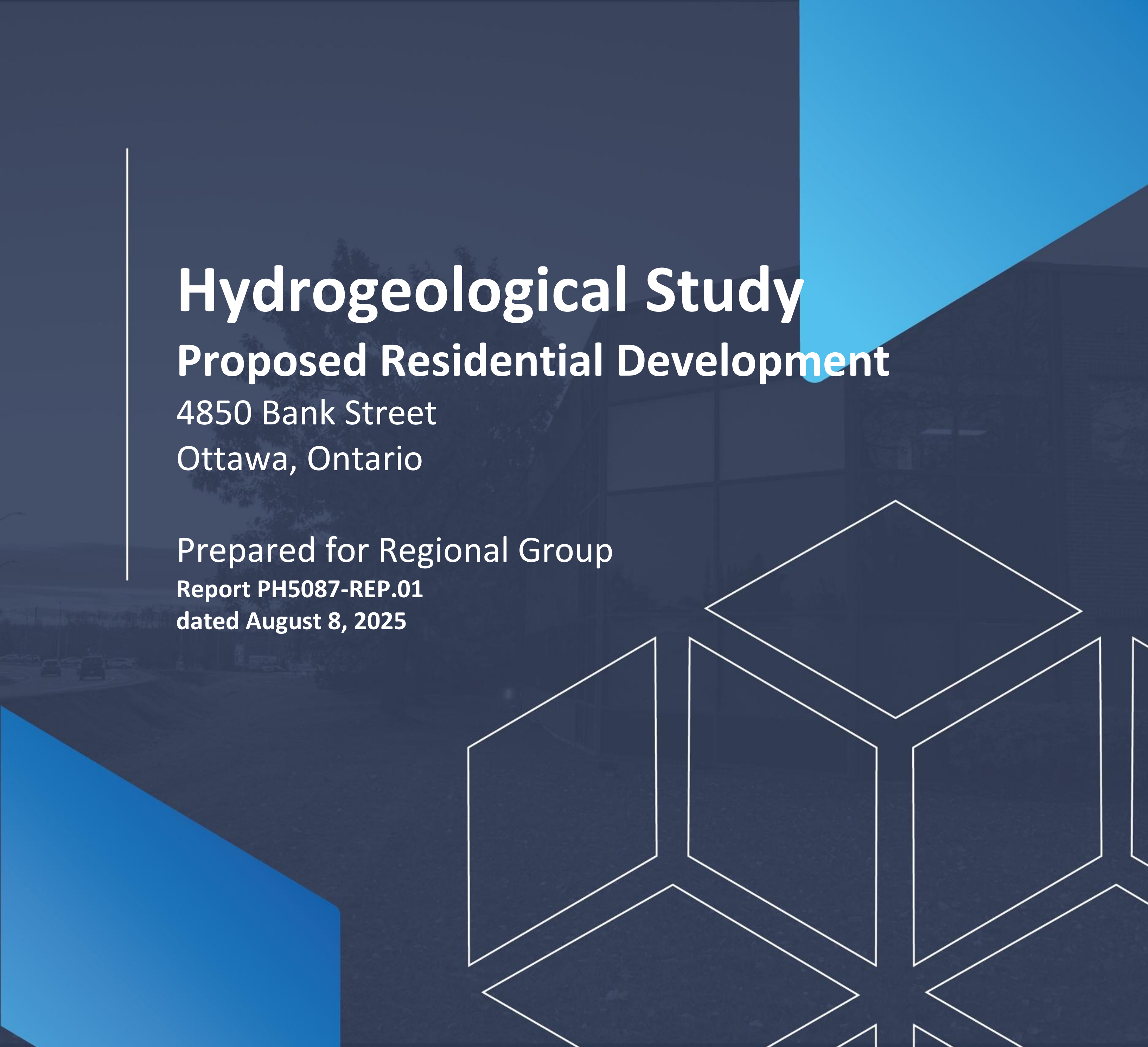

Hydrogeological Study

Proposed Residential Development

4850 Bank Street
Ottawa, Ontario

Prepared for Regional Group
Report PH5087-REP.01
dated August 8, 2025

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1.0 INTRODUCTION

1.1 Background

Paterson Group (Paterson) was retained by Regional Group to conduct a hydrogeological study for the proposed residential development located at 4850 Bank Street in the City of Ottawa (hereinafter referred to as the “subject site”). The location of the subject site is shown on Drawing PH5087-1 - Site Plan appended to this report. This report incorporates the findings of Paterson Report PG6912-1 prepared concurrently.

1.2 Scope of Work

Paterson has completed this report in accordance with the scope prepared by Paterson. As per the agreed upon scope, the purpose of this study was to:

- Characterize the hydrogeological setting of the subject site. Consideration was given to bedrock and surficial geology, aquifer systems, groundwater levels, hydraulic properties and catchment characteristics.
- A groundwater impact assessment to determine potential impacts to adjacent infrastructure, well users and the surrounding environment.

Additionally, the study was to include the standard components of a Water Budget Assessment as per the City of Ottawa’s Water Budget Assessment Terms of Reference, which included the following:

- Review related higher-level studies.
- Conduct pre and post-development water budget analyses, including water budget equations, to determine the hydrogeological function of the subject site in order to assess the need for supplemental stormwater management measures.
- Develop a conceptual model to characterize pre and post-development hydrologic and hydrogeologic site conditions.
- Identify sensitive hydrologic and hydrogeologic features (if any) within the study area.
- Identify water budget targets (if applicable) to mitigate post-development hydrologic and hydrogeologic impacts.
- Identify how climate change projections may impact the water budget.

2.0 PREVIOUS REPORTS

In addition to a review of the general literature summarized in the following sections and in the 'References' section of this report (MECP water well mapping, available geological and physiographic mapping), Paterson reviewed the following site-specific reports:

- ☐ 30260678 – 145172 "Leitrim West Urban Expansion Area – S4 Environmental Impact Study – Riverside South Findlay Creek, City of Ottawa" - prepared by Arcadis Professional Services (Canada) Inc. - prepared concurrently.
- ☐ PG6912-1 Rev.1 "Geotechnical Investigation – Proposed Residential Development & Off-Site Sewer Installation – 4850 Bank Street" - prepared by Paterson Group - prepared concurrently.
- ☐ PE6336-1 "Phase I-II Environmental Site Assessment – 4850 Bank Street" - prepared by Paterson Group - December 20, 2023.

3.0 METHOD OF INVESTIGATION

3.1 Records Review

A review of available geological, and hydrogeological data was completed as a part of this assessment. However, the literature review and previous reports did not provide site-specific data regarding overburden and bedrock aquifers, recharge and discharge conditions or flow contributions to the nearby water features. Further detail is provided in the following sections.

3.2 Field Program

The geotechnical and hydrogeological field programs were developed to assess geology, groundwater conditions, hydraulic gradients and the overall hydrologic/hydrogeologic function of the subject site. The test holes were advanced to various depths across the site to assess hydrogeological and geotechnical conditions.

Geotechnical field investigations were completed by Paterson at the subject site between December 2023 and July 2025. During this time, 18 boreholes were advanced to a maximum depth of 8.5 m below ground surface (bgs). The location of the test holes are shown on Drawing PG6912-1 - Test Hole Location Plan, included in Appendix 1.

Soil samples were obtained from the boreholes by means of split spoon sampling and the sampling of shallow soils directly from auger flights. Split-spoon samples were taken at approximate 0.76 m intervals. In addition to soil sampling, rock core samples were obtained with the use of a standard diamond drill bit. The depth at which split-spoon, auger samples and rock core samples were obtained from the test holes are shown as "**SS**", "**AU**" and “**RC**”, respectively, on the Soil Profile and Test Data sheets, included in Appendix 1.

All samples were classified on site, placed in sealed plastic bags and were transported to our laboratory for further review and testing. Transportation of the samples was completed in accordance with ASTM D4220-95 (2007) - Standard Practice for Preserving and Transporting Soils. Rock core samples were recovered from select boreholes (BH1-25 and BH2-25) drilled during the July 2025 geotechnical 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 Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples. The SPT results are recorded as "N" values on the Soil Profile and Test Data sheets. The "N" value is the number of blows required to drive the split-spoon sampler 300 mm into the ground after an initial penetration of 150 mm using a 63.5 kg hammer falling from a height of 760 mm. This test was done in accordance with ASTM D1586-11 - Standard Method for Penetration Test and Split-Barrel Sampling of Soils.

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.

Reference should be made to the Soil Profile and Test Data sheets presented in Appendix 1 for specific details of the soil profiles encountered at the test hole locations.

Drawdown Analysis - Hydraulic Conductivity Testing

Hydraulic conductivity testing (slug testing) was completed at the monitoring wells installed during the December 2023 and July 2025 geotechnical investigations. Falling head and rising head tests (slug tests) were completed in accordance with ASTM Standard Test Method D 4404 - Field Procedure for Instantaneous Change in Head (Slug) Tests for Determining Hydraulic Properties of Aquifers.

Slug testing was completed during the month of July 2025 by Paterson personnel. The general test method consisted of measuring the static water level in the well, followed by inducing a near-instantaneous change of head in the well and subsequent monitoring of water level recovery with an electronic water level meter and a water level data logger. The change in head was induced by the introduction of either an acetal slug, 0.9 m in length and 38 mm in diameter, or a metal slug, 1.0 m in length and 19 mm in diameter, depending on the well diameter. The slug was introduced to raise the groundwater level in the monitoring well, following which the decrease in water level over time was monitored (falling head test). Once the water level had stabilized (or nearly stabilized), the slug was then removed to lower the groundwater level, following which the increase in water level over time was monitored (rising head test).

Following the completion of the slug tests, the test data was analyzed using AQTESOLV Pro Version 4.5 aquifer analysis software package by HydroSOLVE Inc and the results were processed as per the method set out by Hvorslev (1951). Assumptions inherent in the Hvorslev method include a homogeneous aquifer of infinite extent and a screen length significantly greater than the monitoring well diameter. The assumption regarding screen length and well diameter is considered to be met based upon a typical length of 1.52 m and a diameter of 0.03 to 0.05 m.

While the idealized assumptions regarding aquifer extent, homogeneity are not strictly met in this case (or in any real-world situation), it has been our experience that the Hvorslev method produces effective point estimates of hydraulic conductivity in conditions similar to those encountered at the subject site. Hvorslev analysis is based on the line of best fit through the field data (hydraulic head recovery vs. time), plotted on a semi-logarithmic scale.

Infiltration Testing

In-situ infiltration testing was conducted at the subject site during the month of July 2025 using a Pask (Constant Head) Permeameter. At each testing location, up to two in-situ infiltration tests were conducted at depths of approximately 0.5 to 1.0 m below ground surface (bgs). The tests were conducted to provide general coverage of the subject site and are shown on Drawing PG6912-1 - Test Hole Location Plan included in Appendix 1.

At each testing location, an 83 mm diameter hole was excavated using a Riverside/Bucket auger to remove topsoil and other subsoil material until the desired testing elevation. Up to two holes were advanced at each testing location to approximately 0.5 and 1.0 m bgs. All soil from the auger flights was visually inspected and initially classified on site. An aggregated soil sample was gathered at each test location. Each test was conducted by filling the permeameter reservoir with water and inverting it into the hole, ensuring it was relatively vertical and resting at the bottom of the hole. The water level of the reservoir was monitored at 0.5 to 5 minute intervals until the rate of fall out of the permeameter reached equilibrium, known as a quasi “steady state” flow rate. Quasi steady state flow was considered to be obtained once 3 to 5 consecutive rate of fall readings with identical values were measured. The values for the steady state rate of fall were recorded for each test. The steady state rate of fall was converted to a field saturated hydraulic conductivity value (K_{fs}) using the Engineering Technology Canada Ltd. conversion tables. Unfactored infiltration rates were estimated based on the methodology outlined in Appendix C of the Credit Valley Conservation’s Low Impact Development Stormwater Management Planning and Design Guide.

3.3 Laboratory Testing

All soil samples were retained for laboratory review following the field portion of the subsurface investigation. The soils were classified in general accordance with ASTM D2488-09a, Standard Practice for Description and Identification of Soils (Visual-Manual Procedure). Based on the soil descriptions across the subject site during the geotechnical investigations, these samples are considered to be sufficiently representative of the site.

3.4 Monitoring Well Installations

As part of the December 2023 and July 2025 geotechnical field programs, monitoring wells were installed in select boreholes to permit the monitoring of groundwater levels and conduct drawdown analyses. The well installations were compliant with ASTM D5092 standards.

3.5 Water Level Measurements

Following the completion of the December 2023 and July 2025 drilling programs, groundwater levels were measured at the monitoring well locations. Water levels were measured using an electronic water level meter relative to the ground surface elevation at each location and are noted on the Soil Profile and Test Data sheets, included in Appendix 1.

In addition to the manual groundwater level measurements, select wells were outfitted with dataloggers to record long-term fluctuations in monitoring well water elevations. The monitoring program began in summer 2025 and will last until spring high water levels are captured in 2026.

3.6 Surveying

The test hole locations and ground surface elevations at each test hole location completed by Paterson were surveyed using a GPS unit with respect to a geodetic datum. The locations and ground surface elevations for each test hole are presented on Drawing PG6912-1 - Test Hole Location Plan, included in Appendix 1.

4.0 REVIEW AND EVALUATION

4.1 Physical Setting

At the time of the field investigations, the subject site was undeveloped and heavily forested with mature trees and grass/shrubs. The subject site is located in the City of Ottawa, Ontario and is bordered by Bank Street to the east, forested land and a driving range to the south, forested land and a wetland to the west and a residential development and commercial properties to the north. The location of the subject site is shown on Drawing PH5087-1 - Site Plan, appended to this report.

Based on mapping provided by the Ministry of Conservation and Parks (MECP) Source Protection Information Atlas, the subject site is located within the Castor River subwatershed. There are several surface water features located within the subject site and within 500 m of the subject site. These include unnamed scratch ditches, drainage ditches and conveyance channels. Also, the Leitrim Wetlands lies west of the subject site. The ground surface at the subject site generally slopes downwards to the east and west from the central portion of the subject site. The site is generally at grade with adjacent roadways and properties. The site slopes from east to west with an elevation difference of approximately 6-8 m.

According to available mapping from the Ontario Geologic Survey (OGS; MRD 228), the majority of the subject site is located in the sand plans physiographic region. The eastern tip of the subject site is located in the limestone plains physiographic region. The sand plains region is characterized by silty sand deposits. The limestone plains region is characterized by limestone bedrock. These are generally consistent with field observations at the subject site.

4.2 Geology

Surficial Geology

Overburden mapping provided by the OGS was reviewed as part of this assessment. Available mapping (MRD 128) indicates that overburden soils throughout the subject site consist of till (stone-poor, sandy silt to silty sand-textured till on paleozoic terrain). Overburden soil mapping is shown on Drawing PH5087-3 - Surficial Geology Plan, appended to this report.

Overburden soils identified during the geotechnical investigations by Paterson between December 2023 and July 2025 were generally consistent with the available mapping. Soils generally consisted of topsoil overlying a glacial till deposit followed by bedrock. Fill material was encountered at one (1) borehole location. The fill material generally consisted of brown silty sand with gravel, crushed stone and trace clay, extending to a maximum observed depth of 0.9 m bgs. The glacial till deposit generally consisted of a silty sand matrix with varying amounts of gravel, cobbles and boulders, extending to a maximum observed depth of 6.7 m bgs.

Specific details are provided on the Soil Profile and Test Data Sheets included in Appendix 1 of this report. More details regarding the overburden soils can be found in Paterson Report PG6912-1 prepared concurrently with this report.

Bedrock Geology

Bedrock was encountered between 5.0 to 5.2 m bgs during Paterson’s 2025 geotechnical field investigation and cored to a maximum depth of 8.5 m bgs. The bedrock was observed to vary from poor to excellent quality. Based on available mapping, the overburden drift thickness at the subject site ranges between 3 and 15 m. Bedrock geology mapping is shown on Drawing PH5087-4 - Bedrock Geology Plan, appended to this report.

Karst Features

The term “karst” refers to a geologic formation characterized by the dissolution of carbonate bedrock, such as limestone or dolostone. In order for karstification to occur, precipitation must be able to infiltrate the top of the bedrock, causing dissolution which enlarges previously existing joints and bedding planes. Based on available mapping by the OGS (GRS 005), the subject site is located within an area that does not contain karstic landforms.

4.3 Hydrogeological Setting (Conceptual Model)

Based on the field investigations at the subject site, Paterson used borehole data, existing water well records, topography, monitoring well water levels, hydraulic conductivity and infiltration rates to develop a conceptual model of the transport fate of surface water and groundwater at the subject site. Information related to the conceptual flow model is described below.

Existing Aquifer Systems

Aquifer systems may be defined as geological media, either overburden soils or fractured bedrock, which permit the movement of groundwater under hydraulic gradients. In general, aquifer systems may be present in overburden soils or bedrock. Groundwater was observed within the overburden soils at the subject site and the soils consist of moderate hydraulic conductivities. Given the limited quantity of groundwater within the overburden aquifer, it is not considered an adequate source for water supply wells. If water supply wells are still in use in the vicinity of the subject site, it is anticipated that they are accessing bedrock aquifers.

Based on a review of the MECP water well record database, available geological mapping and field investigations, Paterson has identified two (2) aquifer systems in the vicinity of the study area which consist of the overburden aquifer and underlying bedrock aquifer. The bedrock aquifer system consists of dolostone or dolostone and sandstone of the Oxford or March Formations, respectively.

Groundwater Levels

Piezometers and monitoring wells were placed across the study area for the purpose of monitoring groundwater levels. The piezometers and monitoring wells were installed in the overburden and bedrock. Groundwater levels were observed to be between 0.13 and 1.23 m bgs in the piezometers and between 0.39 and 1.99 m bgs in the monitoring wells. The initial groundwater level measurements are shown on the Soil Profile and Test Data Sheets appended to this report in Appendix 1. Groundwater elevations that were collected on July 3, 2025, were used to determine hydraulic gradients and the general groundwater flow direction at the subject site which is shown on Drawing PH5087-5 - Groundwater Contour Plan appended to this report. The manual measured groundwater levels are presented in Table 1 below.

Table 1 – Summary of Manual Groundwater Level Measurements

Borehole ID	Ground Surface Elevation (m asl)	Manual Groundwater Level Measurement		Date Recorded
		Depth (m bgs)	Elevation (m asl)	
BH 1-25*	101.45	1.74	99.71	July 3, 2025
BH 2-25	103.07	2.18	100.89	July 3, 2025
BH 3-25*	109.30	1.99	107.31	July 3, 2025
BH 1-23*	102.56	2.47	100.09	July 3, 2025
BH 2A-23*	102.25	2.09	100.16	July 3, 2025
BH 6C-23*	101.59	1.21	100.38	July 3, 2025
BH 10-23*	103.35	1.25	102.10	July 3, 2025
BH 1-23*	102.56	1.40	101.16	January 29, 2024
BH 2A-23*	102.25	0.94	101.31	January 29, 2024
BH 10-23*	103.35	0.26	103.09	January 29, 2024
BH 1-23*	102.56	1.30	101.26	December 15, 2023
BH 2A-23*	102.25	0.80	101.45	December 15, 2023
BH 3-23	105.46	1.23	104.23	December 15, 2023
BH 4-23	106.70	Dry	-	December 15, 2023
BH 5-23	107.88	0.40	107.48	December 15, 2023
BH 6C-23*	101.59	0.76	100.83	December 15, 2023
BH 7-23	109.28	0.66	108.62	December 15, 2023
BH 8A-23	107.22	0.27	106.95	December 15, 2023
BH 9-23	106.87	0.13	106.74	December 15, 2023
BH 10-23*	103.35	0.39	102.96	December 15, 2023

Note: The ground surface elevation at each borehole location was surveyed by Paterson using a handheld GPS and was referenced to a geodetic datum.
*Borehole instrumented with groundwater monitoring well.

Horizontal Hydraulic Gradients

Due to the nature of the water levels obtained from field work conducted at the subject site (groundwater monitoring wells), the absolute direction of horizontal hydraulic gradients in the vicinity of the subject site was not determined. However, using the available data, it was possible to approximate the horizontal hydraulic gradients in the overburden materials given that the horizontal hydraulic gradient between any 2 points is the slope of the hydraulic head between those points:

$$i=h_2-h_1/L$$

Where: i = horizontal gradient
h = water elevation (m asl)
L = horizontal distance between test hole locations

Using the above noted formula, the horizontal hydraulic gradients within the eastern portion of the site were observed to be in an approximate easterly orientation with a magnitude of approximately 0.022 m/m in the glacial till deposit. The horizontal hydraulic gradients within the western portion of the site were observed to be in an approximate westerly orientation with a magnitude of approximately 0.016 m/m in the glacial till deposit. The approximate groundwater flow directions are presented on Drawing PH5087-5 - Groundwater Contour Plan, appended to this report. Regional groundwater flow in the overburden is expected to be in a southeasterly orientation towards Castor River.

Hydraulic Conductivity

Hydraulic conductivity testing (slug testing) was completed by Paterson as part of the field investigations at the subject site. The test data was analyzed as per the method set out by Hvorslev (1951). The testing yielded hydraulic conductivity values of 3.71×10^{-7} to 1.21×10^{-5} m/sec for glacial till and 1.05×10^{-4} to 1.20×10^{-4} m/sec for bedrock. Hydraulic conductivity results are summarized in Table 2 below and have been included in Appendix 3.

