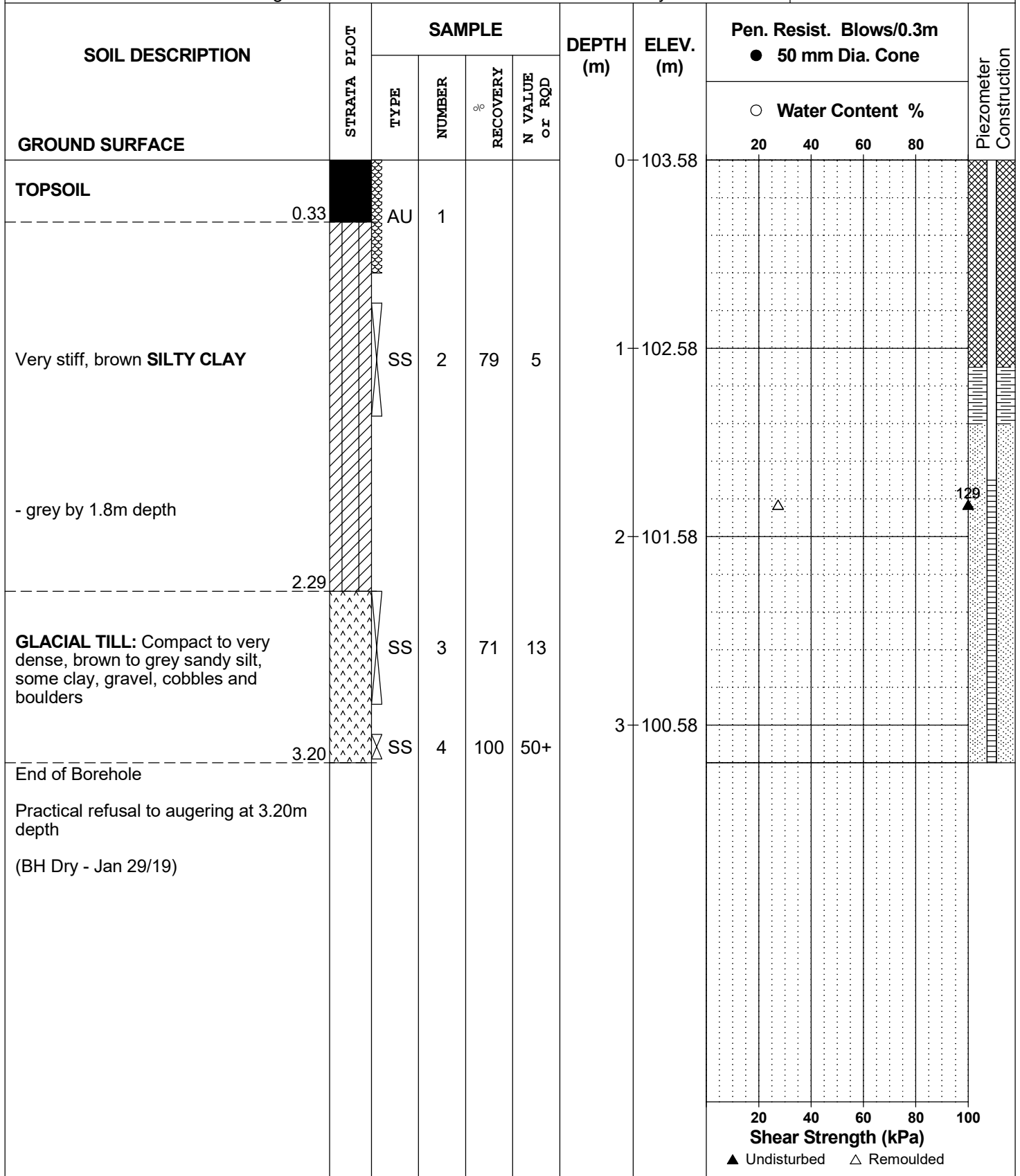
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH21**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16





DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

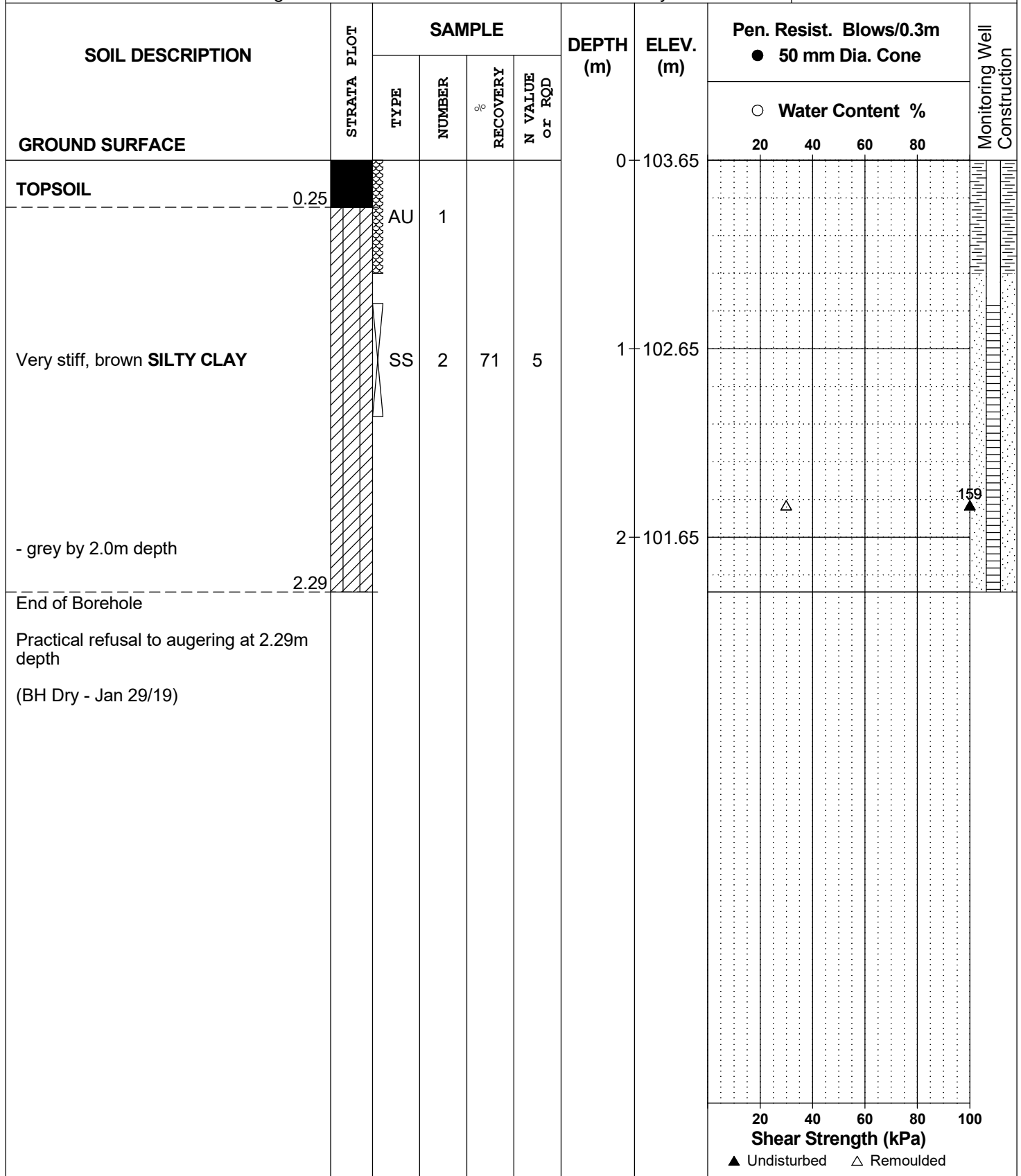
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH22**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16







DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebekk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH25**

BORINGS BY CME 55 Power Auger

DATE 2019 January 16

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %			N VALUE or RQD	20	40	60		80
GROUND SURFACE					0	104.07						
TOPSOIL		AU	1									
Very stiff, brown <b>CLAYEY SILT</b>		SS	2	75	11	1	103.07					
<b>GLACIAL TILL:</b> Very dense, grey clayey silt with sand, gravel, cobbles, boulders		SS	3	75	50+							
End of Borehole  Practical refusal to augering at 1.62m depth (GWL @ 1.68m depth - Jan 29/19)												

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

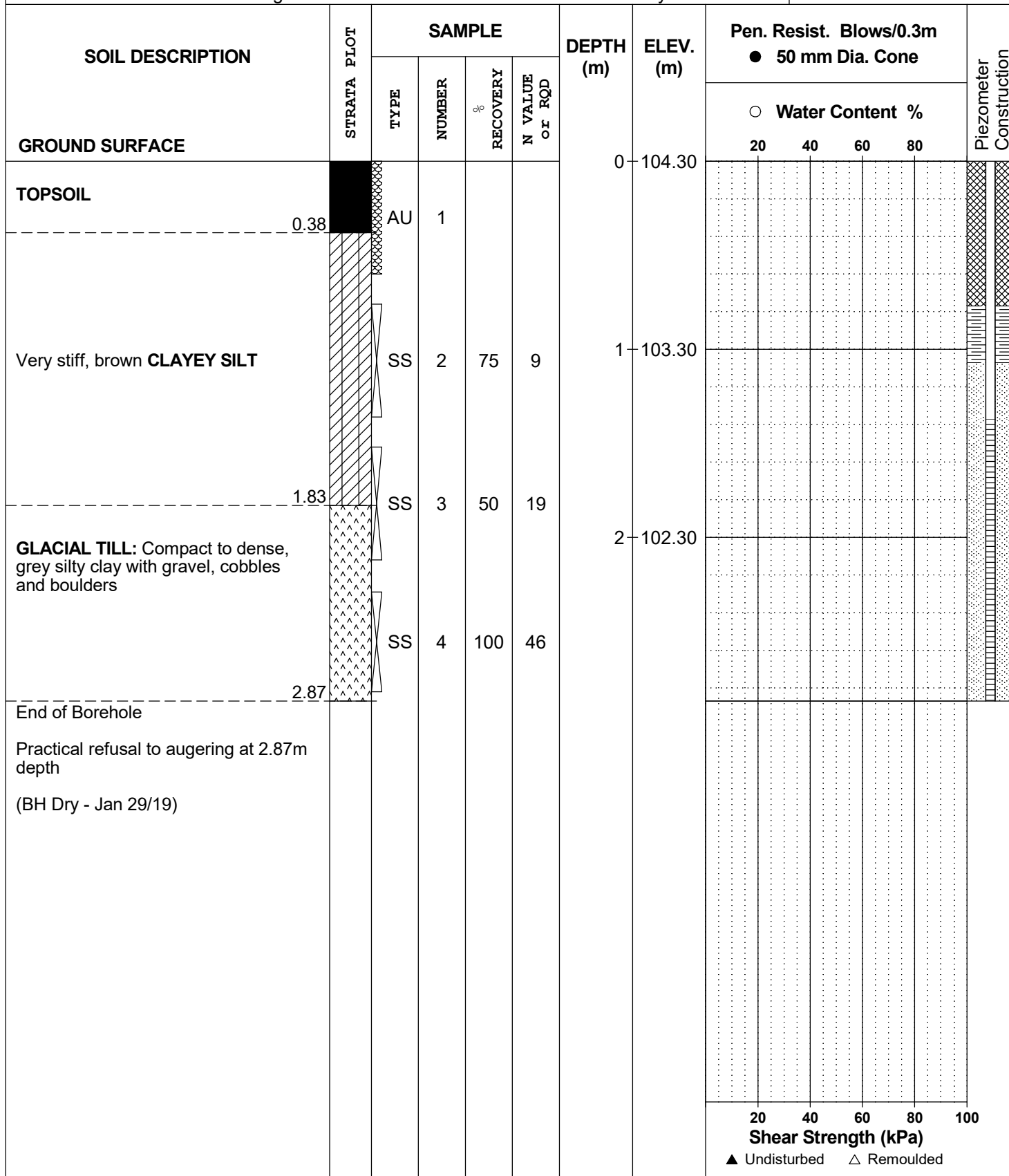
FILE NO. **PG4772**

REMARKS

HOLE NO. **BH26**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH27**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Monitoring Well Construction	
		TYPE	NUMBER	RECOVERY %	N VALUE or RQD			20	40	60	80		
GROUND SURFACE						0	103.97						
TOPSOIL	0.33	AU	1										
Very stiff, brown <b>CLAYEY SILT</b>	1.7	SS	2	71	8	1	102.97						
- grey by 1.7m depth	1.93	SS	3	88	50+								
End of Borehole													
Practical refusal to augering at 1.93m depth (BH Dry - Jan 29/19)													