Table 2 – Summary of Hydraulic Conductivity (Slug) Testing Results

Test Hole ID	Ground Surface Elevation (m asl)	Testing Elevation (m asl)	Hydraulic Conductivity (m/sec)	Test Type	Soil Type
BH1-25	101.45	93.17-94.67	1.05×10^{-4}	Falling Head	Bedrock
			1.20×10^{-4}	Rising Head	
BH3-25	109.30	103.15-104.65	1.33×10^{-6}	Falling Head	Glacial Till
			2.03×10^{-6}	Rising Head	
BH1-23	102.56	98.80-100.30	3.71×10^{-7}	Falling Head	Glacial Till
BH6C-23	101.59	96.97-98.47	9.00×10^{-6}	Falling Head	Glacial Till
			1.21×10^{-5}	Rising Head	
BH10-23	103.35	98.27-99.77	4.28×10^{-6}	Falling Head	Glacial Till
			$5.01 X 10^{-6}$	Rising Head	

In-Situ Infiltration Testing

In-situ infiltration testing was conducted by Paterson using a Pask (Constant Head Well) Permeameter across the subject site to determine the field saturated hydraulic conductivities (K_{fs}) and their respective unfactored infiltration rates of the unsaturated overburden soils. The tests were conducted in a manor to provide general coverage across the subject site. Estimated unfactored infiltration rates varied between 24 and 82 mm/hr for the glacial till unsaturated overburden soils. The variations in the infiltration rates are dependent on the composition/compaction of the glacial till at a given location. In-situ infiltration testing results can be found below in Table 3.

Table 3 – Summary of Field Saturated Hydraulic Conductivity Testing Results and Estimated Infiltration Rates

Test Completed Adjacent to Borehole ID	Ground Surface Elevation (m asl)	Infiltration Testing Elevation (m asl)	K_{fs} (m/sec)	Unfactored Infiltration Rate (mm/hr)	Material
PT1-25	101.67	101.17	2.7 x 10^{-7}	33	Glacial Till
		100.67	9.3 x 10^{-8}	24	Glacial Till
PT2-25	105.82	105.32	5.3 x 10^{-6}	72	Glacial Till
		104.82	8.5 x 10^{-6}	82	Glacial Till
PT3-25	107.29	106.79	2.7 x 10^{-7}	33	Glacial Till
		106.49	1.4 x 10^{-7}	27	Glacial Till
PT4-25	109.49	108.99	2.1 x 10^{-6}	56	Glacial Till
		108.54	1.1 x 10^{-6}	47	Glacial Till
PT5-25	106.11	105.56	1.4 x 10^{-7}	27	Glacial Till

*Field saturated hydraulic conductivity (K_{fs})

**The infiltration rates do not include a safety correction factor. Based on our testing results, a minimum safety correction factor of 2.5 should be applied to the values.

Groundwater Recharge and Discharge

In general, groundwater will follow the path of least resistance from areas of higher hydraulic head to areas of lower hydraulic head. Upward and downward hydraulic gradients are typically indicative of areas of discharge and recharge, respectively.

It is our interpretation that there is some recharge occurring at the subject site due to the soils identified at the time of the field investigations, and the infiltration testing results at the subject site. Recharge of the shallow overburden aquifer will occur as precipitation infiltrates the subsoils where it will flow vertically downward through the unsaturated surficial soils before intercepting the overburden aquifer. However, it should be noted that the site is not mapped as a significant groundwater recharge area (SGRA) by the MECP.

With regards to discharge zones, neither the topographical nor geological conditions are suitable for discharge to be occurring on a large scale at the subject site.

The subject site intersects one subwatershed as previously mentioned. However, within the confines of the subject site, shallow groundwater was found to have flow generally travelling in an eastward direction within the eastern portion of the subject site, and in a western direction within the western portion of the subject site, given the topographic relief.

It should be noted that the subject site is not identified by the MECP as a drinking water protection zone.

Catchment Areas

The subject site is located within the Castor River subwatershed. As shown on Drawing PH5087-5 - Groundwater Contour Plan, the groundwater flows in an eastward direction within the eastern portion of the subject site, and in a western direction within the western portion of the subject site. Therefore, it is Paterson's opinion that the subject site is predominantly characterized by two catchment areas.

Detailed servicing plans were unavailable at the time of report preparation. Based on discussions with the civil design team, the site will continue to be characterized by two catchment areas under post-development which will function similarly to pre-development conditions. However, it is anticipated that the size of the western catchment area will increase to allow for increased hydration to the adjacent Leitrim Wetlands as per recommendations by the Rideau Valley Conservation Authority (RVCA).

Groundwater Inflow/Dewatering Requirements

Three (3) potential sources of dewatering have been identified at the subject site. The sources consist of the excavation footprints related to the building foundations, servicing trenches and stormwater management pond (SWMP). Details regarding the excavation footprints and depths for each potential dewatering source were unavailable at the time of report preparation. Therefore, the building, servicing and SWMP excavations are assumed to encompass an area of approximately 750 m^2, 125 m^2 and 5,000 m^2, respectively, for a preliminary dewatering assessment.

Based on available conceptual drawings for the site, excavation sizes were estimated based on proposed building footprints, typical servicing excavation sizes based on previous experience at similar sites and the proposed SWMP footprint.

The infiltration rates provided for the following sources were calculated using the Dupuit Forchheimer method:

$$Q = \pi k((h_0^2-h_p^2)/ln(R/r))$$

- k = hydraulic conductivity (m/sec)
- h_0 = thickness of the aquifer (m)
- h_p = thickness of the aquifer from the base of the excavation to the base of the aquifer (m)

☐ *R* = effective drawdown radius for the excavation (m)
☐ *r* = equivalent radius of the excavation (m)

The groundwater infiltration calculations for the excavation footprints are provided in Appendix 3 of this report.

The stratigraphy within the anticipated saturated depth of the building excavations generally consists of a glacial till deposit. Specific design details were not available at the time of report preparation. For the purpose of this study, it has been assumed the buildings will consist of slab-on-grade construction or one basement level with a maximum excavation depth of 3 m bgs. Calculations are based on an excavation size of approximately 750 m^2 and a saturated depth of 2 m. Using a representative hydraulic conductivity of 3.03 x 10^{-6} m/sec (geometric mean of the calculated hydraulic conductivities), the steady state volume of groundwater anticipated is approximately 22,000 L/day, per excavation.

The stratigraphy within the anticipated saturated depth of the servicing excavations generally consists of a glacial till deposit. Specific design details were not available at the time of report preparation. For the purpose of this study, it has been assumed the servicing trenches will have a maximum excavation depth of 5 m bgs. Calculations are based on an excavation size of approximately 125 m^2 and a saturated depth of 4 m. Using a representative hydraulic conductivity of 3.03 x 10^{-6} m/sec (geometric mean of the calculated hydraulic conductivities), the steady state volume of groundwater anticipated is approximately 12,000 L/day, per excavation.

The stratigraphy within the anticipated saturated depth of the SWMP excavation generally consists of a glacial till deposit. Specific design details were not available at the time of report preparation. For the purpose of this study, it has been assumed the SWMP excavation will have a maximum excavation depth of 5 m bgs. Calculations are based on an excavation size of approximately 5,000 m^2 and a saturated depth of 4 m. Using a representative hydraulic conductivity of 3.03 x 10^{-6} m/sec (geometric mean of the calculated hydraulic conductivities), the steady state volume of groundwater anticipated is approximately 44,000 L/day.

It is recommended that source specific dewatering calculations be completed once more specific development details are available.

5.0 SITE SPECIFIC WATER BUDGET ASSESSMENT

The site-specific water budget assessment (SSWB) was conducted to determine the hydrogeological function of the subject site, to identify infiltration potential and to identify opportunities for supplemental stormwater management measures. At the time of the field investigations the study area mainly consisted of mature trees, grass and shrubs. The pre and post-development terrain compositions are illustrated on Drawings PH5087-6 - Pre-Development Terrain Composition Plan and PH5087-7 - Post-Development Terrain Composition Plan, appended to this report.

5.1 Calculations

Thornthwaite and Mather Water Balance Calculations

When falling precipitation intercepts the ground, three possible outcomes arise. The water can either evaporate/transpire back into the atmosphere (evapotranspiration), infiltrate into the surface soils (infiltration) or leave the area as runoff.

The method employed by Thornthwaite and Mather (1957) was used along with modelling software by Environment Canada's Engineering Climate Services Unit (EC-ECS) to determine the partitioning of water throughout various portions of the hydrologic cycle. Inputs into the modelling program included monthly temperature, precipitation, water holding capacities and site latitude. Using the long-term averages of these variables, it was possible to calculate annual potential and actual evapotranspiration, change in soil moisture storage and the water surplus.

The formula employed by Thornthwaite and Mather is as follows:

$$S = R + I = P - ET$$

Where:
S = surplus (mm/year)
R = annual runoff (mm/year)
I = annual infiltration (mm/year)
P = annual precipitation (mm/year)
ET = annual evapotranspiration (mm/year).

Shallow unsaturated soils within the study area generally consisted of topsoil overlying a glacial till deposit. Given the similar soil profiles across the entire study area, the above noted calculations were carried out for the soil moisture holding capacity of a fine sandy loam.

Based on the location of the site within the Ottawa area, climatic data was obtained from the climate station located at the McDonald-Cartier International Airport covering the period of January 1939 to December 2022. The information was provided by Environment Canada's Engineering Climate Services Unit and is presented in Appendix 2 of the report.

Table 4, below, displays the soil types present within the study area and their associated water holding capacities (WHC) as well as the actual evapotranspiration (AET) and surplus data. For the purposes of this study, AET values were used as they account for accumulated soil moisture deficit. This deficit represents the volume of water retained within the available pore spaces of the soil and is subtracted from the potential evapotranspiration (PET) value to more accurately calculate the water surplus. The monthly/annual water balance data is presented in Tables 6 and 7 in Appendix 2 of this report. For the purpose of this study, 70% of the detached homes property footprints are considered to be impervious surfaces (100% of surplus will result in runoff) and 30% are considered to be urban lawns.

Table 4 - Site Specific Water Surplus Information

Land Use Unit	Water Holding Capacity (mm)	Actual Evapotranspiration (mm/year)	Surplus Water (mm/year)
Impervious Surfaces	N/A	145*	759
Urban Lawn/Shallow Rooted Crops (Fine Sandy Loam)	75	525	378
Mature Forests (Fine Sandy Loam)	300	605	298

Table reproduced using WHC values from MOE (2003) - Stormwater Management Planning and Design Manual and modelling data from Environment Canada's Engineering Climate Services Unit.
*Values based on evaporation information for urban areas (16% of precipitation) included in the Eastern Ontario Water Resources Management Study prepared by CH2M HILL Canada Limited (March 30, 2001).

Infiltration Factors

In order to break down the surplus water values for the various materials into infiltration and runoff, various factors must be considered. The MOE Stormwater Management Planning and Design Manual (2003) lists three main factors that contribute to surface water infiltration rates.

The first factor is topography, which is broken down further into three sections: flat and average slope, rolling land and hilly land. Flat and average slope provides the greatest potential for infiltration and has the largest infiltration factor applied to it (0.3), while the other two have progressively lower infiltration factors (rolling land is 0.2 and hilly land is 0.1).

The second factor is soil, which is also broken down further into three sections: tight impervious clay, medium combinations of clay and loam and open sandy loam. Open sandy loam provides the greatest potential for infiltration (infiltration factor of 0.4) while the other two have progressively lower potential for infiltration to occur (infiltration factor for medium combinations of clay and loam is 0.2 and for tight impervious clay is 0.1).

The final factor the MOE manual uses to partition infiltration from runoff is land cover. It is broken down into two sections: open fields/cultivated lands and woodlands. Woodlands have greater infiltration potential and an infiltration factor of 0.2. Open fields and cultivated lands have lower potential and with an infiltration factor of 0.1. A summary of the MOE manual's descriptors and their associated infiltration factors is shown below in Table 5.

Table 5 - MOE (2003) Infiltration Factors

Description of Area/Development Site	**Value of Infiltration Factor**
Topography	
Flat and average slope (<0.6 m/km)	0.30
Rolling land (slope of 2.8-3.8 m/km)	0.20
Hilly land (slope of 28-47 m/km)	0.10
Soil	
Tight impervious clay	0.10
Medium combinations of clay and loam	0.20
Open sandy loam	0.40
Cover	
Open fields/cultivated lands	0.10
Woodlands	0.20

Table reproduced from MOE (2003) - Stormwater Management Planning and Design Manual.

The topography of the study area is classified as hilly land (slope of 28-47 m/km throughout the subject site) Therefore, a pre-development topography infiltration factor of 0.1 was given for the materials analysed on this property. In order for development to proceed, it is expected that alterations will be made to the topography of the site. In general, it is expected that the overall slope of the site will be reduced to accommodate buildings and parking areas. Therefore, the topography of the subject site under post-development conditions will consist of a mix rolling and hilly land and was therefore assigned a post-development topography infiltration factor of 0.15. An infiltration factor of 0 was assigned to the impervious surfaces due to its negligible infiltration capacity.

As previously discussed, soils within the study area generally consisted of topsoil overlying a glacial till deposit. Therefore, a pre-development soil infiltration factor of 0.3 was given for the materials analysed on this property. Under post-development conditions, the majority of the site will consist of either landscaped areas or impervious surfaces, with soil infiltration factors ranging from 0.3 for fine sandy loam to 0 for impervious surfaces.

At the time of the field investigations, the subject site generally consisted of mature forest. A pre-development vegetation infiltration factor of 0.2 was therefore used for the site. Post-development, it is expected the majority of the trees remaining on site will be removed to accommodate buildings, parking areas and roadways. As such, a post-development vegetation infiltration factor of 0.1 was assigned to the site, except for impervious surfaces, which were given an infiltration factor of 0 due to its negligible potential to benefit from vegetation cover.

The pre and post-development infiltration factors for all materials considered are included in the water budget calculations provided in Table 8 and Table 9 included in Appendix 2 of this report.

5.2 Pre and Post-Development Water Budget

The pre-development water budget analysis conducted for the study area determined that an estimated 24,482,548 L/year of surplus water currently infiltrates the surface soils. The remaining estimated 16,321,698 L/year of surplus leaves the site as runoff.

The post-development water budget analysis determined that an estimated 9,064,556 L/year of surplus water will infiltrate the surface soils and approximately 58,058,376 L/year will leave the site as runoff. These values equate to an approximate decrease in infiltration of 63% and an increase in runoff of 356%.

The main variable that changed from pre-development conditions to post-development conditions was the addition of approximately 8.8 hectares of impervious surfaces. This results in reducing the area of pervious materials throughout the subject site, therefore, reducing the overall infiltration potential of the subject site. The remaining areas that are not being converted to impervious surfaces will become landscaped surfaces characterized by urban lawn (fine sandy loam) material. Also, it should be noted that the SWMP area was excluded from the post-development water budget analysis given that this area is designed to manage stormwater runoff and does not contribute to the infiltration or runoff potential of the site.

It is important to note that the post-development water budget analysis for the subject site does not consider any potential infiltration of the impervious surfaces (100% runoff was taken as a conservative approach). In reality, some portion of surface water that lands on impervious surfaces infiltrates (asphalt is not 100% impervious) or is diverted to grassed areas where additional infiltration may occur. As such, the post-development runoff volumes should be considered a conservative estimate and not expected to definitively represent future conditions.

Details of pre-development water budget analyses are presented in Tables 8 and 9, included in Appendix 2 of this report.

6.0 GROUNDWATER IMPACT ASSESSMENT

6.1 Impact of Proposed Development on Surrounding Infrastructure

As previously discussed, soils within the subject site generally consisted of topsoil, overlying a glacial till deposit. The expected groundwater infiltration will be encountered within the glacial till deposit.

The steady-state radius of influence calculations completed were based upon the Sichardt equation as shown below. The assumed setting for the analytical solution was one in which open cut trenches were used to install the services at the subject site, creating an unconfined condition which would allow use of the equation to determine the radius of influence.

$$R = 3000 * \Delta h(K^{0.5})$$

Where: R = radius of influence (m)
Δh = expected groundwater drawdown (m)
K = hydraulic conductivity (m/sec).

For the purposes of completing the calculations, the following values were used in the analysis for the glacial till:

- ❒ Δh = 2 to 4 m
- ❒ K = 3.71×10^{-7} to 1.21×10^{-5} m/sec, based on site specific hydraulic conductivity values of the glacial till.

Using the above equation and assumptions, a radius of influence of 5 to 42 m will develop as a steady state condition within the glacial till, extending from the edge of the excavation, in the area of the subject site, depending on the groundwater levels and hydraulic properties of the specific soils encountered.

The surrounding area consists of mostly low-rise residential homes and commercial properties. The buildings located within the theoretical radius of influence are generally expected to be founded on the glacial till deposit. The majority of the groundwater infiltration is expected to occur within the glacial till with minimal compressibility. Furthermore, water takings are also expected to be short term in duration, given the nature of the development. As such, adverse effects to surrounding infrastructure related to dewatering activities at the subject site are expected to be negligible.

6.2 Impact of Proposed Development on Existing Well Users

A search of the Ontario Water Well Records online mapping database indicates there are a number of wells within 500 m of the site as depicted on Drawing PH5087-2 - MECP Water Well Location Plan, appended to this report. However, it is expected that the majority of these wells are either no longer in use due to their installation dates and developed nature of the region or are monitoring well installations. Additionally, the majority of properties surrounding the site are serviced by municipal water supplies. Any wells that may still be in use are cased well below the anticipated excavation depths associated with the proposed development and are accessing the deeper bedrock aquifer. Furthermore, the properties that reside on the eastern boundary of the subject site will have the opportunity to connect to municipal services once the development proceeds. Therefore, it is anticipated that the existing wells will either no longer be in use or have adequate vertical and horizontal separation from proposed construction activities. Therefore, dewatering activities at the subject site are not expected to cause any interference to the water supply of surrounding properties or other negative impacts.

As the potential to interfere with the water quality/quantity of existing well users in the area is negligible, a water well monitoring program is not recommended for the aforementioned water takings. If wells are found to remain in existence on the subject site, they should be decommissioned in accordance with Ontario Regulation 903.

If construction activities are shown to cause negative impacts to the water supplies of existing well users, the contractor shall take action to make available a supply of water equivalent in quality and quantity of their typical takings or shall compensate those affected for reasonable costs for doing so, or shall reduce water taking amounts to alleviate the negative impacts. The contractor shall provide temporary water supplies, to those affected, to meet their typical takings or compensate such persons for reasonable costs associated to do so until permanent restoration of the affected water supply or an equivalent source.

6.3 Impact of Proposed Development on the Environment

A search of the MECP Environmental Site Registry for Records Site Condition (RSCs) was conducted as part of the assessment of the site, neighbouring properties and the general area. No RSCs were identified within the 500 m of the subject site.

A Phase II Environmental Site Assessment (ESA) was completed by Paterson for the subject site. All soil samples were found to be in compliance with MECP Table 2 Residential standards. All groundwater samples were found to be in compliance with MECP Table 2 Potable standards.

There are several surface water features located within the subject site and within 500 m of the subject site. These include unnamed scratch ditches, drainage ditches and conveyance channels. Also, the Leitrim Wetlands lies west of the subject site.

As per the Environmental Impact Study prepared by Arcadis, select drainage features located within the subject site are expected to be infilled/removed as part of the proposed development. However, removal of these features is not anticipated to impact the overall hydrogeologic/hydrologic as stormwater management infrastructure will replicate pre-development discharge conditions. Furthermore, no impacts were identified to the features outside the development area.

6.4 Adjacent PTTW/EASRs/ECAs

A search of the MECP Permit to Take Water (PTTW) database provided one (1) PTTW within a 500 m radius of the subject site.

PTTW 2014-BAQMK2 is registered to 4840 Bank St. Ltd. and is located within the residential development currently under construction to the north of the subject site. The above noted permit contains 2 sources (Services Trenches and Miscellaneous Ponded Areas) with a maximum taking of 5,200,000 L/day.

A search of the MECP Environmental Activity and Sector Registry (EASR) database provided one (1) active EASR within a 500 m radius of the subject site.

EASR R-009-7232525387 is registered to RON EASTERN CONSTRUCTION LTD. and is located within the residential development currently under construction to the north of the subject site. The above noted permit contains 2 sources (Building Excavation and Servicing Excavation) with a maximum taking of 400,000 L/day, per source.

Based on available mapping (GeoOttawa), it is expected that the servicing has been completed for the adjacent residential development at 4840 Bank Street. Therefore, it is unlikely that dewatering between the two sites would be taking place concurrently. Cumulative impacts related to anticipated dewatering activities for the proposed development from adjacent water taking permits are not anticipated.

With respect to Environmental Compliance Approvals (ECAs), given the nature of the development in the area (residential and commercial), there are several ECAs that exist for various purposes in the areas bordering the site. Eight (8) ECAs were found to be in relation to existing stormwater management systems in the area. Upon review of the aforementioned ECAs, the ECAs generally relate to installations of new sanitary and storm sewers for new residential developments along with stormwater management strategies on the respective sites. As the ECAs relate to municipal stormwater infrastructure, no concerns were found from a hydrogeological perspective in relation to the subject site.

7.0 ASSESSMENT AND RECOMMENDTIONS

7.1 Sources of Contamination

Based on the soil and groundwater samples collected at the subject site as part of the Phase II ESA investigation, all soil and groundwater samples were found to be in compliance with MECP Table 2 Residential standards and MECP Table 2 Potable standards, respectively.

Prior to and during site development, it is recommended that construction best management practices with respect to fuels and chemical handling, spill prevention, and erosion and sediment control be followed. This will minimize the potential for the introduction of contaminants to the soil, surface water, or groundwater at the subject site.

It is anticipated that the material on site will be disposed of as per Ontario Regulation 406/19 – On-site and Excess Soil Management.

With respect to stormwater runoff quality, it is recommended that best management practices with respect to operational standards be maintained for any stormwater management facilities constructed for the proposed development. It is also recommended that adherence to the City of Ottawa Salt Management Plan - Appendix A (October, 2011) included in Appendix 4 is enforced to ensure that chloride levels in stormwater runoff are minimized.

7.2 Surface Water Features

There are several surface water features located within the subject site and within 500 m of the subject site. These include unnamed scratch ditches, drainage ditches and conveyance channels. Also, the Leitrim Wetlands lies west of the subject site. However, as previously discussed and as per the Environmental Impact Study prepared by Arcadis, no significant impacts are expected from the proposed development on the overall hydrologic function of the area or sensitive features.

With respect to water discharge, water that is pumped from on site excavations must be managed in an appropriate manner. The contractor will be required to implement a water management program to dispose of the pumped water. If the discharge point for the pumped water is directed to overland drainage, it is expected that a multi-barrier approach (such as hay bales, geosocks, silt fence, etc.) to a non-frozen, well vegetated area will be utilized in order to promote reinfiltration prior to reaching a watercourse. Furthermore, if the discharged water is to be directed to overland drainage within 30 m of a water body/watercourse, the turbidity of the water shall not exceed 8 NTU above background levels of the nearest water body. The contractor will be required to maintain appropriate BMPs with respect to sediment and erosion control to ensure negative effects to the surrounding environment are minimized.

7.3 Existing Wells

Any wells within the subject site must be decommissioned prior to construction in accordance with Ontario Regulation 903.

If construction activities are shown to cause negative impacts to the water supplies of existing well users, the contractor shall take action to make available a supply of water equivalent in quality and quantity of their typical takings, or shall compensate those affected for reasonable costs for doing so, or shall reduce water taking amounts to alleviate the negative impacts. The contractor shall provide temporary water supplies, to those affected, to meet their typical takings or compensate such persons for reasonable costs associated to do so until permanent restoration of the affected water supply or an equivalent source. As the potential to interfere with the water quality/quantity of existing well users in the area is negligible, a water well monitoring program is not recommended for the proposed development.

7.4 Water Taking Permitting Requirements

If water taking volumes are greater than 50,000 L/day, a MECP water taking Environmental Activity Sector Registry (EASR) or Permit to Take Water (PTTW) will be required. Depending on the nature of the proposed water takings, an additional hydrogeological investigation may be required.

7.5 Infiltration Potential

As previously discussed, surficial soils within the study area generally consisted of topsoil overlying a glacial till deposit. With regards to infiltration rates for the soils found on-site, site-specific testing varied from 24 to 82 mm/hr for the glacial till. The variations in the infiltration rates are dependent on the composition/compaction of the glacial till at a given location. A minimum safety correction factor of 2.5 will need to be applied to the estimated infiltration rates noted above prior to consideration in the stormwater management design.

Additional infiltration testing may be required once additional development details become available.

As noted above, the results of the water budget analyses completed at the subject site indicated that 24,482,548 L/year of infiltration and 16,321,698 L/year of surface runoff are occurring under pre-development conditions. Under post development conditions, it is expected that there will be a 63% infiltration deficit and a 356% increase in runoff. Therefore, it will likely be necessary to incorporate various stormwater management measures into the design of the development. It should be noted that Paterson's water budget assessment is based on mean water budget values for the soil types at the subject site that were calculated by modeling conducted by EC-ECS. The EC-ECS model is calibrated to historical climate data and does not account for climate change predictions. Therefore, based on the National Capital Commission and City of Ottawa climate change predictions, the stormwater management design team could consider potential seasonal changes (longer spring and shorter winter) and increases in temperature and precipitation when developing their stormwater management strategy.

The stormwater management strategy should target methods best suited to mitigate the impacts that may arise due to the post-development decrease in infiltration and increase in runoff at the subject site, while maintaining consideration of site constraints as defined by the City of Ottawa's Technical Bulletin IWSTB-2024-04 and the MECP's Consolidated Linear Infrastructure Environmental Compliance Approval documentation. Based on the site constraints observed at the subject site (i.e., shallow water table), infiltration based Low Impact Development (LID) measures are not recommended at the time of report preparation. As an alternative, the stormwater management strategy should include conventional stormwater management measures (i.e., end of pipe quality and quantity control) as well as the consideration of Best Management Practices.

8.0 CLOSURE

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

A hydrogeological review of this nature is a limited sampling of a site. The recommendations are based on information gathered at the specific test locations and can only be extrapolated to an undefined limited area around the test locations. Should any conditions at the site be encountered which differ from those at the test locations, we request notification immediately in order to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than Regional Group or their agent(s) is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Paterson Group Inc.

Zavian Buchanan

Zavian Buchanan, E.I.T.

Oliver Blume, P.Geo.

PROFESSIONAL GEOSCIENTIST
Aug. 8, 2025
OLIVER BLUME
PRACTISING MEMBER
3750
ONTARIO

9.0 REFERENCES

Government of Ontario. Provincial Policy Statement 2020 Under the Planning Act.

Engineering Technologies Canada Ltd, “ETC Pask (Constant Head Well) Permeameter – User Guide”, dated March 2016.

Transportation Association of Canada, “Syntheses of Best Practices – Road Salt Management”, dated 2013.

“Characterization of Ottawa’s Watersheds”, Prepared for the City of Ottawa, dated March 2011.

“Surficial Geology of Southern Ontario (MRD128)”, Prepared by the Ontario Geological Survey, 2010

“Karst of Southern Ontario and Manitoulin Island (GRS005)” Prepared by the Ontario Geological Survey, 2008

“Paleozoic Geology of Southern Ontario (MRD219)”, Prepared by the Ontario Geological Survey, 2007

“Physiography of Southern Ontario (MRD228)”, Prepared by the Ontario Geological Survey, 2007

Chapman, L.J., and Putnam, D. F. “The Physiography of Southern Ontario, Third Edition”. Ontario Ministry of Natural Resources, 1984.

Freeze, R.A., and Cherry, J.A. “Groundwater”. Prentice-Hall, Inc., 1979.

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
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TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL

REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:

SITE PLAN

Scale: 1:3000
Drawn by: GK
Checked by: ZB
Approved by: OB
Date: 08/2025
Report No.: PH5087-REP.01
Dwg. No.: **PH5087-1**
Revision No.:

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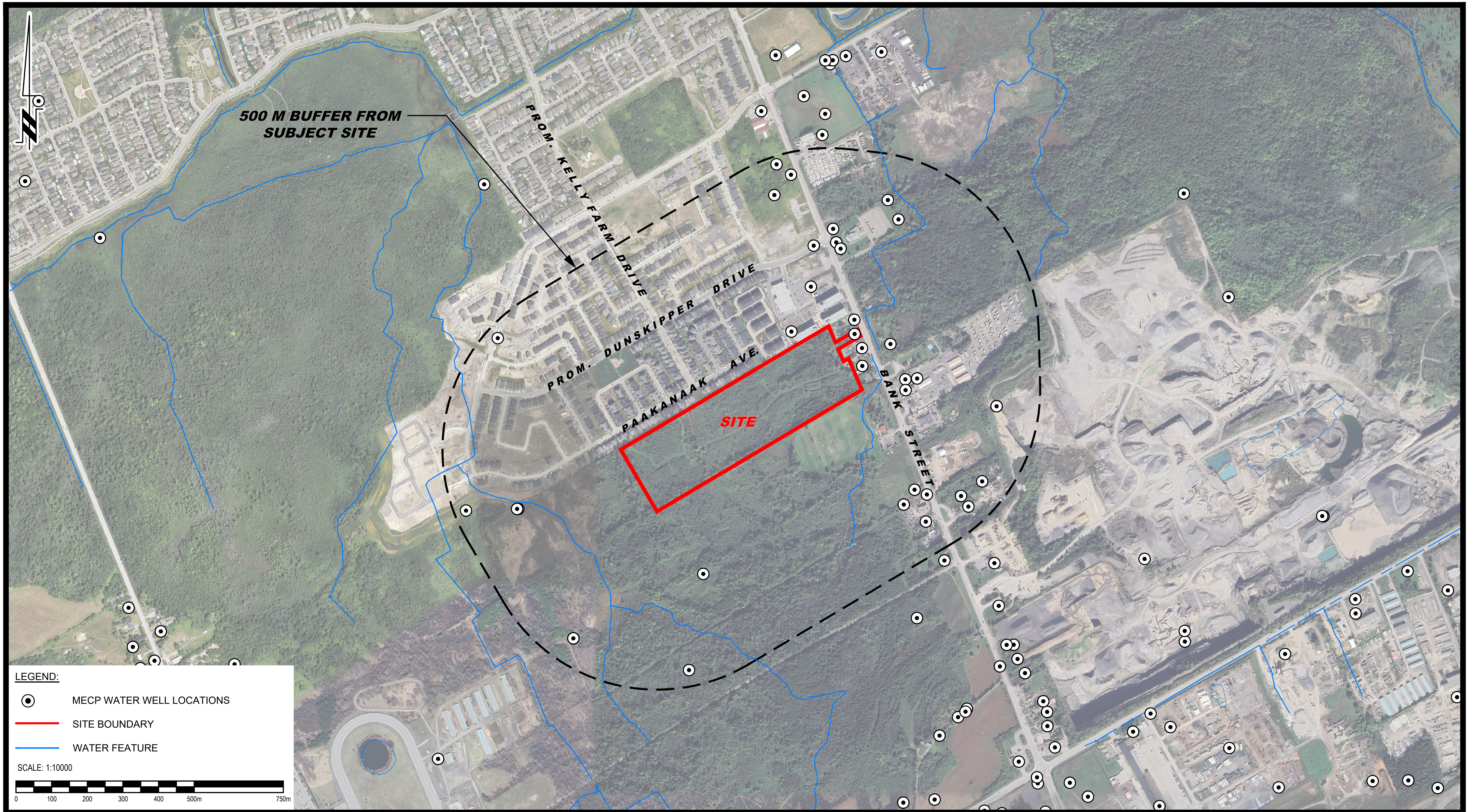

PATERSON GROUP

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NO.	REVISIONS	DATE	INITIAL

REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:

MECP WATER WELL LOCATION PLAN

Scale: 1:10000
Drawn by: GK
Checked by: ZB
Approved by: OB
Date: 08/2025
Report No.: PH5087-REP.01
Dwg. No.: **PH5087-2**
Revision No.:

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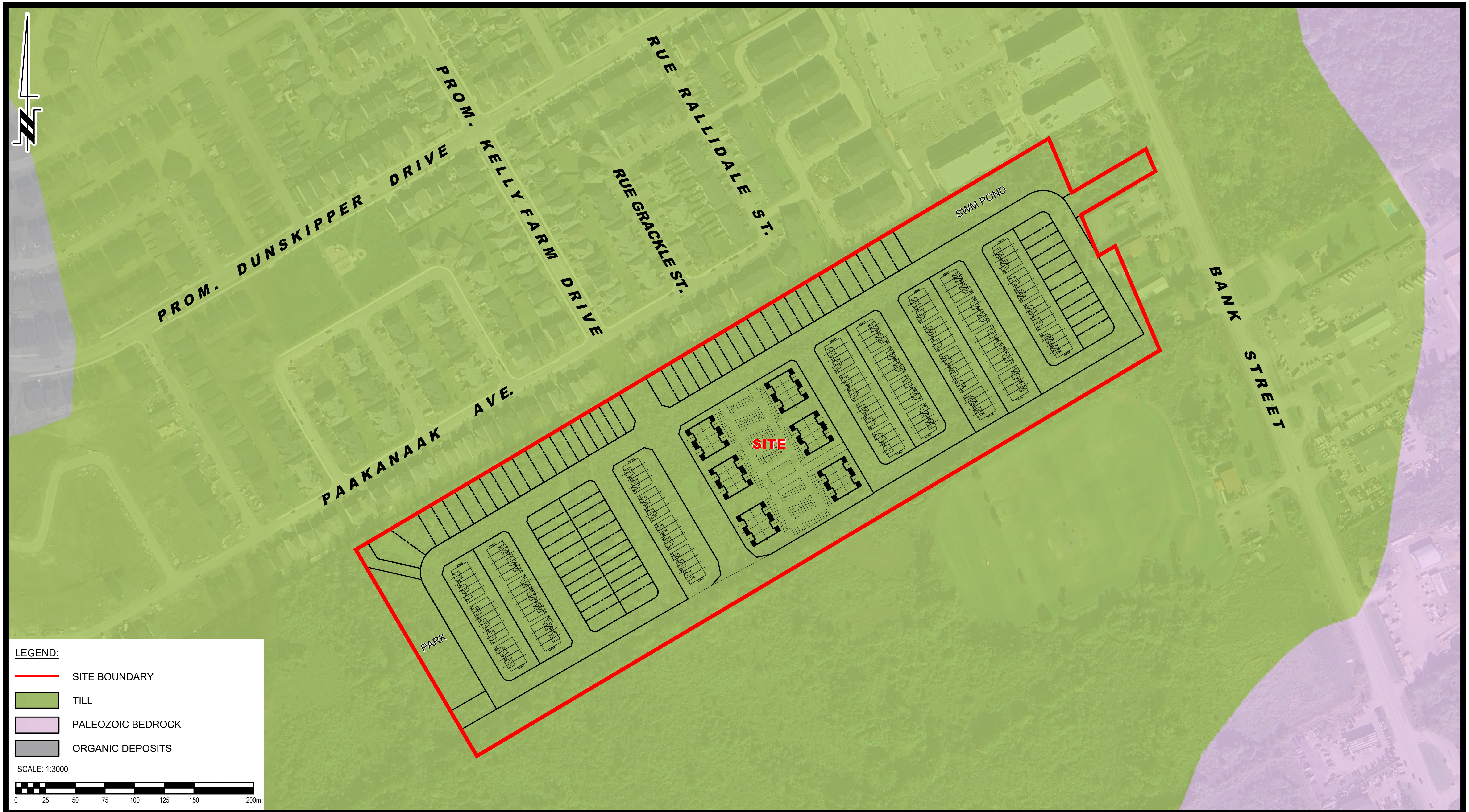

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REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:

SURFICIAL GEOLOGY PLAN

Scale: 1:3000
Date: 08/2025
Drawn by: GK
Report No.: PH5087-REP.01
Checked by: ZB
Dwg. No.: **PH5087-3**
Approved by: OB
Revision No.:

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REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:

BEDROCK GEOLOGY PLAN

Scale: 1:3000
Date: 08/2025
Drawn by: GK
Report No.: PH5087-REP.01
Checked by: ZB
Dwg. No.: **PH5087-4**
Approved by: OB
Revision No.:

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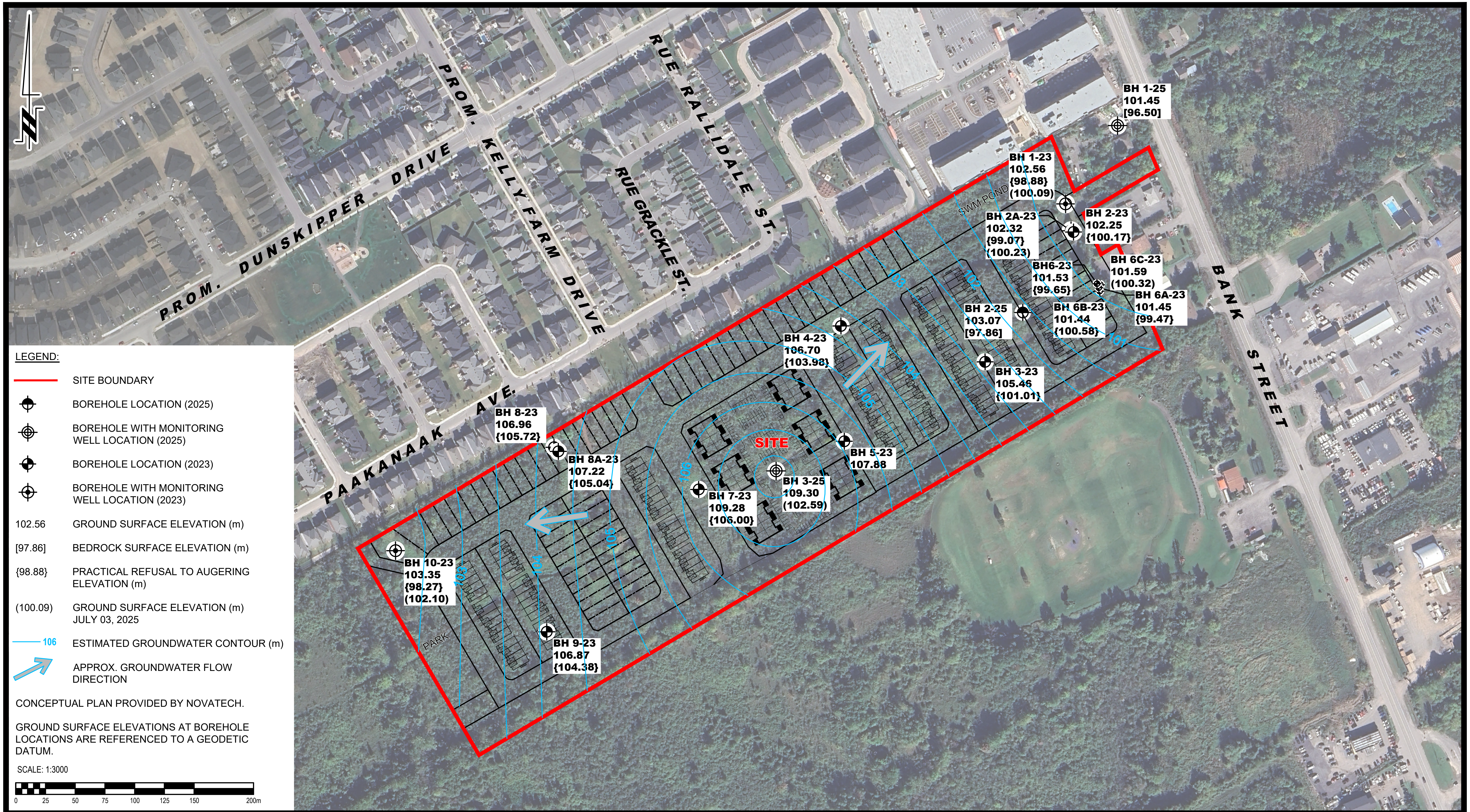

LEGEND:

🟥	SITE BOUNDARY
⌖	BOREHOLE LOCATION (2025)
◎	BOREHOLE WITH MONITORING WELL LOCATION (2025)
●	BOREHOLE LOCATION (2023)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (2023)
102.56	GROUND SURFACE ELEVATION (m)
[97.86]	BEDROCK SURFACE ELEVATION (m)
{98.88}	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
(100.09)	GROUND SURFACE ELEVATION (m) JULY 03, 2025
106	ESTIMATED GROUNDWATER CONTOUR (m)
↗	APPROX. GROUNDWATER FLOW DIRECTION

CONCEPTUAL PLAN PROVIDED BY NOVATECH.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:3000

0 25 50 75 100 125 150 200m

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NO.	REVISIONS	DATE	INITIAL

REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:

OVERBURDEN GROUNDWATER CONTOUR PLAN

Scale: 1:3000
Date: 08/2025
Drawn by: GK
Report No.: PH5087-REP.01
Checked by: ZB
Dwg. No.: PH5087-5
Approved by: OB
Revision No.:

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NO.	REVISIONS	DATE	INITIAL

REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:
PRE-DEVELOPMENT TERRAIN COMPOSITION PLAN

Scale: 1:3000
Drawn by: GK
Checked by: ZB
Approved by: OB

Date: 08/2025
Report No.: PH5087-REP.01
Dwg. No.: PH5087-6
Revision No.:

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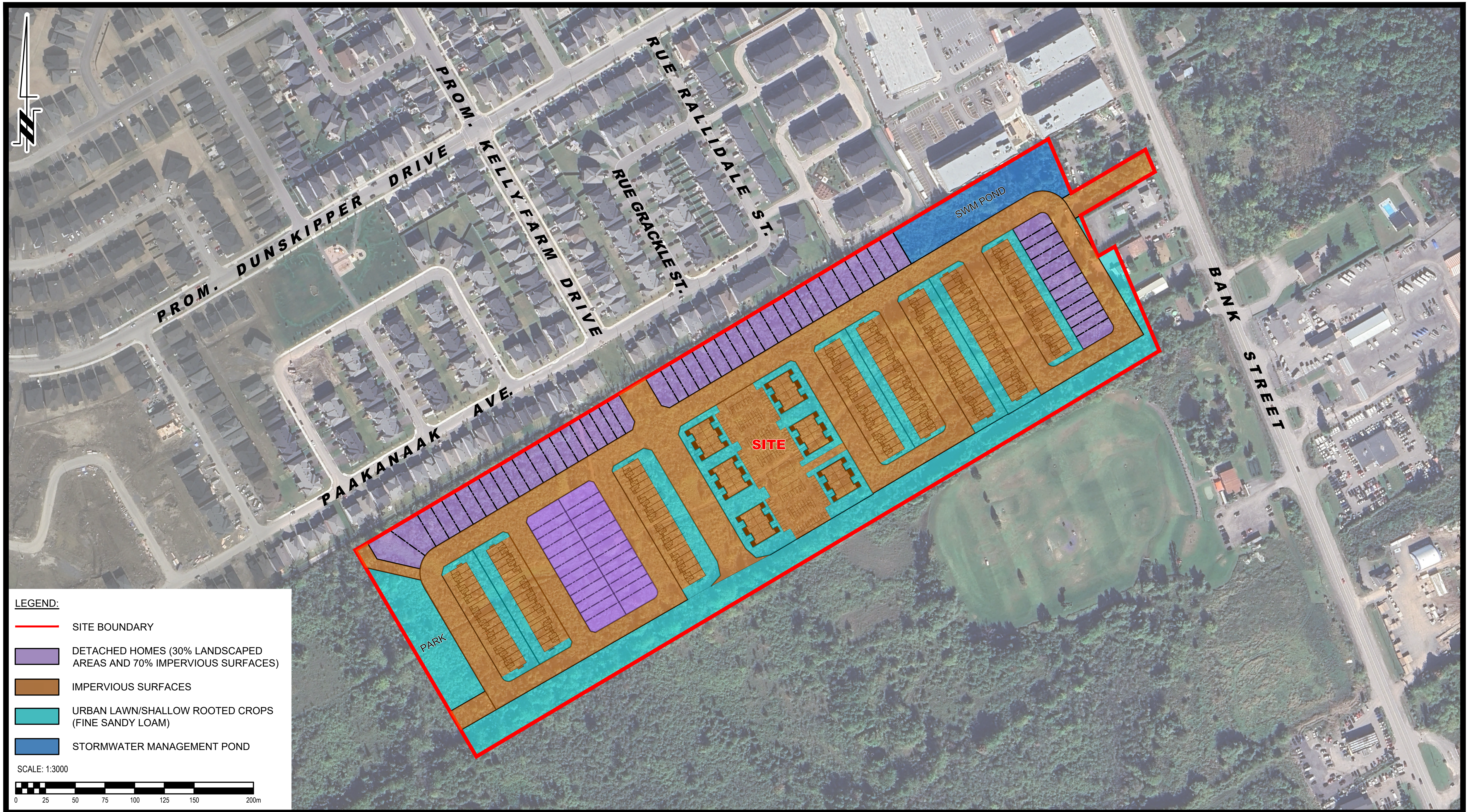

LEGEND:

🟥 SITE BOUNDARY

🟪 DETACHED HOMES (30% LANDSCAPED AREAS AND 70% IMPERVIOUS SURFACES)

🟫 IMPERVIOUS SURFACES

🟦 URBAN LAWN/SHALLOW ROOTED CROPS (FINE SANDY LOAM)

🟦 STORMWATER MANAGEMENT POND

SCALE: 1:3000

0 25 50 75 100 125 150 200m

PATERSON GROUP

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NO.	REVISIONS	DATE	INITIAL

REGIONAL GROUP
HYDROGEOLOGICAL STUDY
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:
POST-DEVELOPMENT TERRAIN COMPOSITION PLAN

Scale: 1:3000
Date: 08/2025
Drawn by: GK
Report No.: PH5087-REP.01
Checked by: ZB
Dwg. No.: PH5087-7
Approved by: OB
Revision No.:

11x17

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

DRAWING PG6912-1 - TEST HOLE LOCATION PLAN

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4850 Bank Street, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 376393.34 **NORTHING:** 5019077.45 **ELEVATION:** 101.45

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 18, 2025

FILE NO. : PG6912

HOLE NO. : BH 1-25

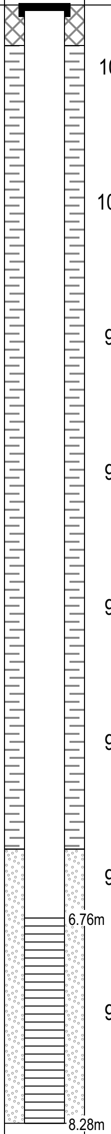
■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION GROUND SURFACE	DEPTH (m) [ELEVATION]	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
FILL: Compact, brown silty sand with gravel and crushed stone	0.61m [100.84m]	AU 1			
FILL: Compact, brown silty sand, trace gravel	0.91m [100.55m]				
GLACIAL TILL: Very dense, brown silty fine sand with gravel, trace clay, occasional cobbles and boulders		SS 2	58	2-5-12-10 17	
		SS 3	85	9-24-50-/ 74/0.05	
	3.20m [98.25m]	SS 4	63	50-/-/-/ 50/0.13	
GLACIAL TILL: Very dense, grey silty fine sand with gravel, cobbles and boulders		SS 5	46	14-45-50-/ 95/0.03	
		RC 1	71		
	4.95m [96.50m]	RC 2	100	RQD 65	
BEDROCK: Poor quality bedrock - Vertical fractures from 5.94 m to 6.96 m depth		RC 3	100	RQD 31	
- Excellent quality bedrock below 6.96 m depth	8.46m [93.00m]	RC 4	100	RQD 90	
End of Borehole					

MONITORING WELL CONSTRUCTION



ELEVATION (m): 101, 100, 99, 98, 97, 96, 95, 94, 93, 92

DEPTH (m): 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4850 Bank Street, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 376313.61 **NORTHING:** 5018920.17 **ELEVATION:** 103.07

PROJECT: Proposed Residential Development

ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 18, 2025

FILE NO. : PG6912

HOLE NO. : BH 2-25

■ PEN. RESIST. (BLOWS/0.3m) DCPT (50mm DIA. CONE) 20 40 60 80

△ REMOULDED SHEAR STRENGTH (kPa)
▲ UNDRAINED SHEAR STRENGTH (kPa) 20 40 60 80

PL (%) WATER CONTENT (%) LL (%) 20 40 60 80

SAMPLE DESCRIPTION	Bottom depth [elevation]
GROUND SURFACE	
TOPSOIL: with organics	0.18m [102.89m]
GLACIAL TILL: Very dense, brown silty fine sand with gravel, trace clay, occasional cobbles and boulders	1.98m [101.09m]
GLACIAL TILL: Very dense, grey silty fine sand with gravel, cobbles and boulders	5.21m [97.86m]
BEDROCK: Excellent quality bedrock	7.06m [96.01m]
End of Borehole	

TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
AU 1			
SS 2	74	12-25-50-/ 75/0.1	
SS 3	73	29-50-/-/ 50/0.05	
SS 4	76	32-50-/-/ 50/0.05	
SS 5	0	50-/-/-/ 50/0.03	
RC 1	89		
RC 2	100	RQD 57	
RC 3	100	RQD 100	

STRATA PLOT | DEPTH (m) | PIEZOMETER CONSTRUCTION | ELEVATION (m)

DEPTH (m)	ELEVATION (m)
	103
1	102
2	101
3	100
4	99
5	98
6	97
7	96
8	95
9	94
10	

P:/AutoCAD Drawings/Test Hole Data Files/PE69xx/PG6912/data.sqlite 2025-07-10, 12:08 Paterson_Template AE

SOIL PROFILE AND TEST DATA

Geotechnical Investigation

4850 Bank Street, Ottawa, ON

COORD. SYS.: MTM ZONE 9 **EASTING:** 376106.17 **NORTHING:** 5018786.90 **ELEVATION:** 109.30

PROJECT: Proposed Residential Development

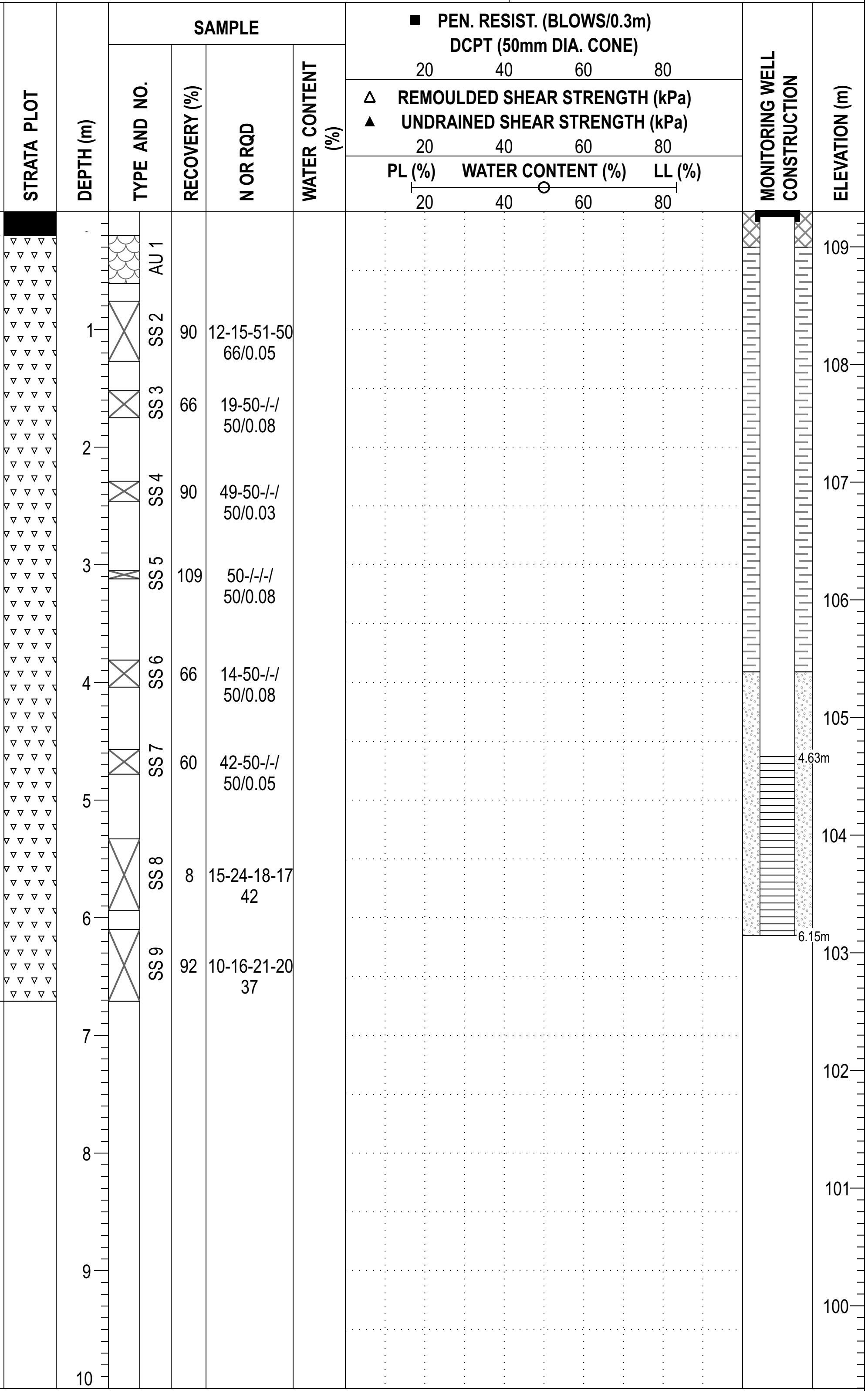
ADVANCED BY: CME-55 Low Clearance Drill

REMARKS:

DATE: June 20, 2025

FILE NO. : PG6912

HOLE NO. : BH 3-25



SAMPLE DESCRIPTION	TYPE AND NO.	RECOVERY (%)	N OR RQD	WATER CONTENT (%)
GROUND SURFACE				
TOPSOIL: with organics 0.20m [109.10m]				
GLACIAL TILL: Very dense, brown silty fine sand, some gravel, cobbles and boulders	AU 1			
	SS 2	90	12-15-51-50 66/0.05	
	SS 3	66	19-50-/-/ 50/0.08	
	SS 4	90	49-50-/-/ 50/0.03	
	SS 5	109	50-/-/-/ 50/0.08	
	SS 6	66	14-50-/-/ 50/0.08	
- Grey by 4.57 m depth	SS 7	60	42-50-/-/ 50/0.05	
- Cobbles and boulders decreasing with depth				
- Silty fine to medium sand by 6.10 m depth	SS 8	8	15-24-18-17 42	
6.71m [102.59m]	SS 9	92	10-16-21-20 37	
End of Borehole				

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376352.245 **NORTHING:** 5019009.799 **ELEVATION:** 102.56

PROJECT: Proposed Development

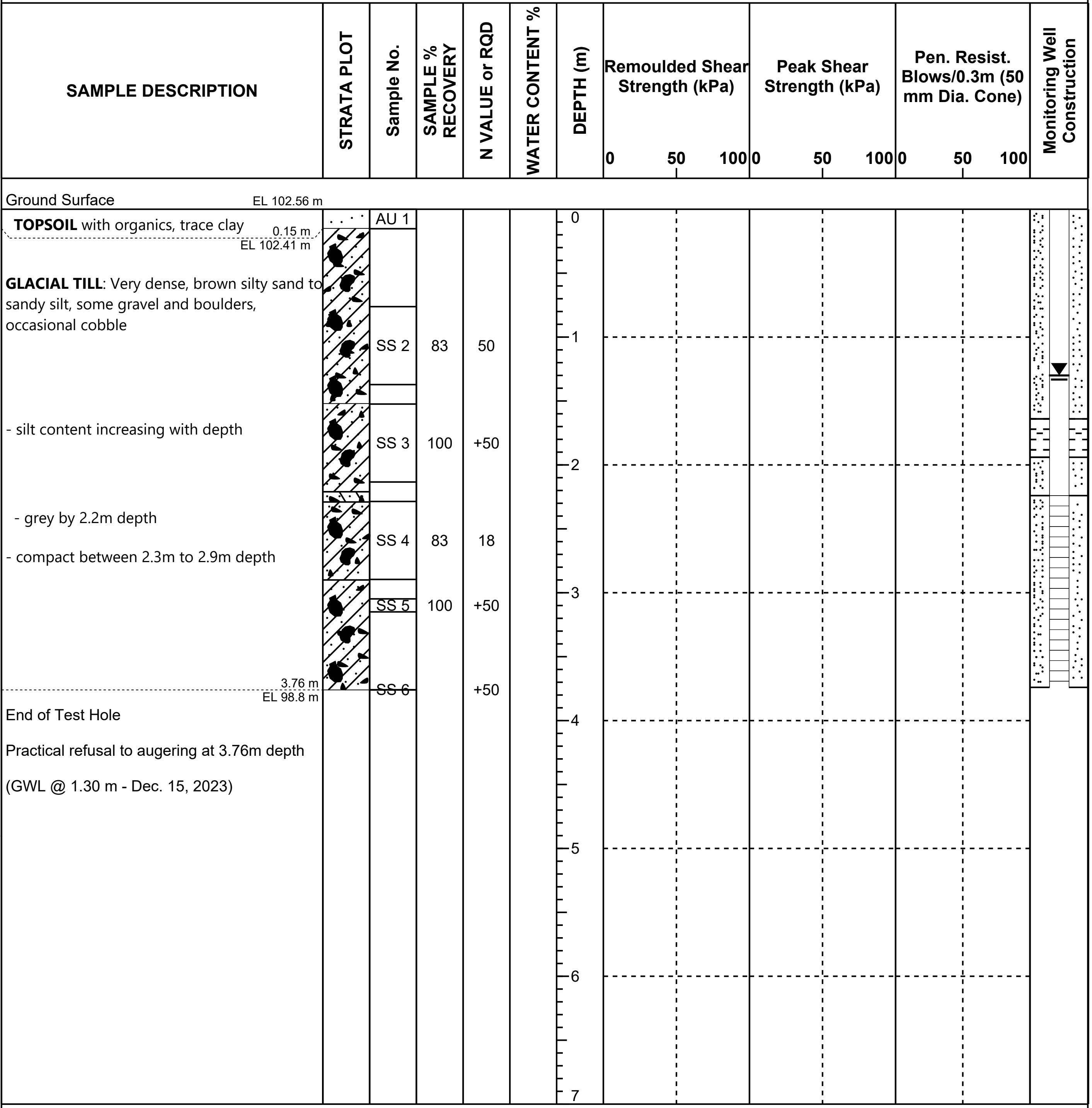
BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 11, 2023

FILE NO. **PG6912**

HOLE NO. BH 1-23



SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
Ground Surface EL 102.56 m				
TOPSOIL with organics, trace clay 0.15 m EL 102.41 m	AU 1			
GLACIAL TILL: Very dense, brown silty sand to sandy silt, some gravel and boulders, occasional cobble	SS 2	83	50	
- silt content increasing with depth	SS 3	100	+50	
- grey by 2.2m depth - compact between 2.3m to 2.9m depth	SS 4	83	18	
	SS 5	100	+50	
3.76 m EL 98.8 m	SS 6		+50	

End of Test Hole

Practical refusal to augering at 3.76m depth

(GWL @ 1.30 m - Dec. 15, 2023)

RSLog / Geotechnical Borehole - Geodetic - MW / paterson-group / admin / December 20, 2023 12:05 PM

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376358.876 **NORTHING:** 5018986.132 **ELEVATION:** 102.25

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 11, 2023

FILE NO. **PG6912**

HOLE NO. BH 2-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa)	Peak Shear Strength (kPa)	Pen. Resist. Blows/0.3m (50 mm Dia. Cone)	Monitoring Well Construction
							0 50 100	0 50 100	0 50 100	
Ground Surface EL 102.25 m										
GLACIAL TILL: Brown clayey silt with gravel and occasional cobble, trace organics 0.46 m EL 101.79 m		AU 1				0				
GLACIAL TILL: Dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble		SS 2	63	48		1				
- very dense by 1.2m depth 2.08 m EL 100.17 m		SS 3	76	+50						
End of Test Hole Practical refusal to augering at 2.08m depth						2				
						3				
						4				
						5				
						6				
						7				

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376356.813 **NORTHING:** 5018986.889 **ELEVATION:** 102.32

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

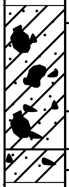
REMARKS:

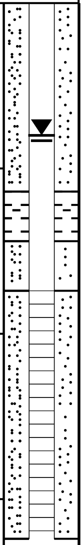
DATE: December 11, 2023

FILE NO. **PG6912**

HOLE NO. BH 2A-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Monitoring Well Construction
Ground Surface EL 102.32 m										
Overburden						0				
Augered to 2.18 m depth						1				
2.18 m EL 100.14 m						2				
GLACIAL TILL: Very dense, grey silty sand to sandy silt with gravel and boulders, occasional cobble		SS 1	86	+50						
- trace clay by 3.0m depth		SS 2	100	+50		3				
3.25 m EL 99.07 m										
End of Test Hole										
Practical refusal to augering at 3.25m depth										
(GWL @ 0.80 m - Dec. 15, 2023)						4				
						5				
						6				
						7				





SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376284.113 **NORTHING:** 5018877.194 **ELEVATION:** 105.46 m

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

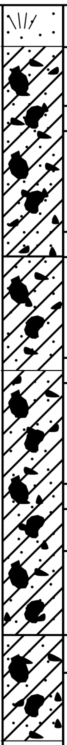
REMARKS:

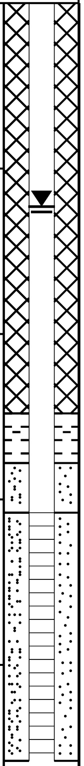
DATE: December 11, 2023

FILE NO. **PG6912**

HOLE NO. BH 3-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Piezometer Construction
Ground Surface EL 105.46 m						0				
TOPSOIL with organics 0.25 m EL 105.21 m										
GLACIAL TILL: Compact, brown silty sand to sandy silt, trace gravel		AU 1								
		SS 2	42	12		1				
- no gravel by 1.5m depth		SS 3	83	19		2				
2.21 m EL 103.25 m										
GLACIAL TILL: Very dense, grey silty sand to sandy silt with gravel and boulders, occasional cobble		SS 4	75	+50						
		SS 5	100	+50		3				
- trace clay by 3.8m depth		SS 6	78	+50		4				
4.45 m EL 101.01 m										
End of Test Hole Practical refusal to augering at 4.45m depth (GWL @ 1.23 m - Dec. 15, 2023)						5				
						6				
						7				





PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376162.967 **NORTHING:** 5018907.818 **ELEVATION:** 106.7 m