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

DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

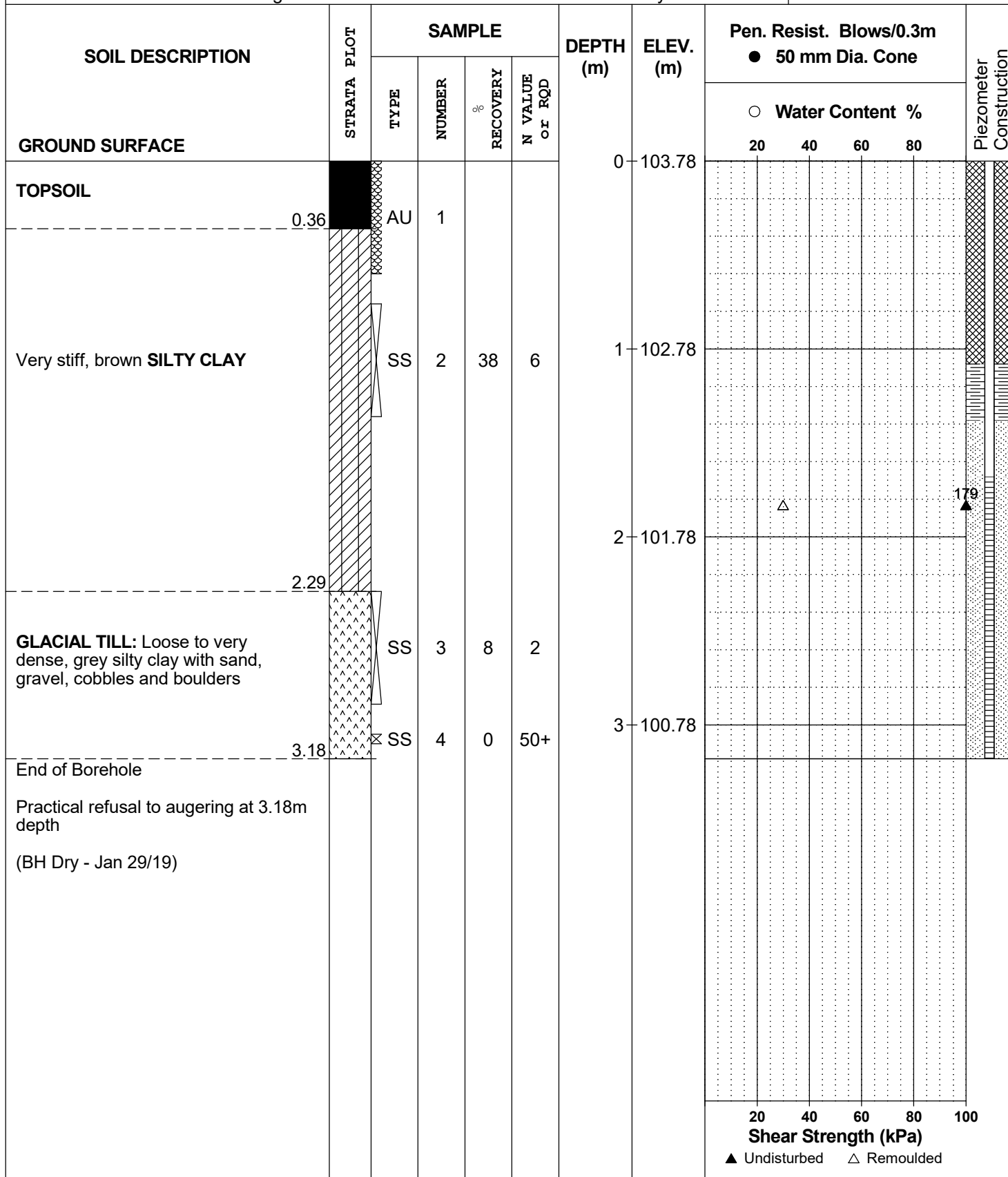
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REMARKS

HOLE NO. **BH28**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17



DATUM Ground surface elevations provided by Annis, O'Sullivan, Vollebakk Ltd.

FILE NO. **PG4772**

REMARKS

HOLE NO. **BH29**

BORINGS BY CME 55 Power Auger

DATE 2019 January 17

SOIL DESCRIPTION	STRATA PLOT	SAMPLE			DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone				Piezometer Construction	
		TYPE	NUMBER	RECOVERY %			N VALUE or RQD	20	40	60		80
GROUND SURFACE					0	103.71						
TOPSOIL	0.38	AU	1									
Very stiff, brown <b>SILTY CLAY</b>		SS	2	50	7	1	102.71					
		SS	3	71	4	2	101.71					
<b>GLACIAL TILL:</b> Loose, grey silty clay with sand, gravel, cobbles and boulders	2.29	SS	4	17	7							
End of Borehole	2.95											
Practical refusal to augering at 2.95m depth (BH Dry - Jan 29/19)												
							20	40	60	80	100	
							<b>Shear Strength (kPa)</b>					
							▲ Undisturbed    △ Remoulded					



# SYMBOLS AND TERMS

## SOIL DESCRIPTION

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

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

The standard terminology to describe the relative strength of cohesionless soils is the compactness condition, usually inferred from the results of the Standard Penetration Test (SPT) 'N' value. The SPT N value is the number of blows of a 63.5 kg hammer, falling 760 mm, required to drive a 51 mm O.D. split spoon sampler 300 mm into the soil after an initial penetration of 150 mm. An SPT N value of "P" denotes that the split-spoon sampler was pushed 300 mm into the soil without the use of a falling hammer.

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

The standard terminology to describe the strength of cohesive soils is the consistency, which is based on the undisturbed undrained shear strength as measured by the in situ or laboratory shear vane tests, unconfined compression tests, or occasionally by the Standard Penetration Test (SPT). Note that the typical correlations of undrained shear strength to SPT N value (tabulated below) tend to underestimate the consistency for sensitive silty clays, so Paterson reviews the applicable split spoon samples in the laboratory to provide a more representative consistency value based on tactile examination.

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

## SYMBOLS AND TERMS (continued)

### SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their “sensitivity”. The sensitivity,  $S_t$ , is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil. The classes of sensitivity may be defined as follows:

Low Sensitivity:	$S_t < 2$
Medium Sensitivity:	$2 < S_t < 4$
Sensitive:	$4 < S_t < 8$
Extra Sensitive:	$8 < S_t < 16$
Quick Clay:	$S_t > 16$

### ROCK DESCRIPTION

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

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

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

### SAMPLE TYPES

SS	-	Split spoon sample (obtained in conjunction with the performing of the Standard Penetration Test (SPT))
TW	-	Thin wall tube or Shelby tube, generally recovered using a piston sampler
G	-	"Grab" sample from test pit or surface materials
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size BQ, NQ, HQ, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

## SYMBOLS AND TERMS (continued)

### PLASTICITY LIMITS AND GRAIN SIZE DISTRIBUTION

WC%	-	Natural water content or water content of sample, %
LL	-	Liquid Limit, % (water content above which soil behaves as a liquid)
PL	-	Plastic Limit, % (water content above which soil behaves plastically)
PI	-	Plasticity Index, % (difference between LL and PL)
D <sub>xx</sub>	-	Grain size at which xx% of the soil, by weight, is of finer grain sizes These grain size descriptions are not used below 0.075 mm grain size
D <sub>10</sub>	-	Grain size at which 10% of the soil is finer (effective grain size)
D <sub>60</sub>	-	Grain size at which 60% of the soil is finer
C <sub>c</sub>	-	Concavity coefficient = $(D_{30})^2 / (D_{10} \times D_{60})$
C <sub>u</sub>	-	Uniformity coefficient = $D_{60} / D_{10}$

C<sub>c</sub> and C<sub>u</sub> are used to assess the grading of sands and gravels:

Well-graded gravels have:  $1 < C_c < 3$  and  $C_u > 4$

Well-graded sands have:  $1 < C_c < 3$  and  $C_u > 6$

Sands and gravels not meeting the above requirements are poorly-graded or uniformly-graded.