PROJECT: Proposed Development

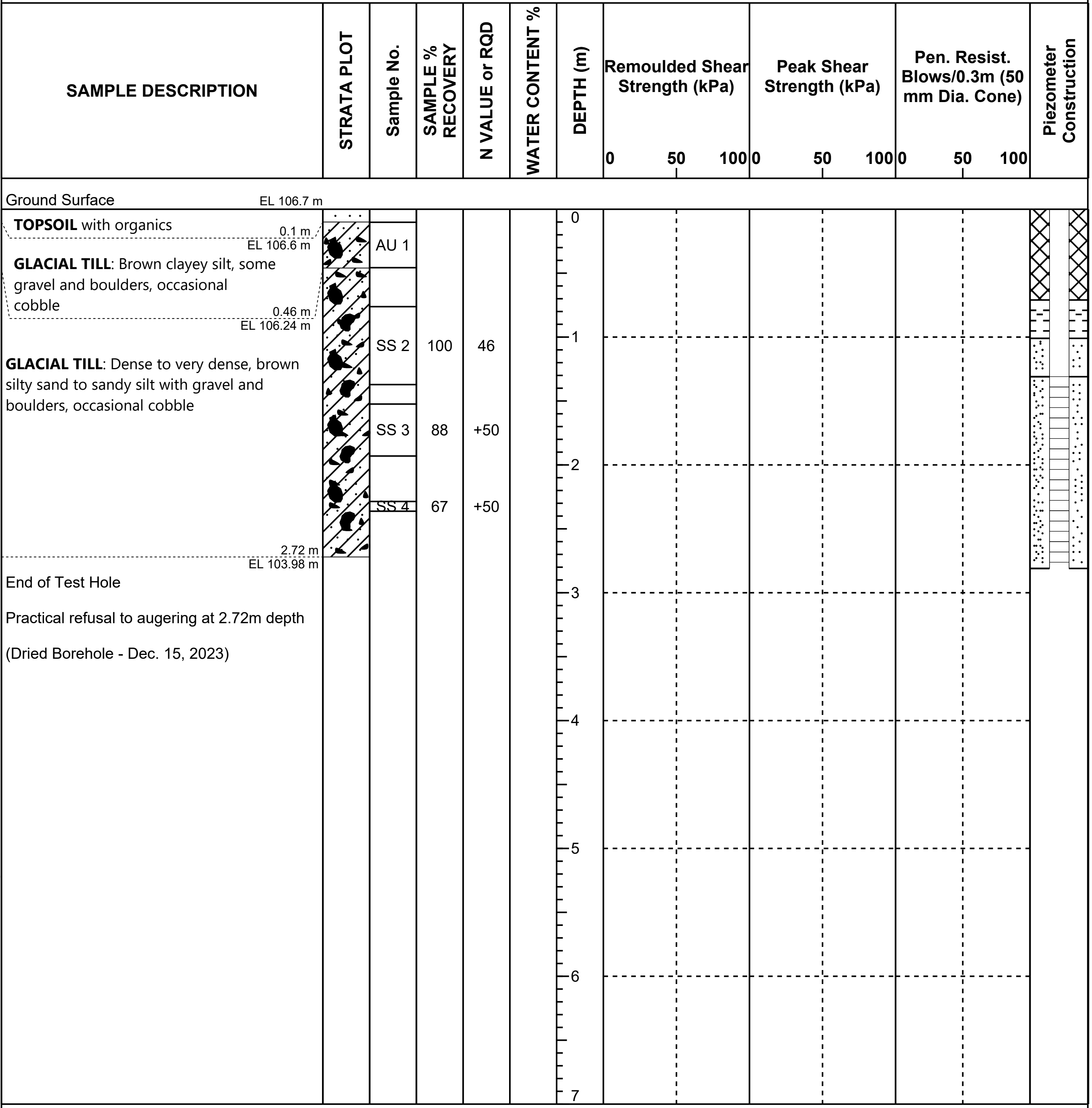
BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 11, 2023

FILE NO. **PG6912**

HOLE NO. BH 4-23



SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)
Ground Surface EL 106.7 m					0
TOPSOIL with organics 0.1 m EL 106.6 m	AU 1				
GLACIAL TILL: Brown clayey silt, some gravel and boulders, occasional cobble 0.46 m EL 106.24 m					
GLACIAL TILL: Dense to very dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble	SS 2	100	46		1
	SS 3	88	+50		
2.72 m EL 103.98 m	SS 4	67	+50		2
End of Test Hole					3
Practical refusal to augering at 2.72m depth					
(Dried Borehole - Dec. 15, 2023)					4

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376165.452 **NORTHING:** 5018810.785 **ELEVATION:** 107.88 m

PROJECT: Proposed Development

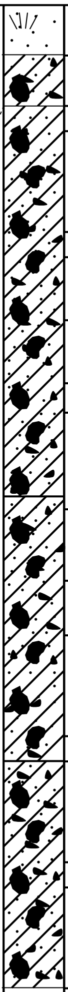
BORINGS BY: CME 55 Track-Mounted Mechanical Auger

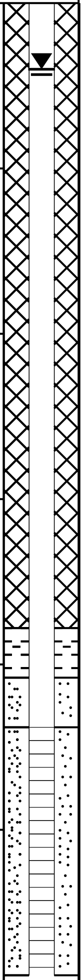
REMARKS:

DATE: December 11, 2023

FILE NO. PG6912

HOLE NO. BH 5-23





SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100
Ground Surface EL 107.88 m							
TOPSOIL with organics 0.3 m EL 107.58 m							
GLACIAL TILL: Brown clayey silt with gravel, occasional cobble 0.61 m EL 107.27 m	AU 1						
GLACIAL TILL: Dense to very dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble	SS 2	83	43				
	SS 3	100	+50				
	SS 4	100	+50				
- grey by 3.0m depth	SS 5	100	+50				
	SS 6	100	+50				
- dense by 4.6m depth	SS 7	100	46				
5.94 m EL 101.94 m	SS 8	75	34				
End of Test Hole (GWL @ 0.40 m - Dec. 15, 2023)							

DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7

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SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

PATERSON GROUP

DATUM: Geodetic **EASTING:** 376381.101 **NORTHING:** 5018941.64 **ELEVATION:** 101.53

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 6-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Monitoring Well Construction
Ground Surface EL 101.53 m										
TOPSOIL with organics 0.25 m EL 101.28 m						0				
GLACIAL TILL: Brown clayey silt, trace gravel 0.61 m EL 100.92 m		AU 1								
GLACIAL TILL: Compact, brown silty sand to sandy silt with gravel and boulders, occasional cobble		SS 2	75	19		1				
- very dense by 1.5m depth 1.88 m EL 99.65 m		SS 3	89	+50						
End of Test Hole Practical refusal to augering at 1.88m depth						2				
						3				
						4				
						5				
						6				
						7				

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376380 **NORTHING:** 5018939.186 **ELEVATION:** 101.45

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 6A-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Monitoring Well Construction
Ground Surface EL 101.45 m										
Overburden Augered to 1.98 m depth						0 1				
1.98 m EL 99.47 m										
End of Test Hole Practical refusal to augering at 1.98m depth						2 3 4 5 6 7				

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376380.758 **NORTHING:** 5018936.262 **ELEVATION:** 101.44

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 6B-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Monitoring Well Construction
Ground Surface EL 101.44 m										
Overburden						0				
Augered to 0.86 m depth 0.86 m EL 100.58 m										
End of Test Hole						1				
Practical refusal to augering at 0.86m depth										
						2				
						3				
						4				
						5				
						6				
						7				

RSLog / Geotechnical Borehole - Geodetic - MW / paterson-group / admin / December 20, 2023 12:05 PM

SOIL PROFILE AND TEST DATA
GEOTECHNICAL INVESTIGATION
4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376378.444 **NORTHING:** 5018942.565 **ELEVATION:** 101.59

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 6C-23

SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
Ground Surface EL 101.59 m				
TOPSOIL with organics 0.25 m EL 101.34 m				
GLACIAL TILL: Brown clayey silt, trace gravel and boulders 0.61 m EL 100.98 m				
GLACIAL TILL: Compacto to dense, brown silty sand with gravel, cobble and boulders - grey by 2.1m depth 2.21 m EL 99.38 m				
GLACIAL TILL: Dense to very dense, grey silty sand with gravel, cobble and boulders, trace clay	SS 1	67	43	
	SS 2	0	+50	
- boulders by 3.4m depth	RC 1	35		
4.62 m EL 96.97 m				
End of Test Hole (GWL @ 0.76 m - Dec. 15, 2023)				

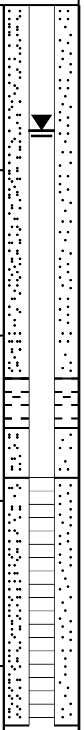
DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7

Remoulded Shear Strength (kPa): 0 50 100

Peak Shear Strength (kPa): 0 50 100

Pen. Resist. Blows/0.3m (50 mm Dia. Cone): 0 50 100

Monitoring Well Construction



PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 376042.873 **NORTHING:** 5018770.725 **ELEVATION:** 109.28 m

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

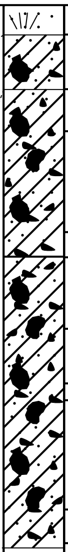
REMARKS:

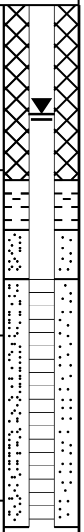
DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 7-23

SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100
Ground Surface EL 109.28 m							
TOPSOIL with organics 0.18 m EL 109.1 m							
GLACIAL TILL: Brown clayey silt with gravel 0.51 m EL 108.77 m	AU 1						
GLACIAL TILL: Dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble	SS 2	75	30				
- very dense by 1.5m depth	SS 3		+50				
	SS 4	50	+50				
3.28 m EL 106 m	SS 5	100	+50				





DEPTH (m): 0, 1, 2, 3, 4, 5, 6, 7

End of Test Hole

Practical refusal to augering at 3.28m depth

(GWL @ 0.66 m - Dec. 15, 2023)

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 375921.34 **NORTHING:** 5018806.672 **ELEVATION:** 106.96 m

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 8-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Piezometer Construction
Ground Surface EL 106.96 m										
TOPSOIL with organics 0.15 m EL 106.81 m						0				
GLACIAL TILL: Brown clayey silt with gravel 0.69 m EL 106.27 m		AU 1								
GLACIAL TILL: Very dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble, trace clay 1.24 m EL 105.72 m		SS 2	100	+50		1				
End of Test Hole Practical refusal to augering at 1.24m depth						2 3 4 5 6 7				

RSLog / Geotechnical Test Pit - Geodetic / paterson-group / admin / December 20, 2023 09:56 AM

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 375924.947 **NORTHING:** 5018803.173 **ELEVATION:** 107.22 m

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

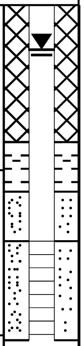
REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 8A-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Piezometer Construction
Ground Surface EL 107.22 m										
Overburden Augered to 0.69 m depth						0				
0.69 m EL 106.54 m										
GLACIAL TILL: Very dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble						1				
		SS 1	100	+50						
2.18 m EL 105.04 m						2				
End of Test Hole Practical refusal to augering at 2.18m depth (GWL @ 0.27 m - Dec. 15, 2023)						3				
						4				
						5				
						6				
						7				



PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 375914.663 **NORTHING:** 5018651.441 **ELEVATION:** 106.87 m

PROJECT: Proposed Development

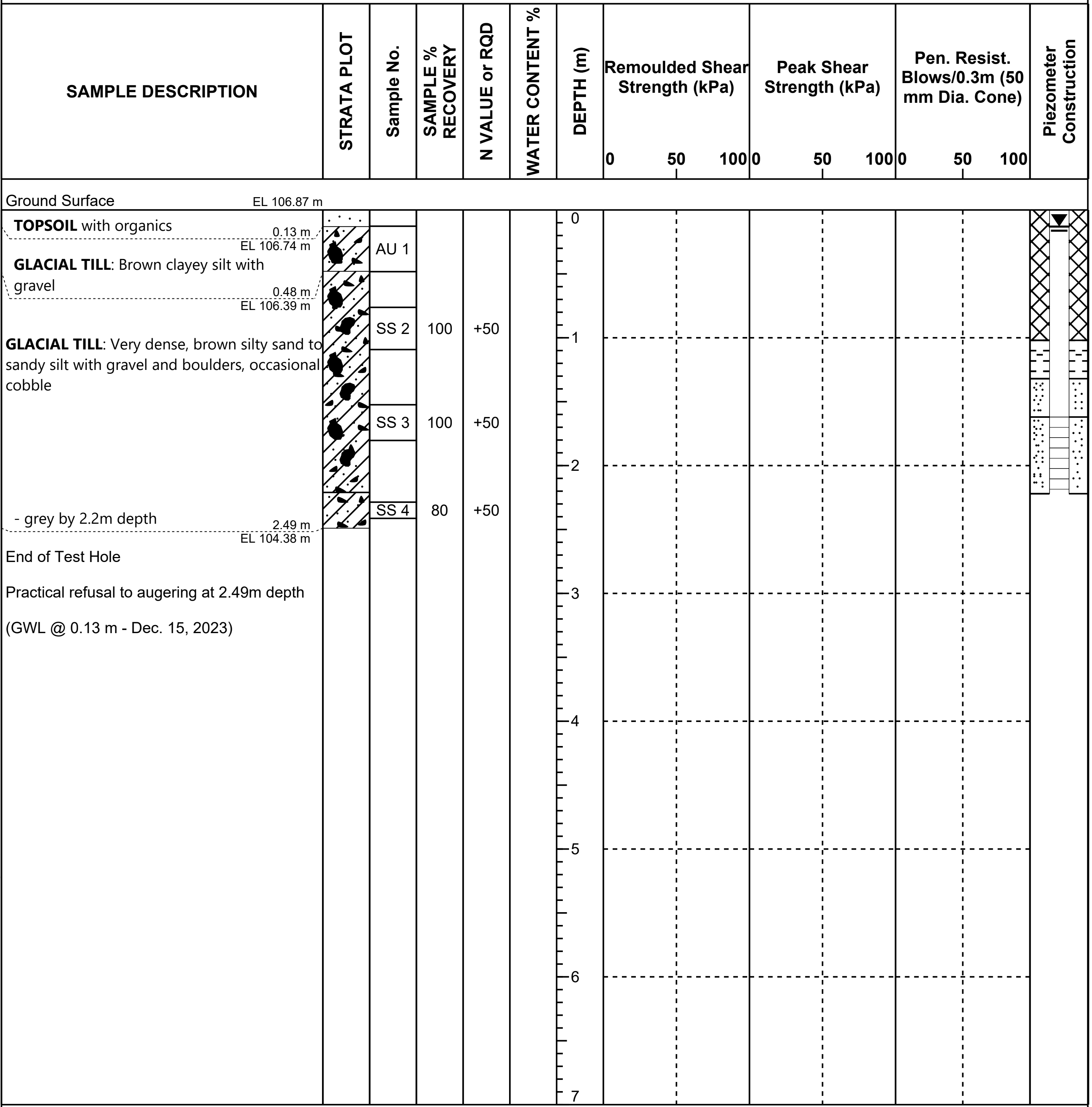
BORINGS BY: CME 55 Track-Mounted Mechanical Auger

REMARKS:

DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH 9-23



SAMPLE DESCRIPTION	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %
Ground Surface EL 106.87 m				
TOPSOIL with organics 0.13 m EL 106.74 m	AU 1			
GLACIAL TILL: Brown clayey silt with gravel 0.48 m EL 106.39 m				
GLACIAL TILL: Very dense, brown silty sand to sandy silt with gravel and boulders, occasional cobble	SS 2	100	+50	
	SS 3	100	+50	
- grey by 2.2m depth 2.49 m EL 104.38 m	SS 4	80	+50	

End of Test Hole

Practical refusal to augering at 2.49m depth

(GWL @ 0.13 m - Dec. 15, 2023)

PATERSON GROUP

SOIL PROFILE AND TEST DATA

GEOTECHNICAL INVESTIGATION

4850 Bank Street, Ottawa, Ontario

DATUM: Geodetic **EASTING:** 375787.732 **NORTHING:** 5018720.138 **ELEVATION:** 103.35

PROJECT: Proposed Development

BORINGS BY: CME 55 Track-Mounted Mechanical Auger

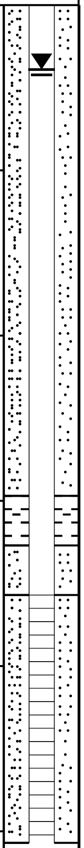
REMARKS:

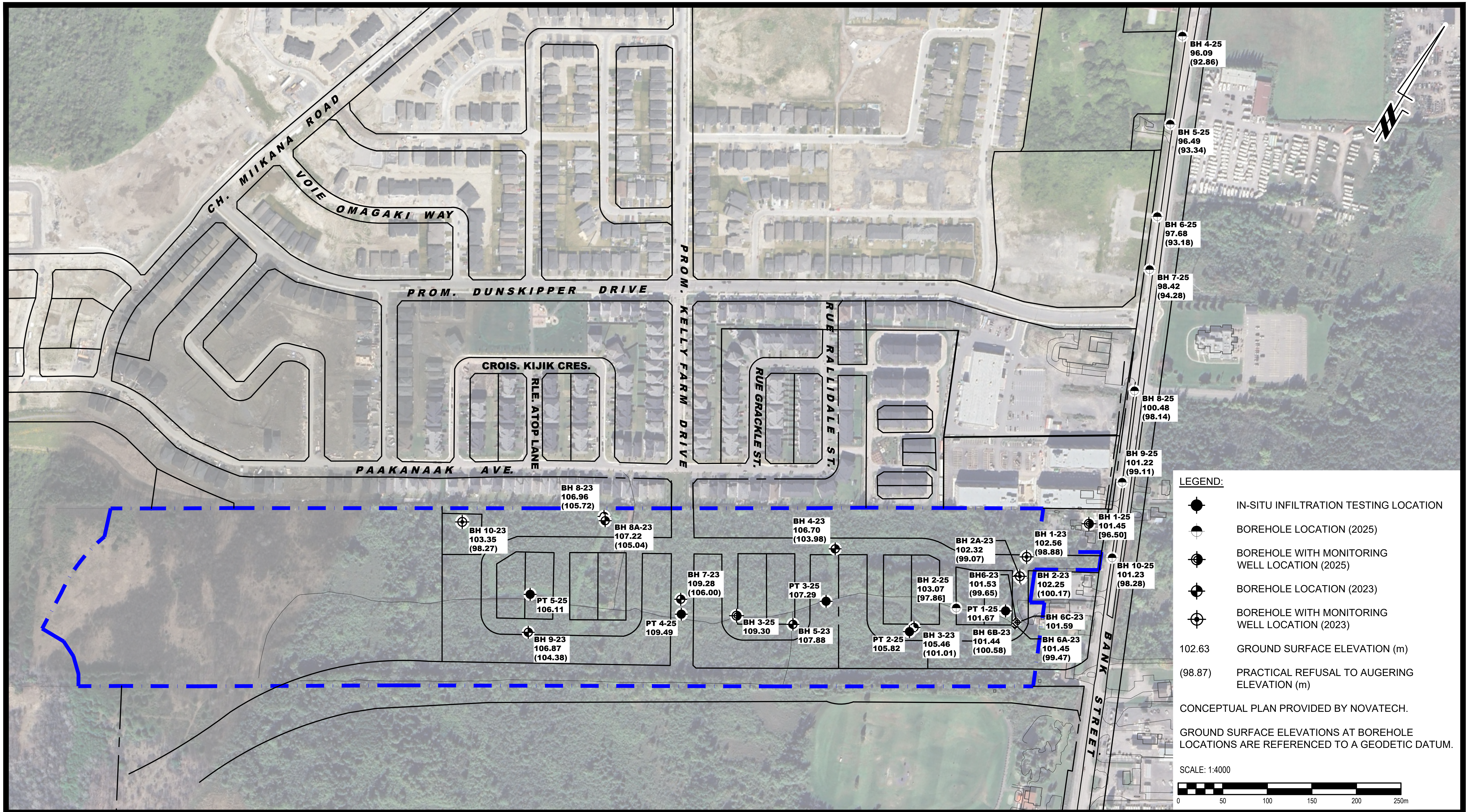
DATE: December 13, 2023

FILE NO. **PG6912**

HOLE NO. BH10-23

SAMPLE DESCRIPTION	STRATA PLOT	Sample No.	SAMPLE % RECOVERY	N VALUE or RQD	WATER CONTENT %	DEPTH (m)	Remoulded Shear Strength (kPa) 0 50 100	Peak Shear Strength (kPa) 0 50 100	Pen. Resist. Blows/0.3m (50 mm Dia. Cone) 0 50 100	Monitoring Well Construction
Ground Surface EL 103.35 m										
TOPSOIL with organics 0.33 m EL 103.02 m						0				
GLACIAL TILL: Compact, brown silty sand to sandy silt with gravel, occasional cobble		AU 1								
		SS 2	46	10		1				
2.21 m EL 101.14 m		SS 3	67	24		2				
GLACIAL TILL: Dense to very dense, grey silty sand with gravel, cobble, and boulders		SS 4A SS 4B	60	+50						
		SS 5	88	48		3				
		SS 6	63	37		4				
5.08 m EL 98.27 m		SS 7	58	+50		5				
End of Test Hole Practical refusal to augering on inferred boulder or bedrock at 5.08m depth (GWL @ 0.39 m - Dec. 15, 2023)						6 7				





LEGEND:

●	IN-SITU INFILTRATION TESTING LOCATION
◓	BOREHOLE LOCATION (2025)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (2025)
⊕	BOREHOLE LOCATION (2023)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (2023)
102.63	GROUND SURFACE ELEVATION (m)
(98.87)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)

CONCEPTUAL PLAN PROVIDED BY NOVATECH.

GROUND SURFACE ELEVATIONS AT BOREHOLE LOCATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:4000

PATERSON GROUP

9 AURIGA DRIVE
OTTAWA, ON
K2E 7T9
TEL: (613) 226-7381

NO.	REVISIONS	DATE	INITIAL
2	ADDED 2025 IN-SITU INFILTRATION TESTING TO PLAN	08/08/2025	ZB
1	ADDED 2025 BOREHOLES BH 1-25 TO BH 10-25	09/07/2025	PB

REGIONAL GROUP
GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT
4850 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:4000	Date: 12/2023
Drawn by: GK	Report No.: PG6912-1
Checked by: DR	Dwg. No.: PG6912-1
Approved by: SD	Revision No.: 2

11x17

p:\autocad drawings\geotechnical\pg69xx\pg6912\pg6912-1 test hole location plan (rev.02).dwg

APPENDIX 2

TABLE 6 - MONTHLY WATER BALANCE FOR SOIL WITH 75 mm WATER HOLDING CAPACITY AT THE OTTAWA INTERNATIONAL AIRPORT

TABLE 7 - MONTHLY WATER BALANCE FOR SOIL WITH 300 mm WATER HOLDING CAPACITY AT THE OTTAWA INTERNATIONAL AIRPORT

TABLE 8 - PRE-DEVELOPMENT ANNUAL WATER BUDGET

TABLE 9 - POST-DEVELOPMENT ANNUAL WATER BUDGET

Table 6 - Monthly Water Balance for Soil With 75 mm Water Holding Capacity at the Ottawa International Airport				
Month	**Temperature (℃)**	**Total Precipitation (mm)**	**Actual Evapotranspiration (mm)**	**Water Surplus (mm)**
January	-10.6	62	0	25
February	-9.0	56	1	26
March	-2.8	65	6	103
April	5.7	73	31	110
May	13.1	76	80	14
June	18.3	85	107	4
July	20.9	88	104	3
August	19.7	85	84	1
September	14.8	82	65	3
October	8.3	76	36	14
November	1.3	77	10	38
December	-6.8	79	1	37
Annual	**6**	**904**	**525**	**378**

Table 7 - Monthly Water Balance for Soil With 300 mm Water Holding Capacity at the Ottawa International Airport				
Month	**Temperature (℃)**	**Total Precipitation (mm)**	**Actual Evapotranspiration (mm)**	**Water Surplus (mm)**
January	-10.6	62	0	17
February	-9.0	56	1	21
March	-2.8	65	6	91
April	5.7	73	31	105
May	13.1	76	80	14
June	18.3	85	116	4
July	20.9	89	135	3
August	19.7	84	114	1
September	14.8	82	73	2
October	8.3	76	37	6
November	1.3	77	10	16
December	-6.8	80	1	18
Annual	**6**	**904**	**605**	**298**

Table 8 - Pre-Development Annual Water Budget Calculations

Land Use Unit	Area (m^2)	Water Surplus (mm)	Topography Factor	Soil Factor	Vegetation Factor	Infiltration Factor	Runoff Factor	Total Infiltration (mm/year)	Total Infiltration (L/year)	Total Runoff (mm/year)	Total Runoff (L/year)
Mature Forest (Fine Sandy Loam)	136,927	298	0.1	0.3	0.2	0.6	0.4	178.8	24,482,548	119.2	16,321,698
Total	**136,927**								**24,482,548**		**16,321,698**

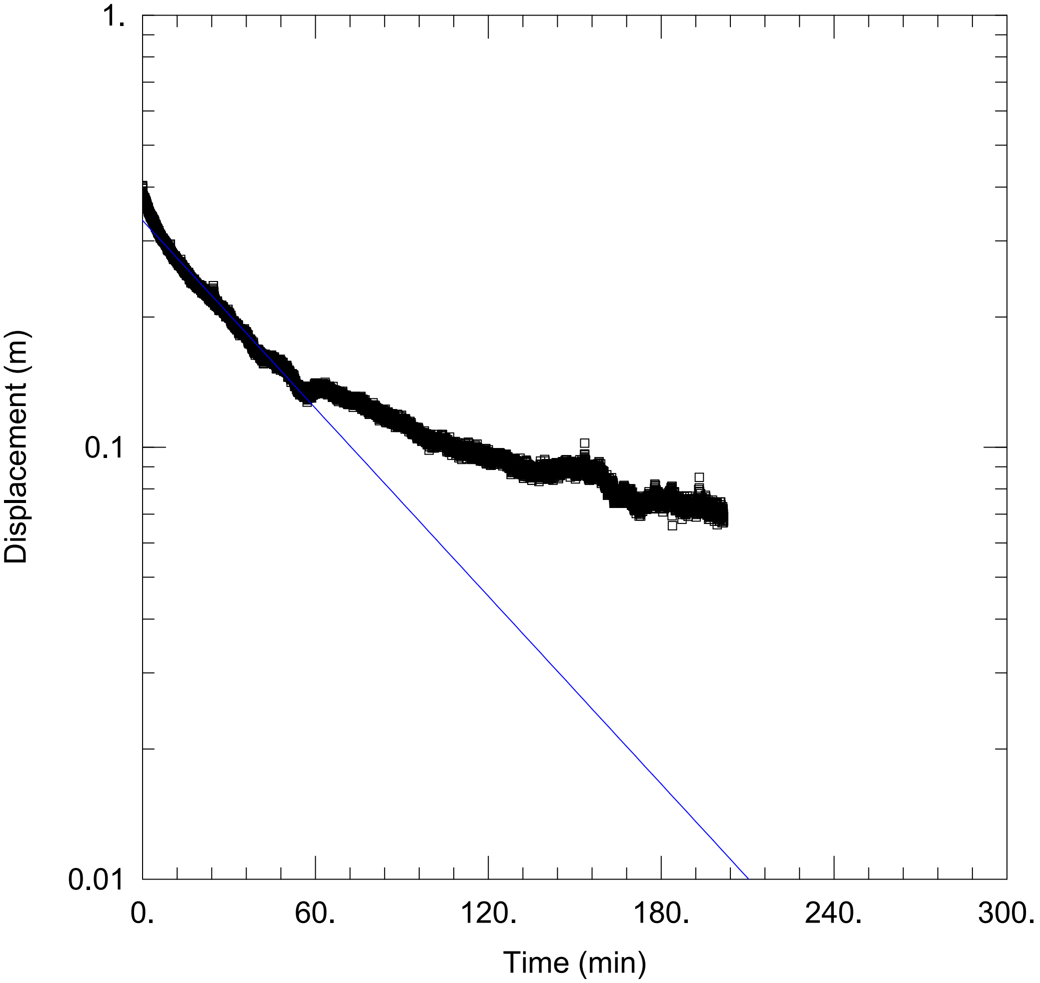
Table 9 - Post-Development Annual Water Budget Calculations

Land Use Unit	Area (m^2)	Water Surplus (mm)	Topography Factor	Soil Factor	Vegetation Factor	Infiltration Factor	Runoff Factor	Total Infiltration (mm/year)	Total Infiltration (L/year)	Total Runoff (mm/year)	Total Runoff (L/year)
Impervious Surfaces	88,226	759	0	0	0	0	1	0	0	759.0	66,963,619
Urban Lawn/Shallow Rooted Crops (Fine Sandy Loam)	43,601	378	0.15	0.3	0.1	0.55	0.45	207.9	9,064,556	170.1	7,416,455
Stormwater Management Pond	5,100	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A	N/A
Totals	**136,927**								**9,064,556**		**74,380,074**
Difference (L/year)									**-15,417,992**		**58,058,376**
Percentage Variation									**-63%**		**356%**

APPENDIX 3

HYDRAULIC CONDUCTIVITY RESULTS - FALLING AND RISING HEAD TESTS

SAMPLE CALCULATIONS - DUPUIT FORCHHEIMER



BH1-23 - FALLING HEAD 1 OF 1

PROJECT INFORMATION

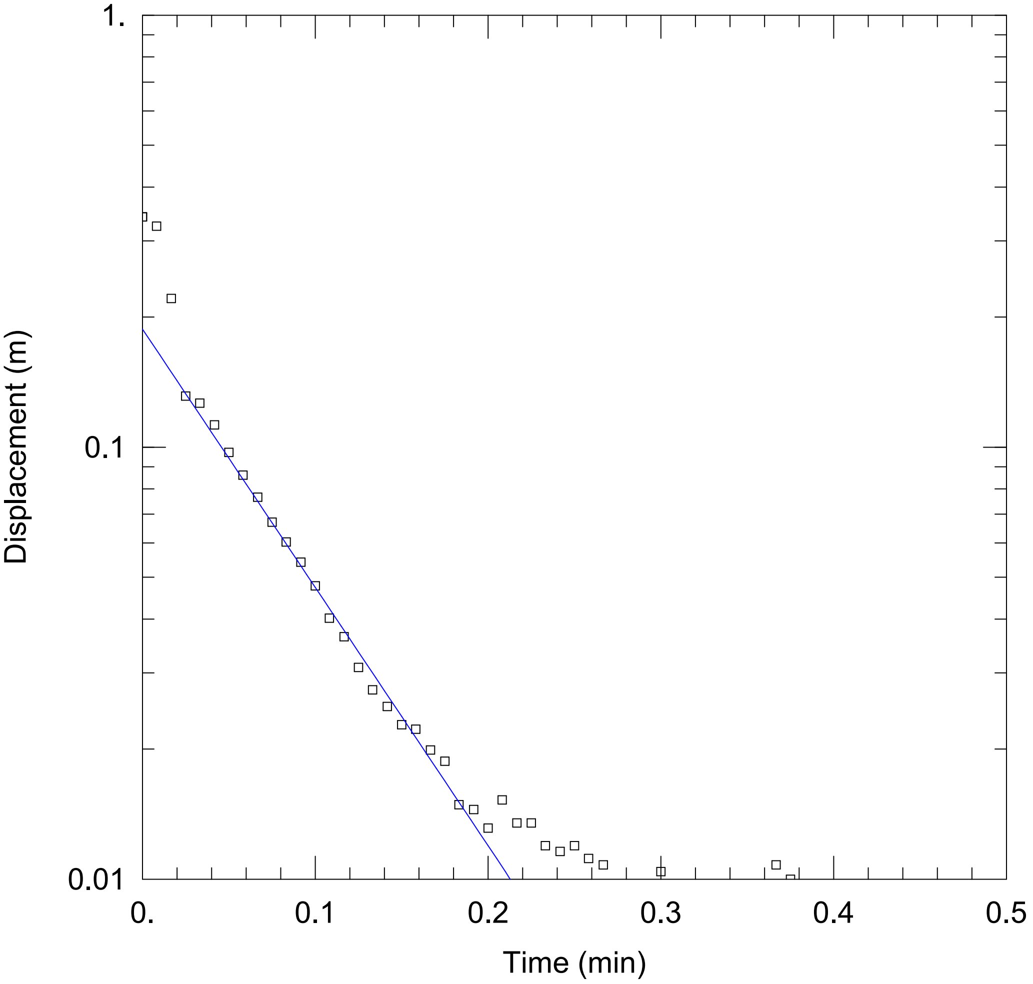
Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH1-23
Test Date: July 3, 2025

WELL DATA (BH1-23)

Initial Displacement: 0.403 m
Total Well Penetration Depth: 1.28 m
Casing Radius: 0.0254 m
Static Water Column Height: 1.28 m
Screen Length: 1.28 m
Well Radius: 0.1045 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev
K = 3.714E-7 m/sec
y0 = 0.3347 m



BH1-25 - FALLING HEAD 1 OF 1

PROJECT INFORMATION

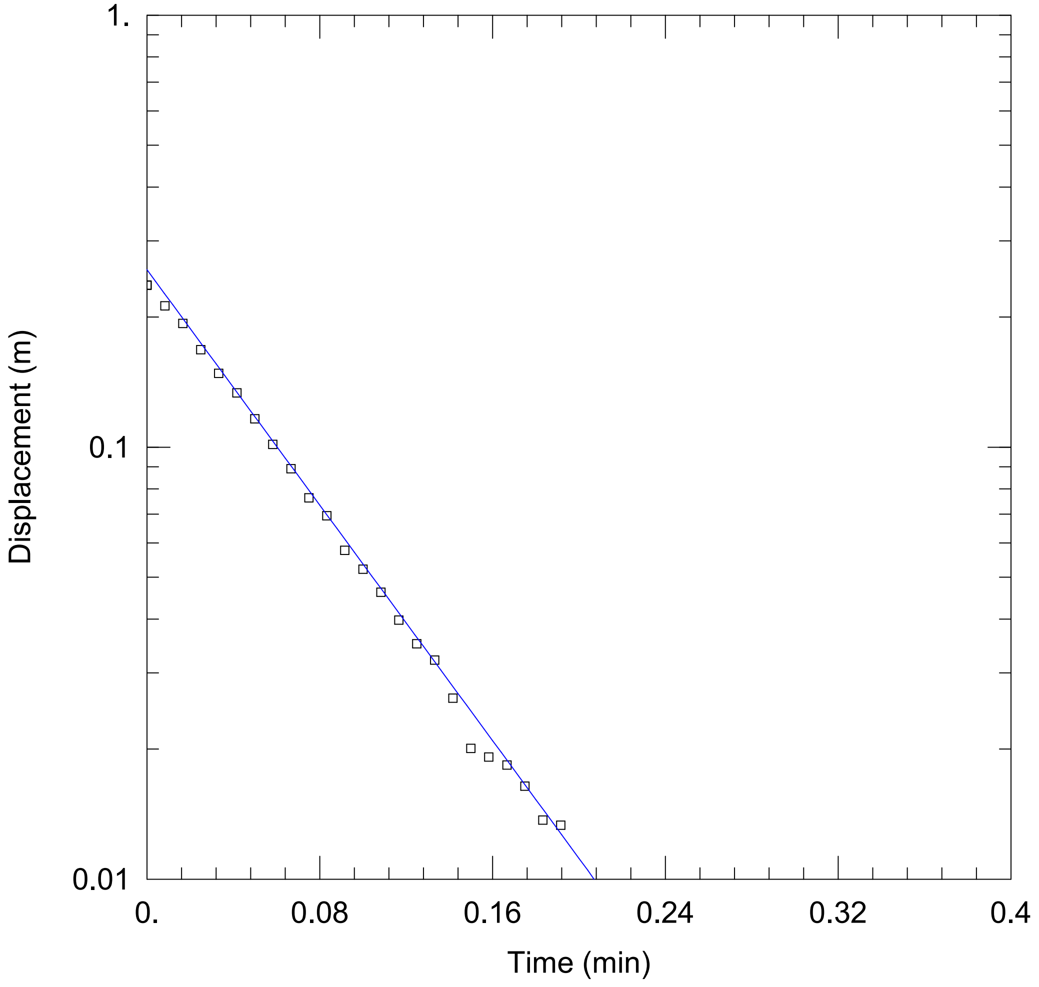
Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH1-25
Test Date: July 3, 2025

WELL DATA (BH1-25)

Initial Displacement: 0.341 m
Total Well Penetration Depth: 6.54 m
Casing Radius: 0.01588 m
Static Water Column Height: 6.54 m
Screen Length: 1.524 m
Well Radius: 0.0381 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev
K = 0.000105 m/sec
y0 = 0.1876 m



BH1-25 - RISING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH1-25
Test Date: July 3, 2025

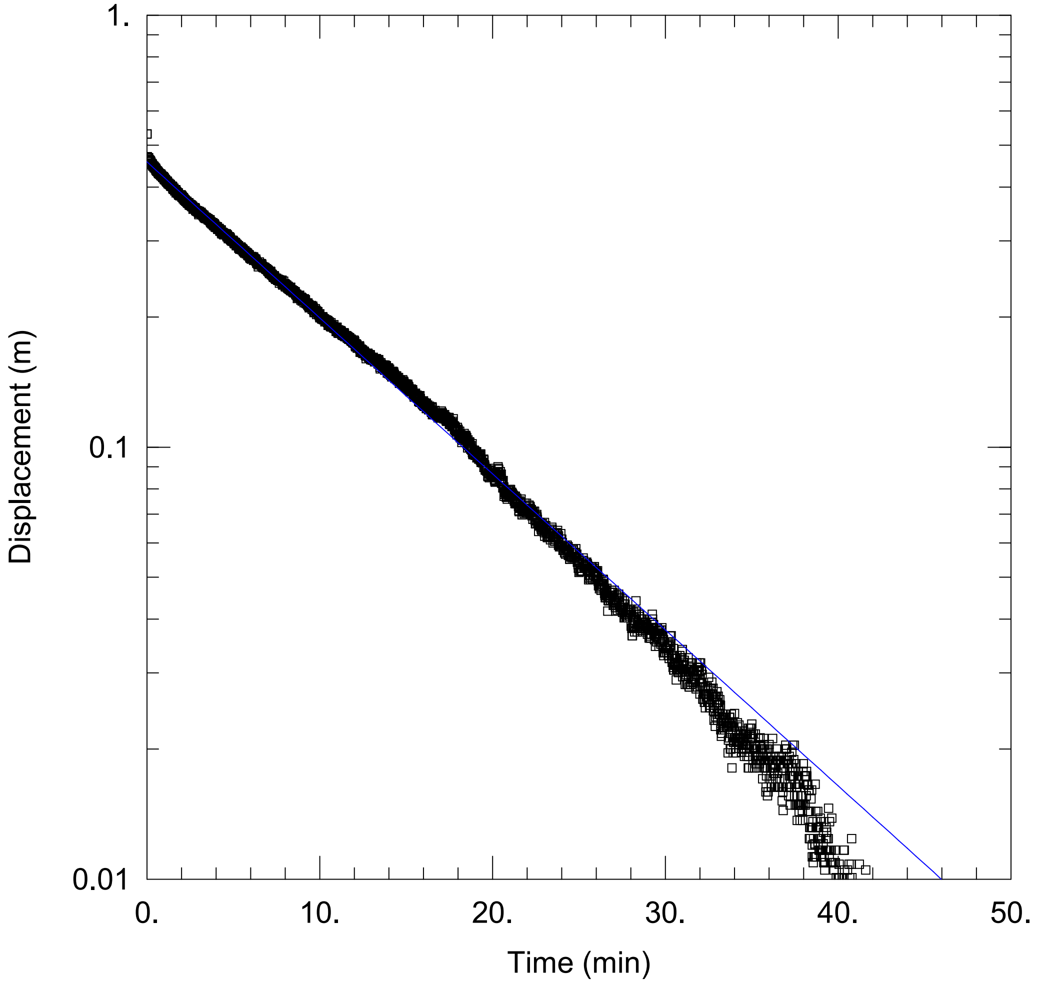
WELL DATA (BH1-25)

Initial Displacement: 0.237 m
Total Well Penetration Depth: 6.54 m
Casing Radius: 0.01588 m
Static Water Column Height: 6.54 m
Screen Length: 1.524 m
Well Radius: 0.0381 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 0.0001196 m/sec
y0 = 0.2575 m



BH3-25 - FALLING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH3-25
Test Date: July 3, 2025

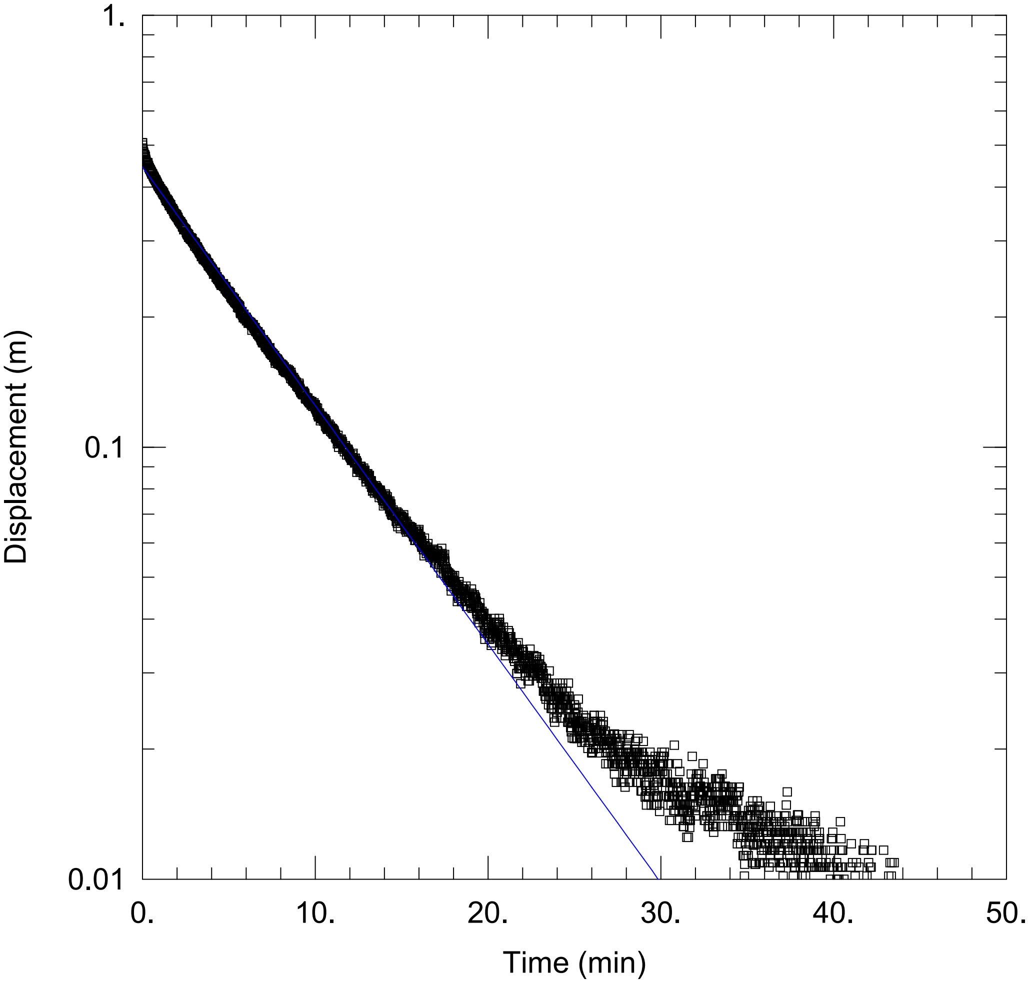
WELL DATA (BH3-25)