C<sub>c</sub> and C<sub>u</sub> are not applicable for the description of soils with more than 10% silt and clay (more than 10% finer than 0.075 mm or the #200 sieve)

### CONSOLIDATION TEST

p' <sub>o</sub>	-	Present effective overburden pressure at sample depth
p' <sub>c</sub>	-	Preconsolidation pressure of (maximum past pressure on) sample
C <sub>cr</sub>	-	Recompression index (in effect at pressures below p' <sub>c</sub> )
C <sub>c</sub>	-	Compression index (in effect at pressures above p' <sub>c</sub> )
OC Ratio		Overconsolidation ratio = $p'_c / p'_o$
Void Ratio		Initial sample void ratio = volume of voids / volume of solids
W <sub>o</sub>	-	Initial water content (at start of consolidation test)

### PERMEABILITY TEST

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

### STRATA PLOT



Topsoil



Asphalt



Fill



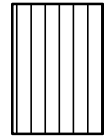
Peat



Sand



Silty Sand



Silt



Sandy Silt



Clay



Silty Clay



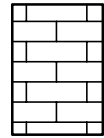
Clayey Silty Sand



Glacial Till



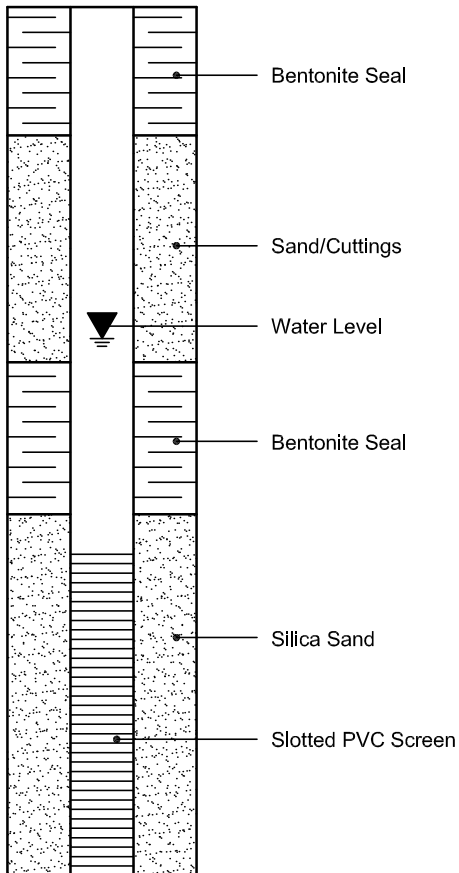
Shale



Bedrock

### MONITORING WELL AND PIEZOMETER CONSTRUCTION

#### MONITORING WELL CONSTRUCTION



#### PIEZOMETER CONSTRUCTION



Certificate of Analysis  
 Client: Paterson Group Consulting Engineers  
 Client PO: 25648

Report Date: 22-Jan-2019

Order Date: 16-Jan-2019

**Project Description: PG4772**

<b>Client ID:</b>	BH#16-19 SS#3	-	-	-
<b>Sample Date:</b>	01/15/2019 09:00	-	-	-
<b>Sample ID:</b>	1903309-01	-	-	-
<b>MDL/Units</b>	Soil	-	-	-

**Physical Characteristics**

% Solids	0.1 % by Wt.	85.8	-	-	-
----------	--------------	------	---	---	---

**General Inorganics**

pH	0.05 pH Units	7.80	-	-	-
Resistivity	0.10 Ohm.m	76.2	-	-	-

**Anions**

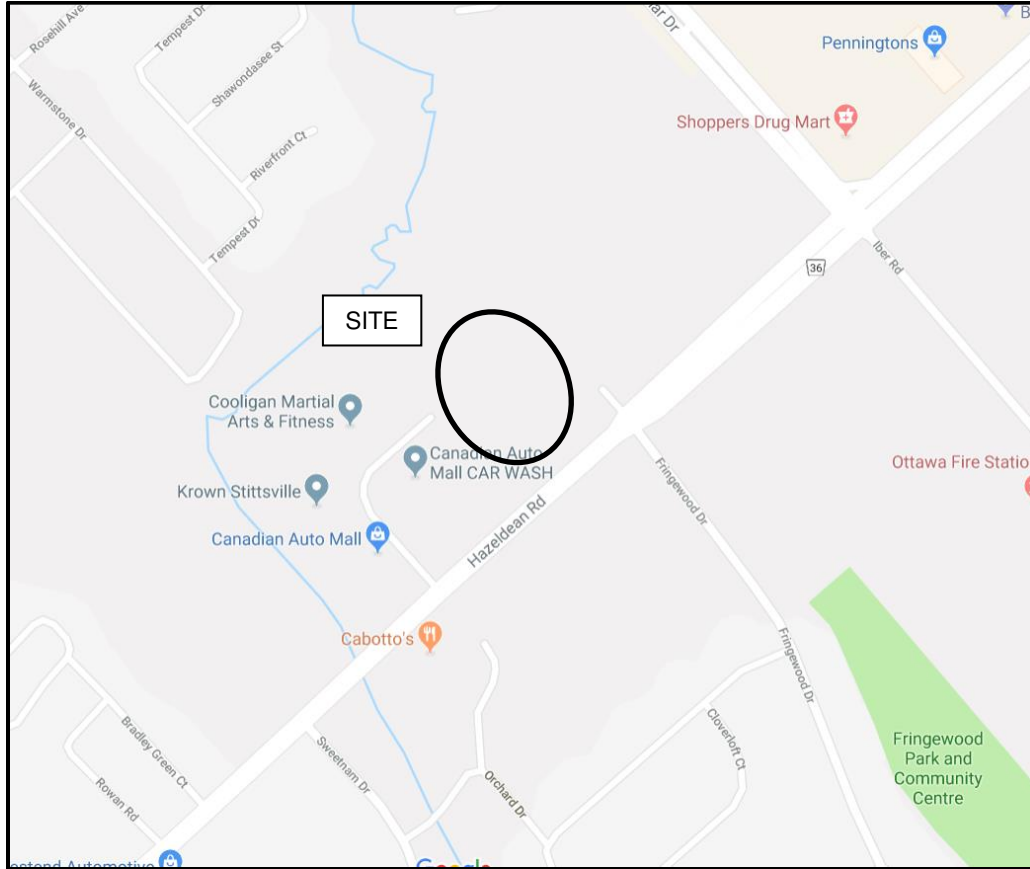
Chloride	5 ug/g dry	6	-	-	-
Sulphate	5 ug/g dry	6	-	-	-