Initial Displacement: 0.531 m
Total Well Penetration Depth: 4.17 m
Casing Radius: 0.0254 m
Static Water Column Height: 4.17 m
Screen Length: 1.524 m
Well Radius: 0.1045 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 1.327E-6 m/sec
y0 = 0.4572 m



BH3-25 - RISING HEAD 1 OF 1

PROJECT INFORMATION

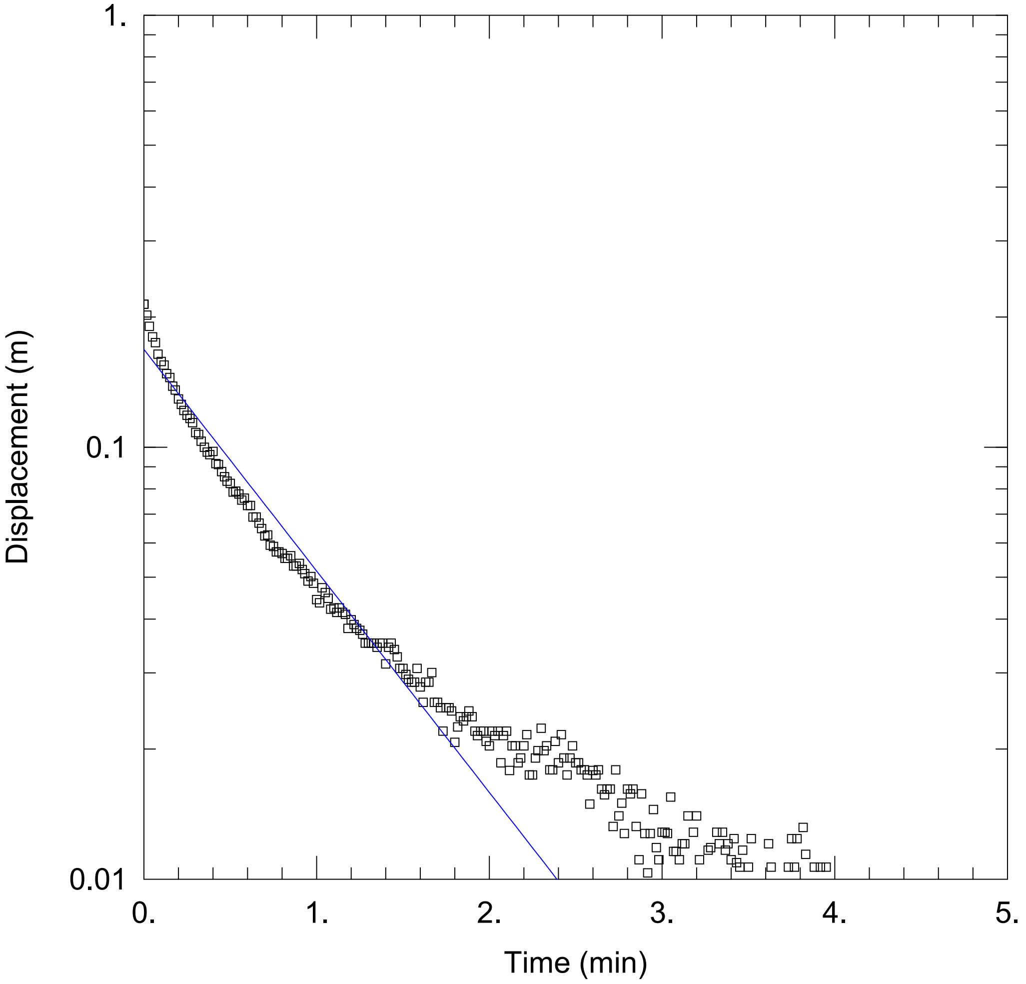
Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH3-25
Test Date: July 3, 2025

WELL DATA (BH3-25)

Initial Displacement: 0.506 m
Total Well Penetration Depth: 4.17 m
Casing Radius: 0.0254 m
Static Water Column Height: 4.16 m
Screen Length: 1.524 m
Well Radius: 0.1045 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev
K = 2.029E-6 m/sec
y0 = 0.4448 m



BH6C-23 - FALLING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH6C-23
Test Date: July 3, 2025

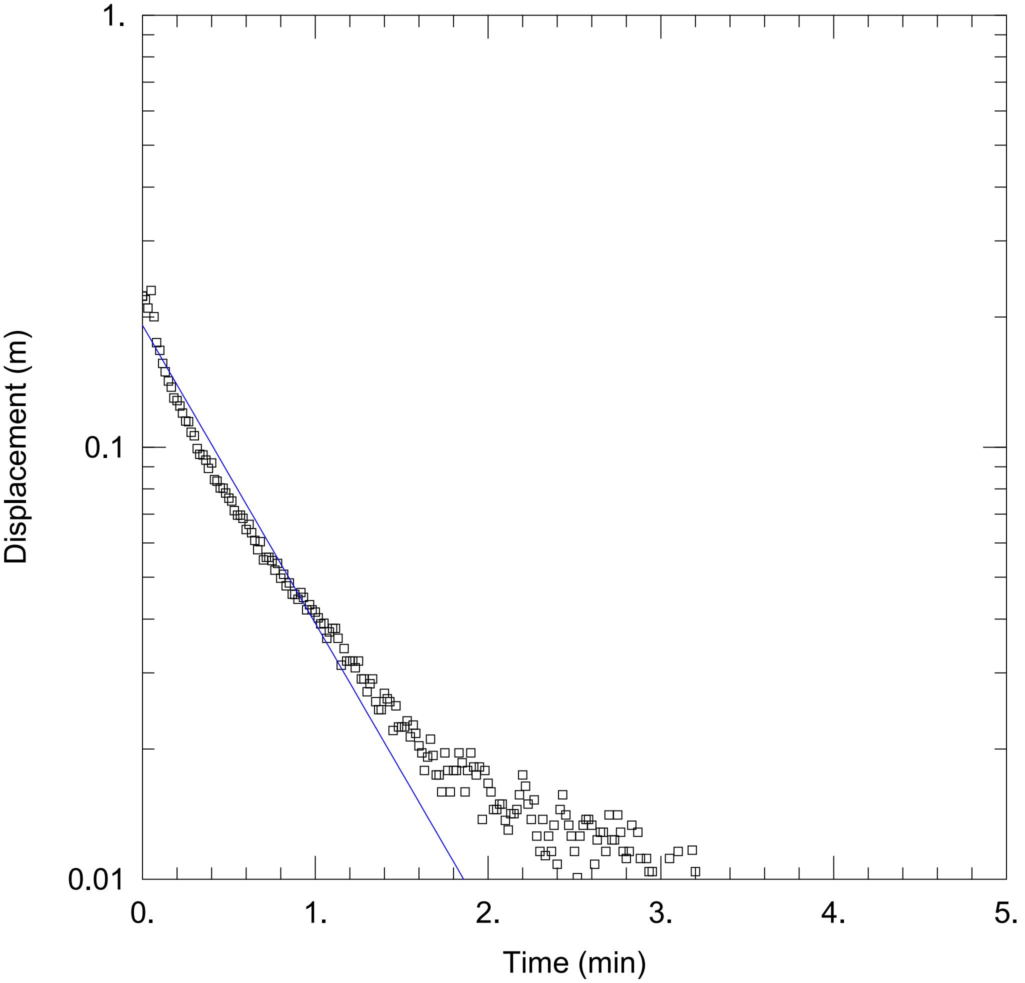
WELL DATA (BH6C-23)

Initial Displacement: 0.214 m
Total Well Penetration Depth: 3.41 m
Casing Radius: 0.01588 m
Static Water Column Height: 3.41 m
Screen Length: 1.524 m
Well Radius: 0.0381 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 9.002E-6 m/sec
y0 = 0.1683 m



BH6C-23 - RISING HEAD 1 OF 1

PROJECT INFORMATION

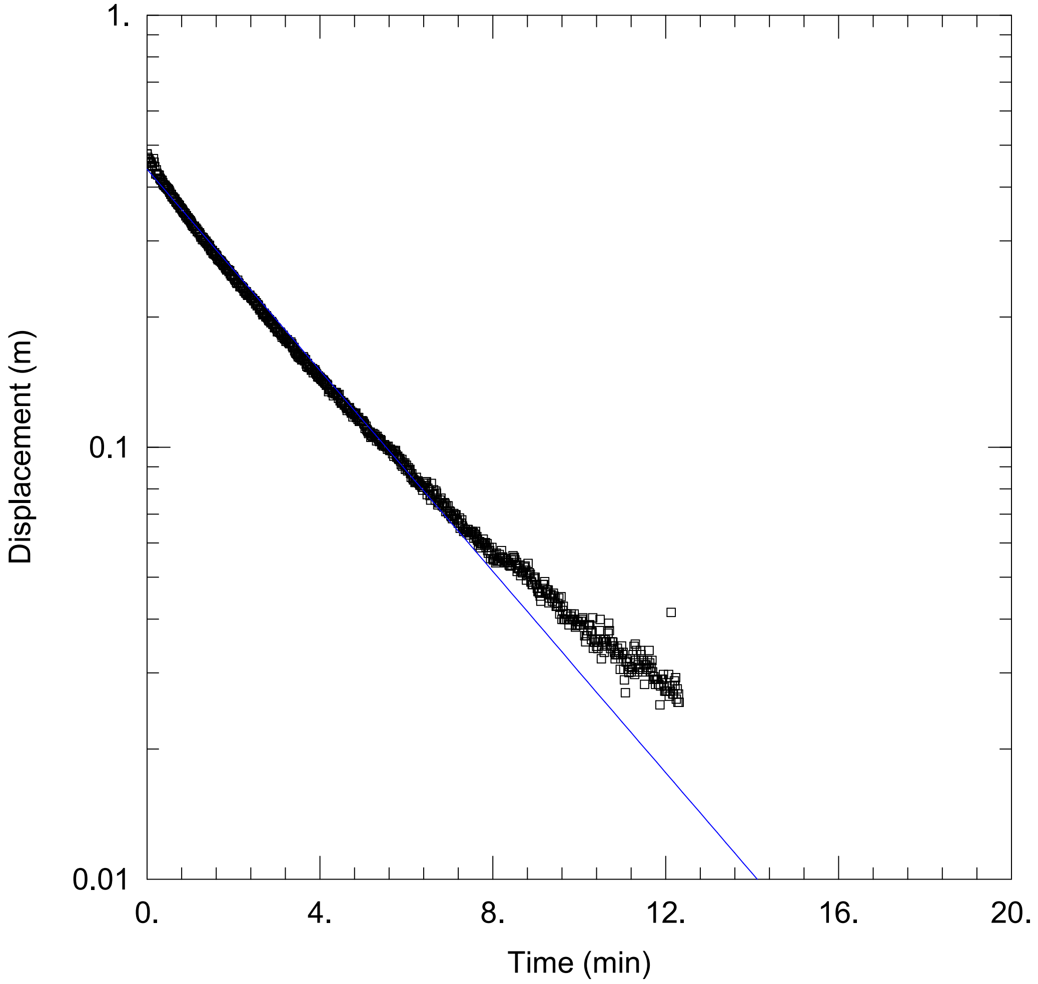
Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH6C-23
Test Date: July 3, 2025

WELL DATA (BH6C-23)

Initial Displacement: 0.224 m
Total Well Penetration Depth: 3.41 m
Casing Radius: 0.01588 m
Static Water Column Height: 3.41 m
Screen Length: 1.524 m
Well Radius: 0.0381 m

SOLUTION

Aquifer Model: Unconfined
K = 1.211E-5 m/sec
Solution Method: Hvorslev
y0 = 0.1914 m



BH10-23 - FALLING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH10-23
Test Date: July 3, 2025

WELL DATA (BH10-23)

Initial Displacement: 0.477 m
Total Well Penetration Depth: 3.84 m
Casing Radius: 0.0254 m

Static Water Column Height: 3.84 m
Screen Length: 1.524 m
Well Radius: 0.1045 m

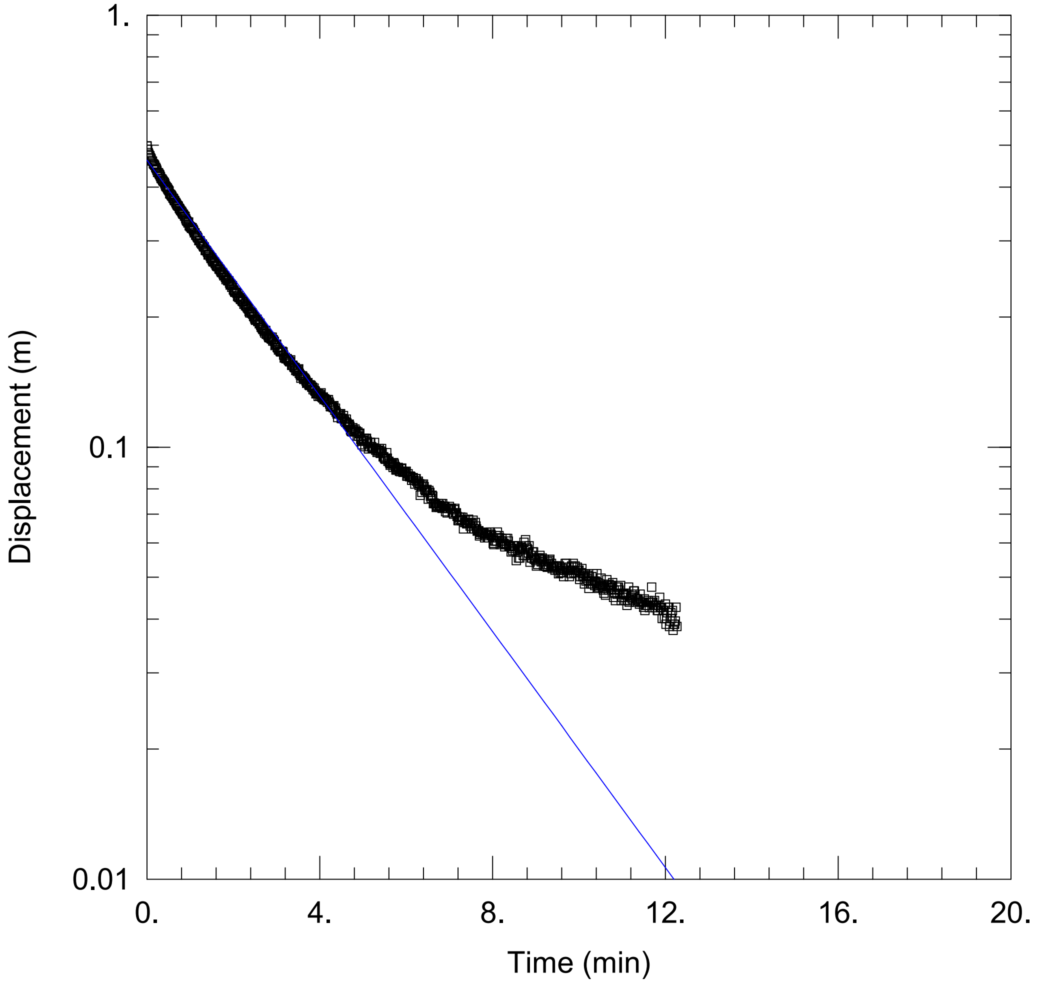
SOLUTION

Aquifer Model: Unconfined

Solution Method: Hvorslev

K = 4.275E-6 m/sec

y0 = 0.4396 m



BH10-23 - RISING HEAD 1 OF 1

PROJECT INFORMATION

Company: Paterson Group Inc.
Client: Regional Group
Project: PH5087
Location: 4850 Bank Street
Test Well: BH10-23
Test Date: July 3, 2025

WELL DATA (BH10-23)

Initial Displacement: 0.498 m
Total Well Penetration Depth: 3.84 m
Casing Radius: 0.0254 m
Static Water Column Height: 3.84 m
Screen Length: 1.524 m
Well Radius: 0.1045 m

SOLUTION

Aquifer Model: Unconfined
Solution Method: Hvorslev

K = 5.011E-6 m/sec
y0 = 0.4612 m

Estimated Groundwater Inflow

Regional Group - 4850 Bank Street - Building Excavation

Dupuit-Forchheimer Equation $Q= \pi K((h_0^2-h_p^2)/\ln(R/r))$

Equivalent Radius of Excavation = A+B=Pi*r

K (m/sec) = 3.03E-06
h0 (m) = 4
hp (m) = 2
r (m) = 17.35

Excavation Width (A) = 26.5 m
Excavation Length (B) = 28 m
Perimeter Length = 109 m
Equivalent Radius (r) = 17.35 m

R	Distance to edge of excavation (D)
18.35	1.00
20.35	3.00
22.35	5.00
24.35	7.00
26.35	**9.00**
28.35	**11.00**
30.35	13.00
32.35	15.00
34.35	17.00
36.35	19.00
38.35	21.00
40.35	23.00
42.35	25.00
44.35	27.00
46.35	29.00
48.35	31.00
50.35	33.00
52.35	35.00
54.35	37.00
56.35	39.00
58.35	41.00
60.35	43.00
62.35	45.00
64.35	47.00
66.35	49.00
68.35	51.00
70.35	53.00
72.35	55.00
74.35	57.00
76.35	59.00

Q (m^3/s)	Q (m^3/day)	Q (L/day)
0.0020	176	176,101
0.0007	62	61,874
0.0005	39	38,969
0.0003	29	29,115
0.0003	**24**	**23,615**
0.0002	**20**	**20,097**
0.0002	18	17,647
0.0002	16	15,840
0.0002	14	14,448
0.0002	13	13,343
0.0001	12	12,442
0.0001	12	11,693
0.0001	11	11,059
0.0001	11	10,515
0.0001	10	10,043
0.0001	10	9,629
0.0001	9	9,263
0.0001	9	8,936
0.0001	9	8,643
0.0001	8	8,378
0.0001	8	8,137
0.0001	8	7,917
0.0001	8	7,715
0.0001	8	7,529
0.0001	7	7,357
0.0001	7	7,198
0.0001	7	7,050
0.0001	7	6,911
0.0001	7	6,782
0.0001	7	6,660

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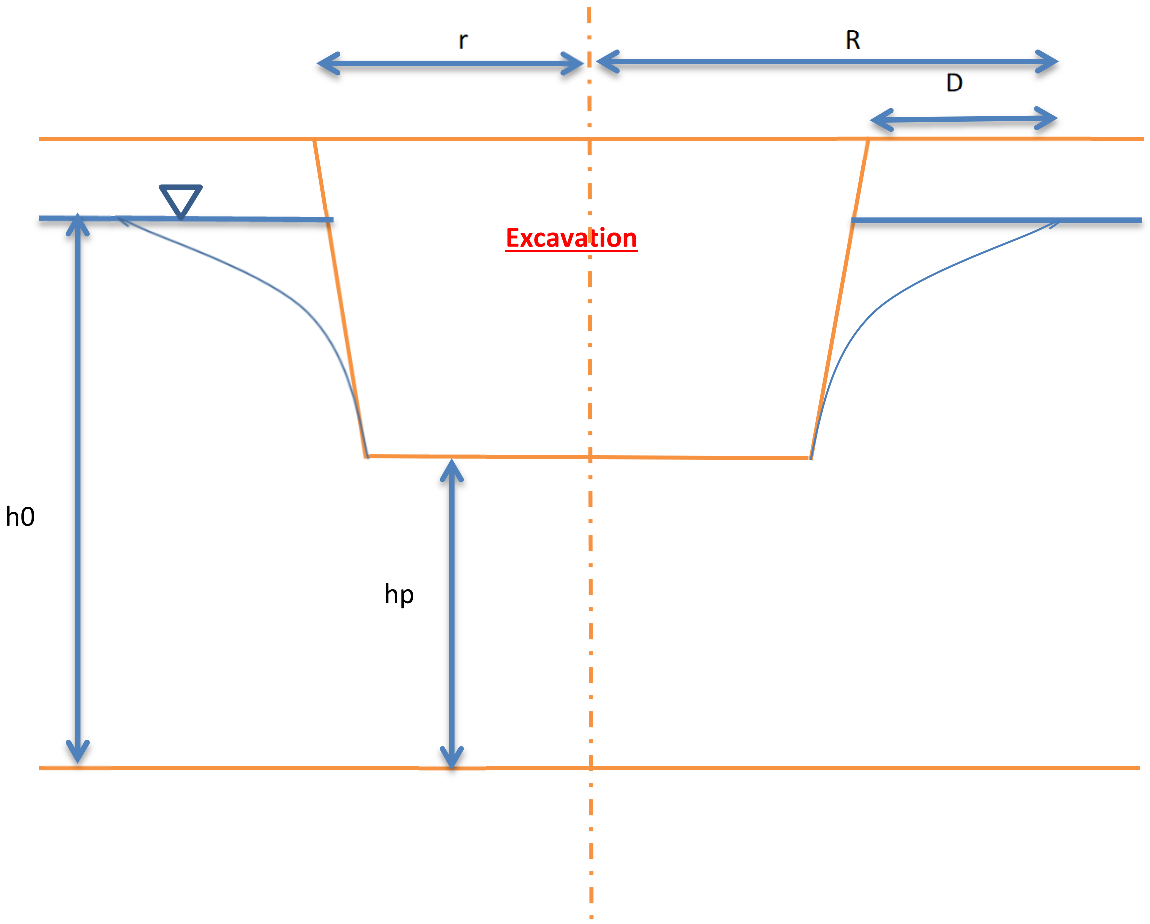
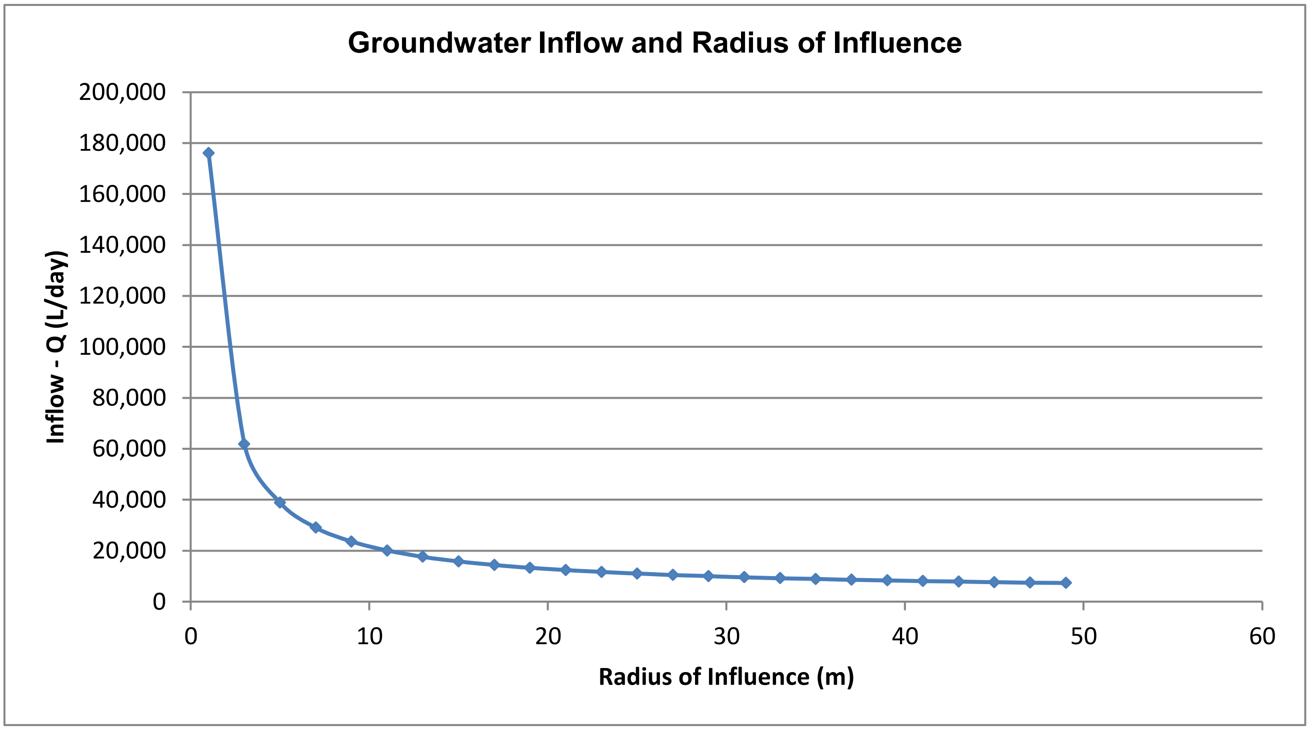