# **APPENDIX 2**

**FIGURE 1 - KEY PLAN**

**FIGURES 2 TO 4 - SLOPE STABILITY ANALYSIS SECTIONS**

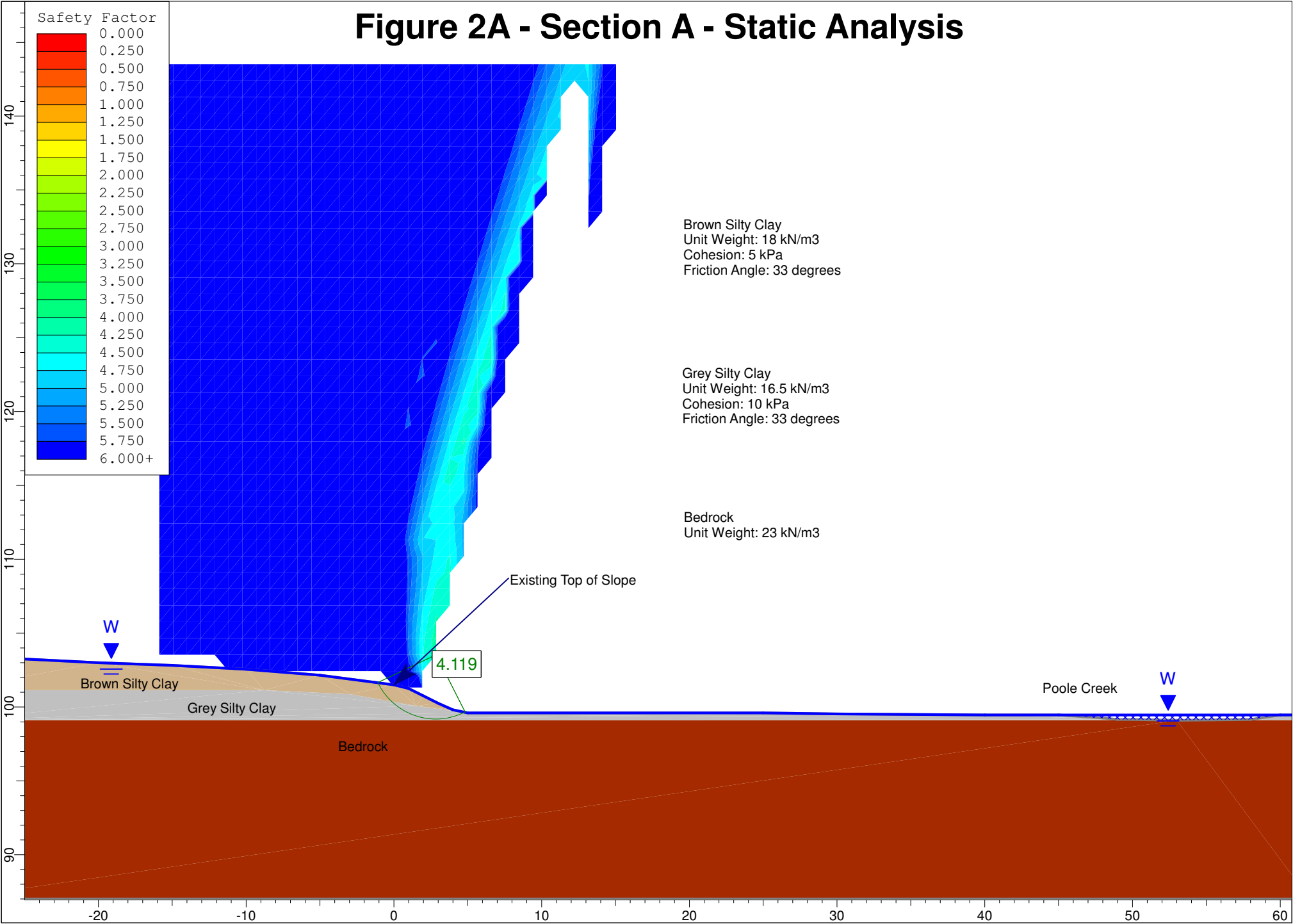
**DRAWING PG4772-1 - TEST HOLE LOCATION PLAN**



# FIGURE 1

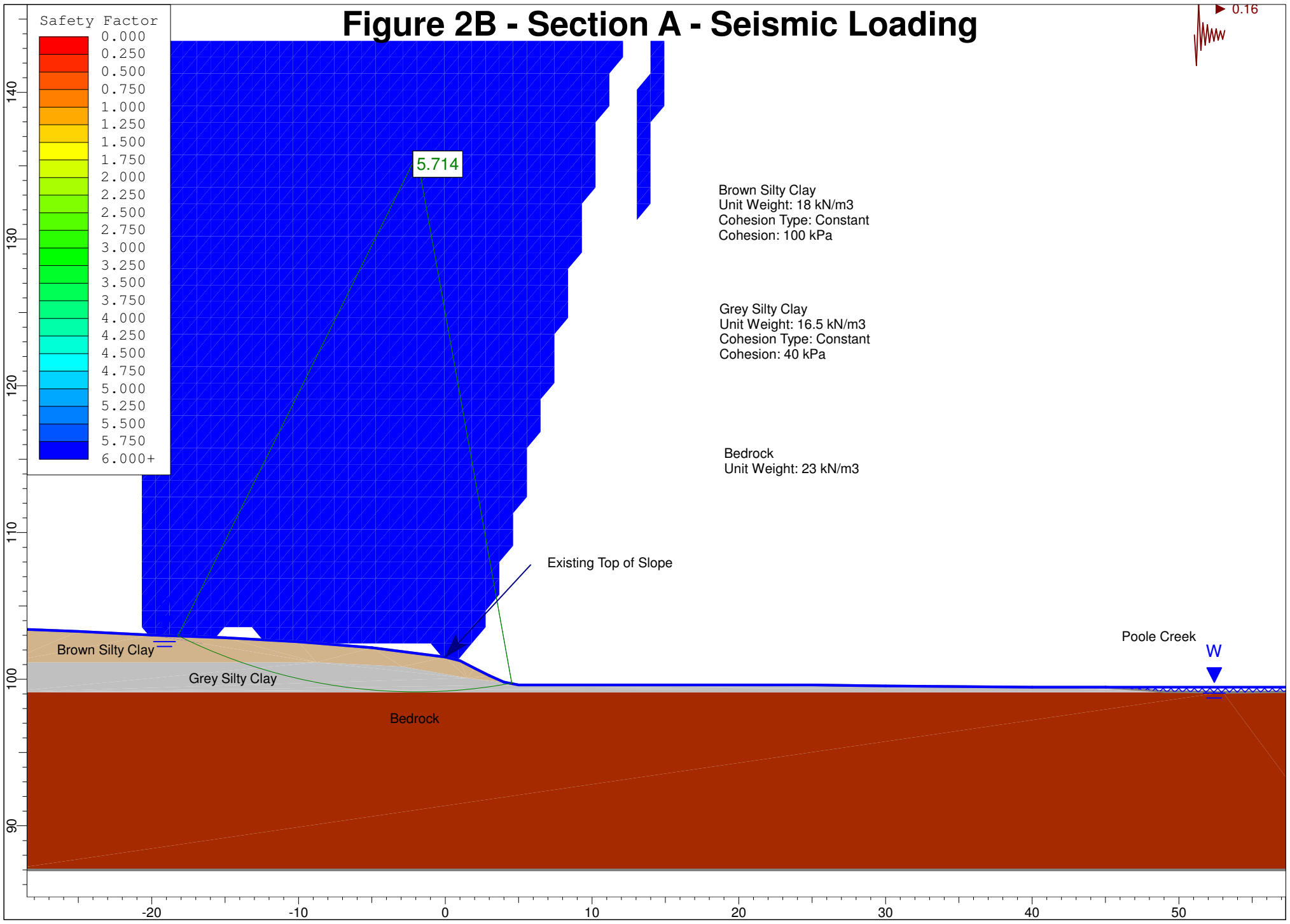
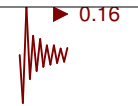
## KEY PLAN

# Figure 2A - Section A - Static Analysis

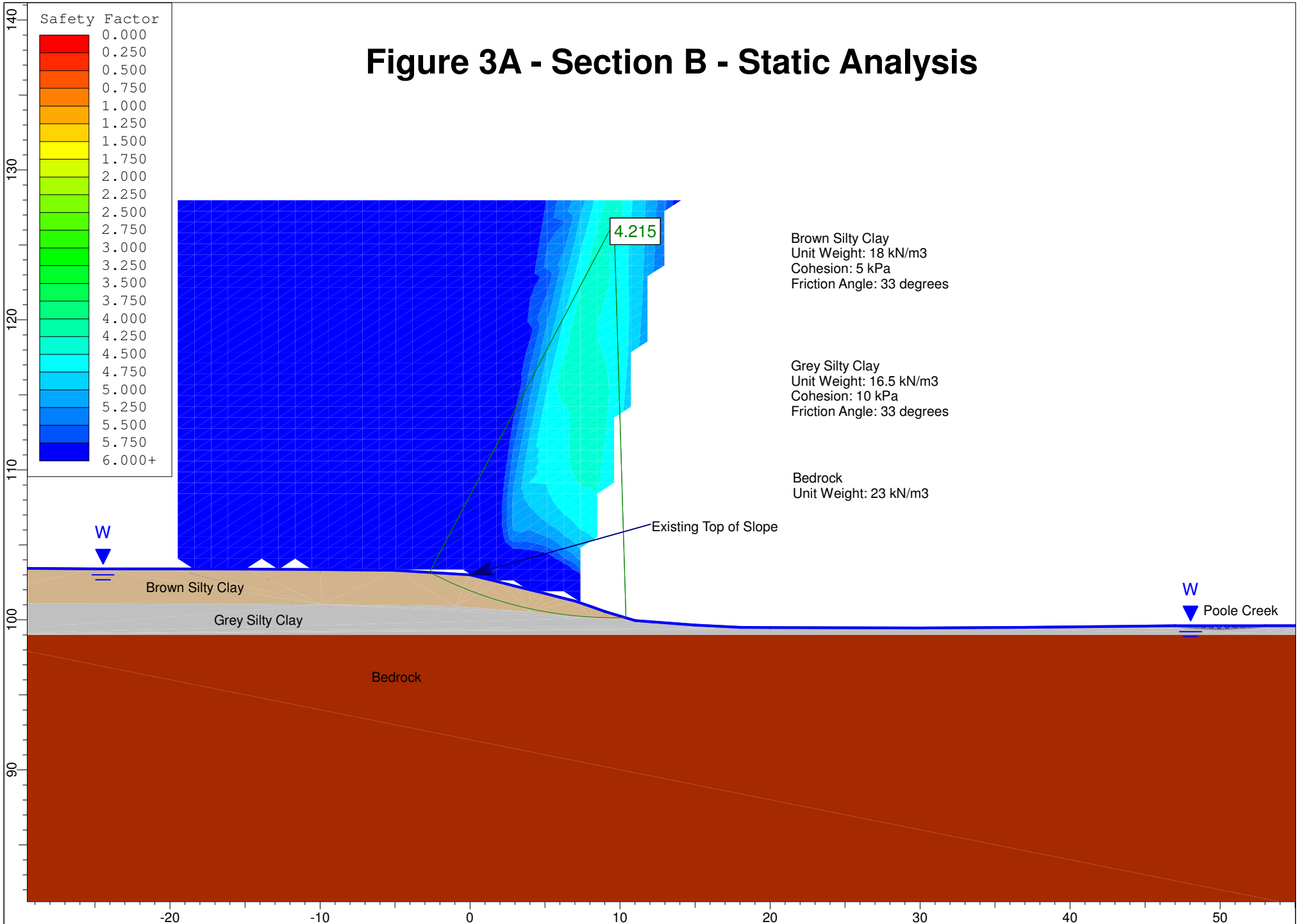




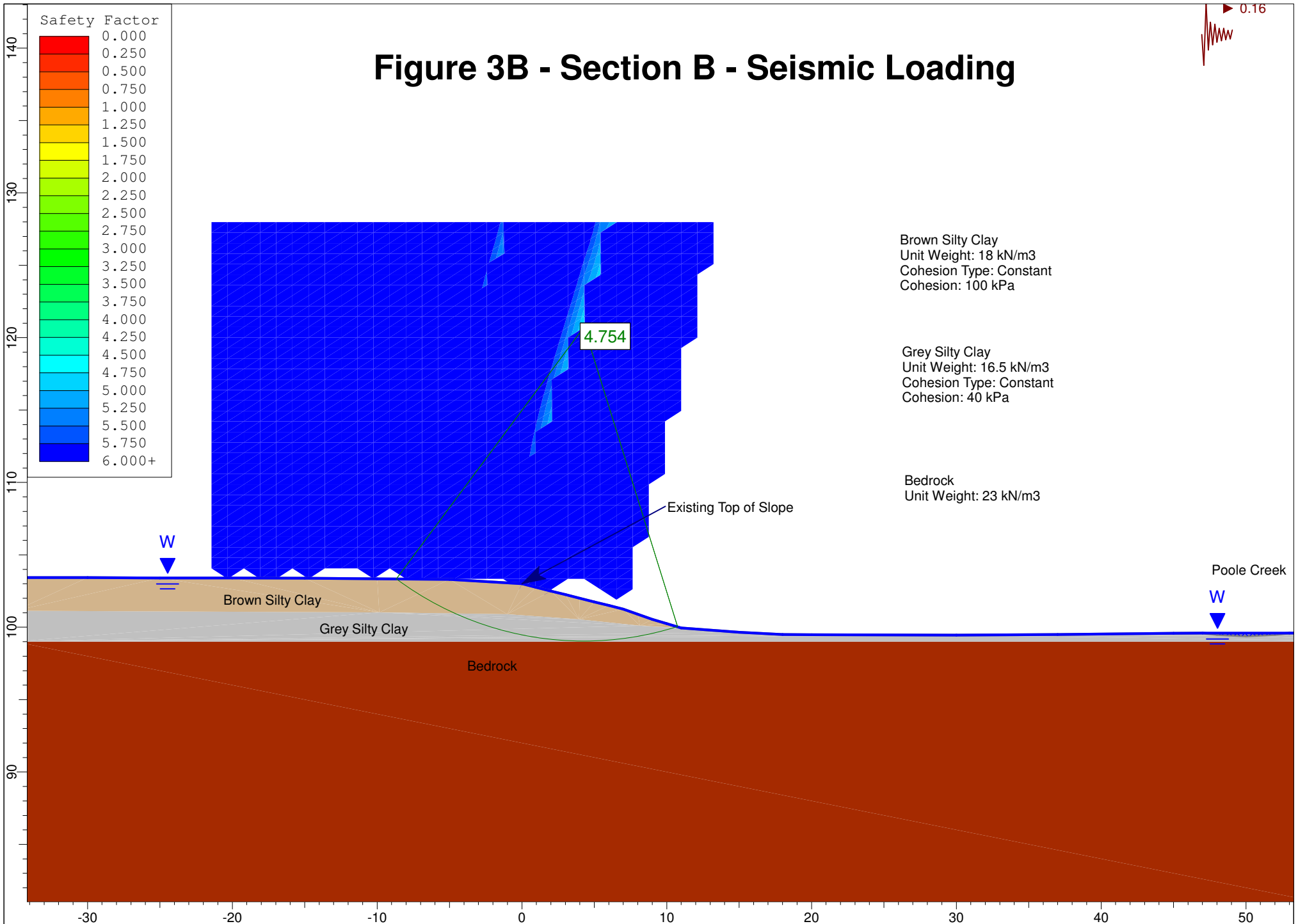
# Figure 2B - Section A - Seismic Loading



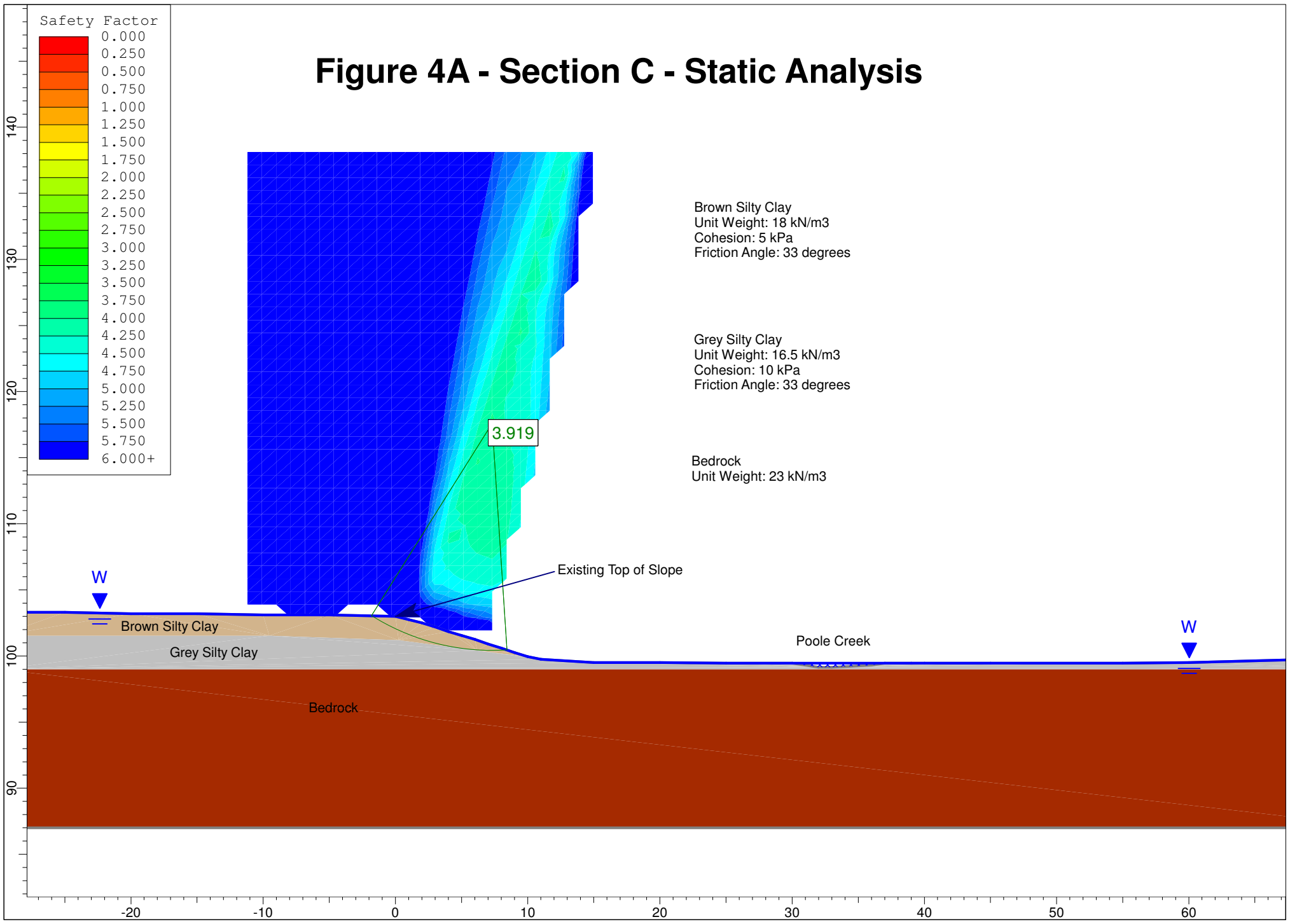
# Figure 3A - Section B - Static Analysis



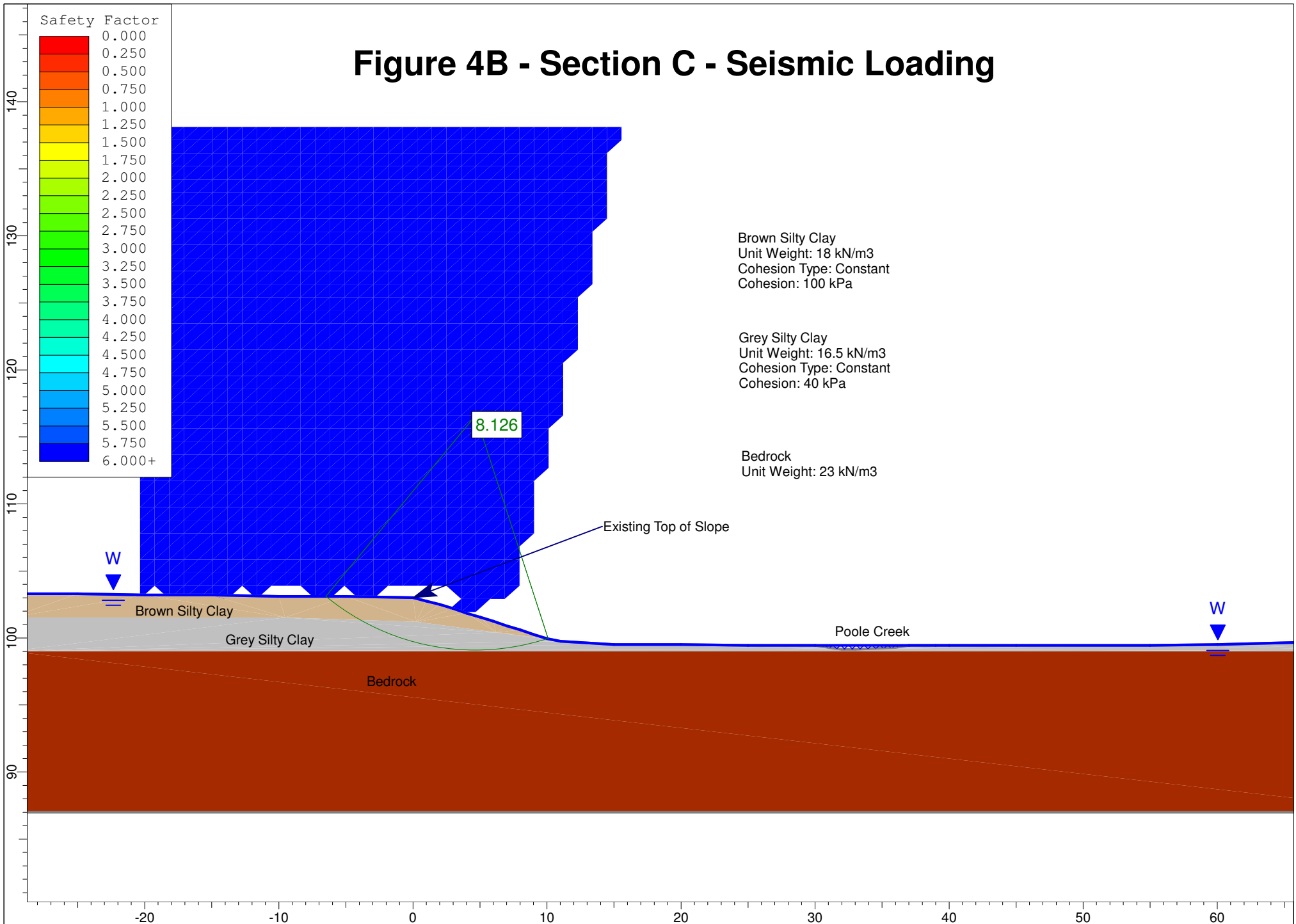
# Figure 3B - Section B - Seismic Loading

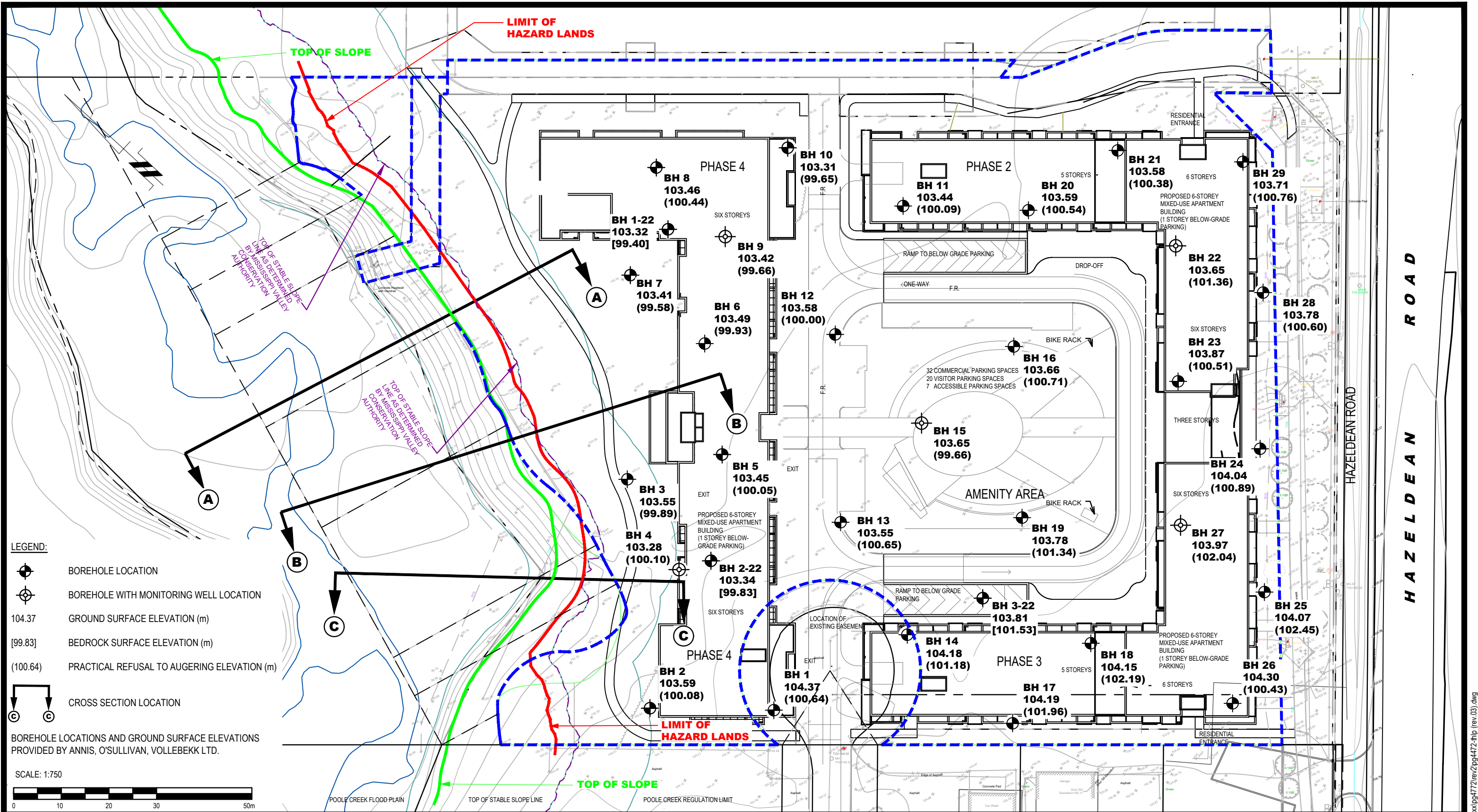


# Figure 4A - Section C - Static Analysis



# Figure 4B - Section C - Seismic Loading





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Tel: (613) 226-7381 Fax: (613) 226-6344

NO.	REVISIONS	DATE	INITIAL
3	2022 BOREHOLES ADDED TO PLAN	03/02/2022	OC
2	REVISED SITE PLAN	22/09/2020	JV
1	LIMIT OF HAZARD LANDS REVISED	28/02/2019	FA

NAUTICAL LANDS GROUP  
GEOTECHNICAL INVESTIGATION  
PROP. MIXED-USE DEVELOPMENT - WELLINGS OF STITTSVILLE PHASE 2 - 20 CEDAROW CT.  
OTTAWA, ONTARIO

Title: **TEST HOLE LOCATION PLAN**

Scale:	1:750	Date:	01/2019
Drawn by:	MPG	Report No.:	PG4772-1
Checked by:	OC	Dwg. No.:	<b>PG4772-1</b>
Approved by:	FA	Revision No.:	3

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**SERVICING AND STORMWATER MANAGEMENT BRIEF –  
WELLINGS OF STITTSVILLE PHASE 2, 20 CEDAROW COURT**

Appendix E Drawings  
March 29, 2022

## **Appendix E DRAWINGS**