r
R
D
Excavation
h0
hp


Groundwater Inflow and Radius of Influence
Inflow - Q (L/day)
200,000
180,000
160,000
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
0
10
20
30
40
50
60
Radius of Influence (m)

File: PH5087

Estimated Groundwater Inflow

Regional Group - 4850 Bank Street - Servicing Excavation

Dupuit-Forchheimer Equation $Q= \pi K((h_0^2-h_p^2)/\ln(R/r))$

Equivalent Radius of Excavation = A+B=Pi*r

K (m/sec) = 3.03E-06
h0 (m) = 4
hp (m) = 0
r (m) = 9.55

Excavation Width (X) = 5 m
Excavation Length (Y) = 25 m
Perimeter Length = 60 m
Equivalent Radius (r) = 9.55 m

R	Distance to edge of excavation (D)
14.55	5.00
16.55	7.00
18.55	9.00
20.55	11.00
22.55	13.00
24.55	15.00
26.55	17.00
28.55	**19.00**
30.55	**21.00**
32.55	23.00
34.55	25.00
36.55	27.00
38.55	29.00
40.55	31.00
42.55	33.00
44.55	35.00
46.55	37.00
48.55	39.00
50.55	41.00
52.55	43.00
54.55	45.00
56.55	47.00
58.55	49.00
60.55	51.00
62.55	53.00
64.55	55.00
66.55	57.00
68.55	59.00
70.55	61.00
72.55	63.00

Q (m^3/s)	Q (m^3/day)	Q (L/day)
0.0004	31	31,251
0.0003	24	23,931
0.0002	20	19,819
0.0002	17	17,171
0.0002	15	15,315
0.0002	14	13,937
0.0001	13	12,869
0.0001	**12**	**12,016**
0.0001	**11**	**11,316**
0.0001	11	10,731
0.0001	10	10,233
0.0001	10	9,804
0.0001	9	9,430
0.0001	9	9,100
0.0001	9	8,807
0.0001	9	8,544
0.0001	8	8,307
0.0001	8	8,092
0.0001	8	7,896
0.0001	8	7,717
0.0001	8	7,551
0.0001	7	7,398
0.0001	7	7,257
0.0001	7	7,125
0.0001	7	7,001
0.0001	7	6,886
0.0001	7	6,778
0.0001	7	6,676
0.0001	7	6,580
0.0001	6	6,489

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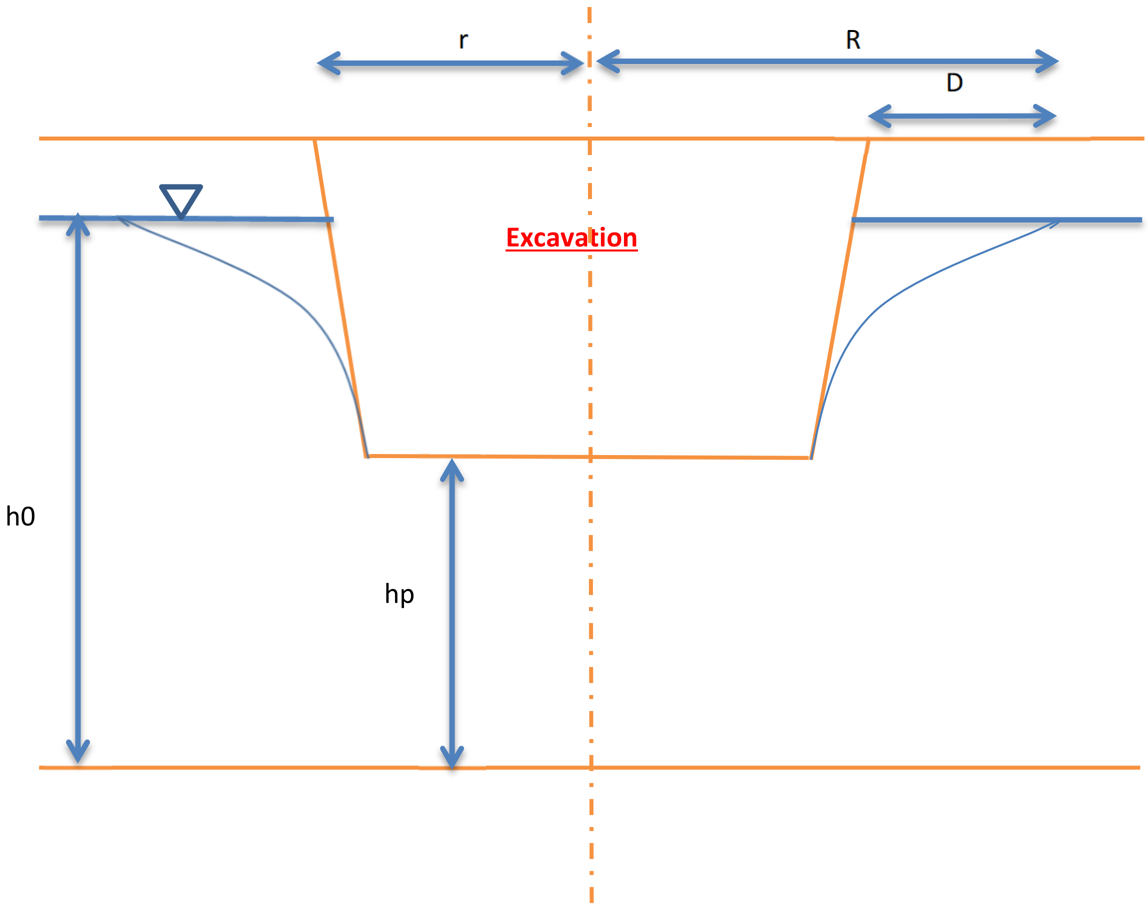
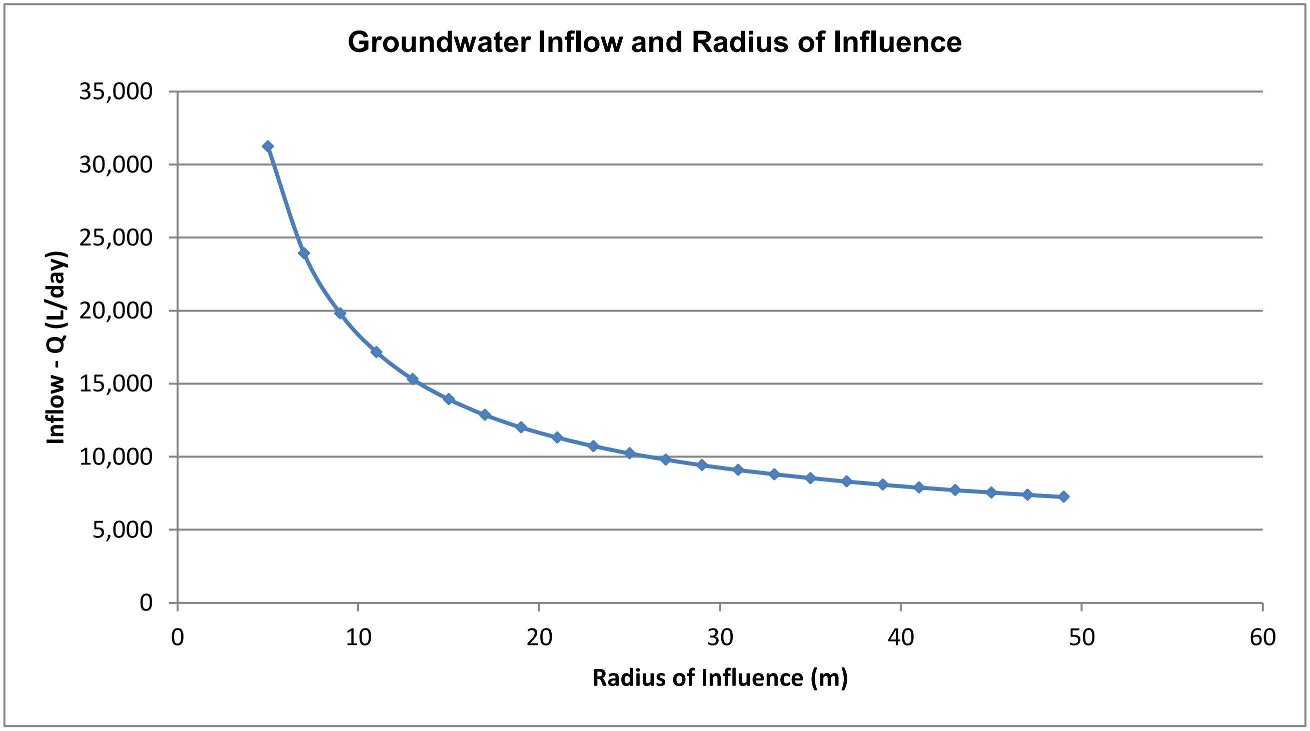

r
R
D
Excavation
h0
hp


Groundwater Inflow and Radius of Influence
Inflow - Q (L/day)
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
0
10
20
30
40
50
60
Radius of Influence (m)

File: PH5087

Estimated Groundwater Inflow

Regional Group - 4850 Bank Street - SWMP Excavation

Dupuit-Forchheimer Equation **Q= πK((h0^2-hp^2)/ln(R/r))**

K (m/sec) = 3.03E-06
h0 (m) = 4
hp (m) = 0
r (m) = 58.73

Equivalent Radius of Excavation = A+B=Pi + r

Excavation Width (X) = 32.5 m
Excavation Length (Y) = 152 m
Perimeter Length = 369 m
Equivalent Radius (r) = 58.73 m

R	Distance to edge of excavation (D)
59.73	1.00
60.73	2.00
61.73	3.00
62.73	4.00
63.73	5.00
64.73	6.00
65.73	7.00
66.73	8.00
67.73	9.00
68.73	10.00
69.73	11.00
70.73	12.00
71.73	13.00
72.73	14.00
73.73	15.00
74.73	16.00
75.73	17.00
76.73	18.00
77.73	19.00
78.73	**20.00**
79.73	**21.00**
80.73	22.00
81.73	23.00
82.73	24.00
83.73	25.00
84.73	26.00
85.73	27.00
86.73	28.00
87.73	29.00
88.73	30.00

Q (m^3/s)	Q (m^3/day)	Q (L/day)
0.0090	779	779,371
0.0045	393	392,948
0.0031	264	264,128
0.0023	200	199,710
0.0019	161	161,052
0.0016	135	135,275
0.0014	117	116,858
0.0012	103	103,041
0.0011	92	92,291
0.0010	84	83,688
0.0009	77	76,647
0.0008	71	70,777
0.0008	66	65,807
0.0007	62	61,546
0.0007	58	57,851
0.0006	55	54,616
0.0006	52	51,760
0.0006	49	49,221
0.0005	47	46,947
0.0005	**45**	**44,899**
0.0005	**43**	**43,045**
0.0005	41	41,359
0.0005	40	39,818
0.0004	38	38,405
0.0004	37	37,104
0.0004	36	35,902
0.0004	35	34,788
0.0004	34	33,753
0.0004	33	32,789
0.0004	32	31,889

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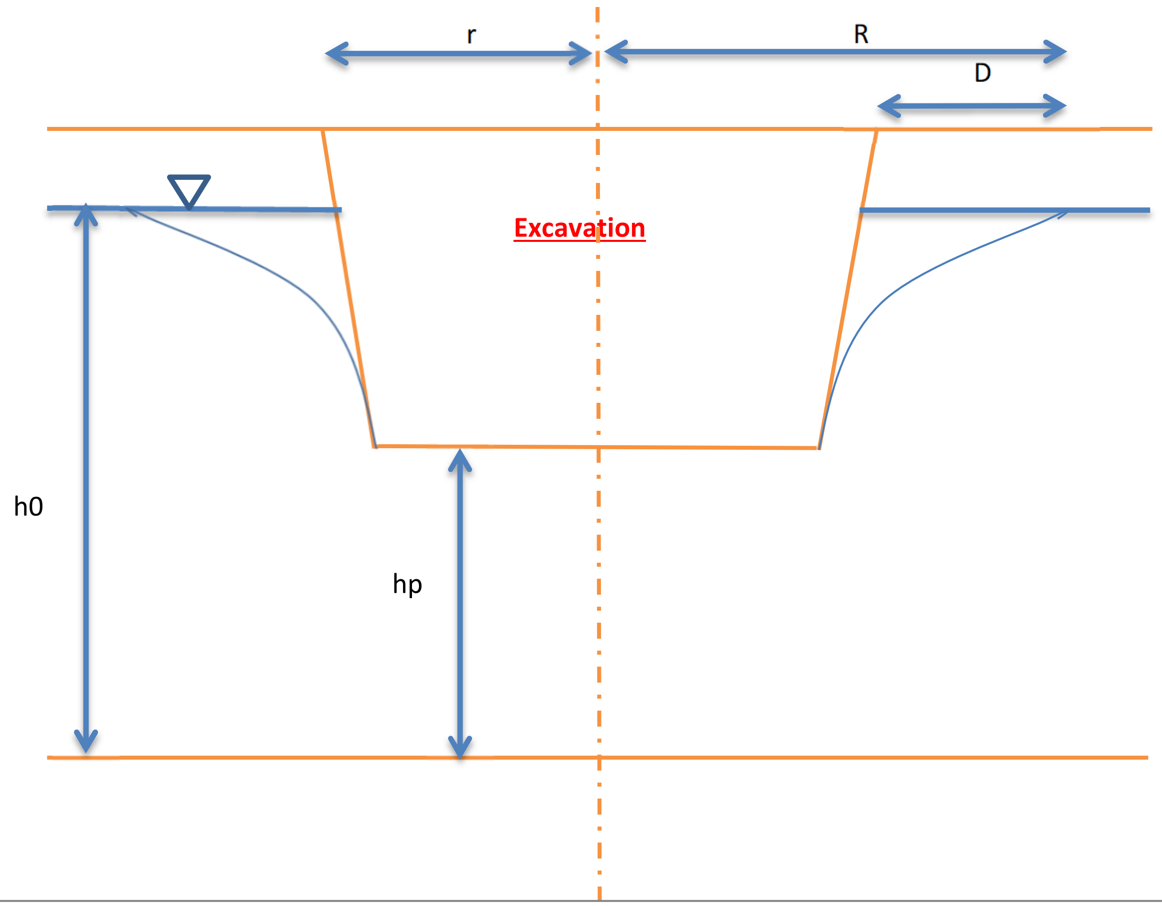
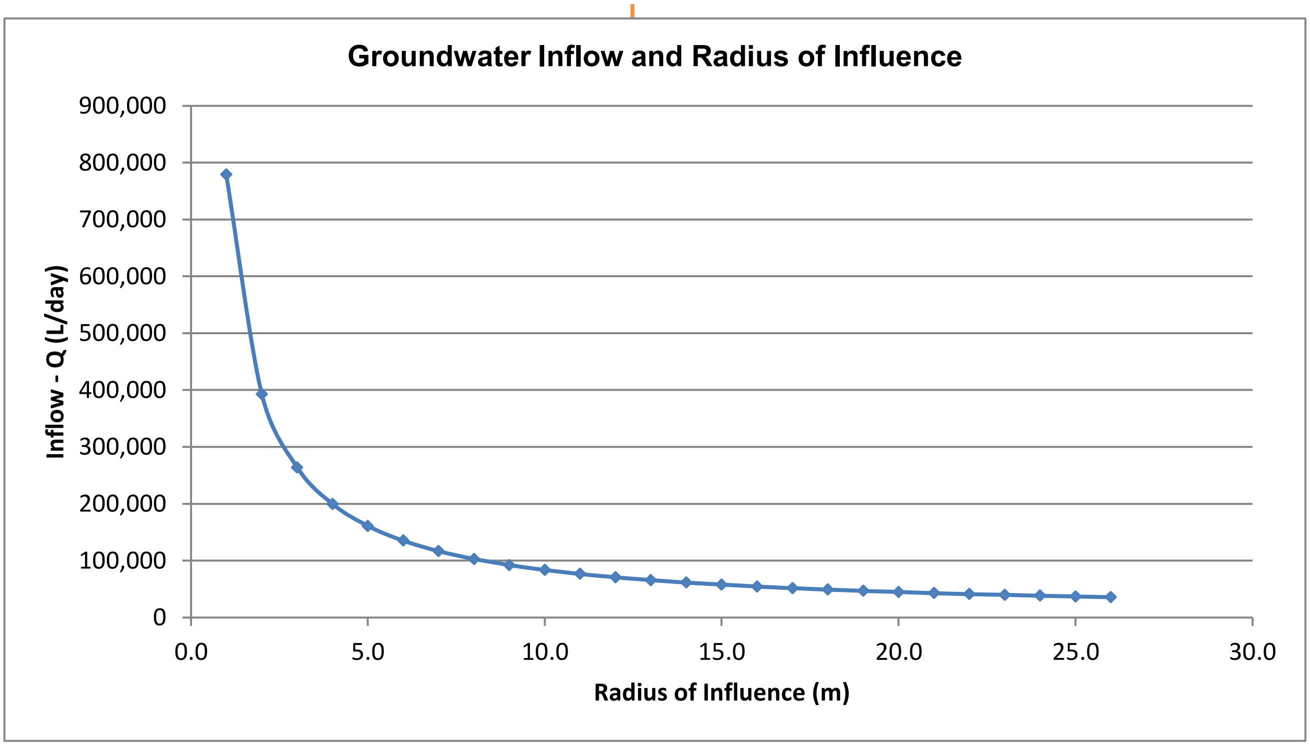

r
R
D
Excavation
h0
hp


Groundwater Inflow and Radius of Influence
Inflow - Q (L/day)
900,000
800,000
700,000
600,000
500,000
400,000
300,000
200,000
100,000
0
0.0
5.0
10.0
15.0
20.0
25.0
30.0
Radius of Influence (m)

APPENDIX 4

CITY OF OTTAWA - SALT MANAGEMENT PLAN - APPENDIX A - OCTOBER 31, 2011

City of Ottawa

Public Works and Services Department
Surface Operations Branch

Salt Management Plan

Appendix A

MATERIAL APPLICATION POLICY

CONTENT

Maintenance Quality Standards – Snow and Ice Control on Roads
General Information
Use of Liquid Chemicals
Material Application Guideline and Policy – Bare Pavement Roads
Material Application Guideline and Policy – Centre-Bare Roads
Material Application Guideline and Policy – Snow Packed Roads
Blast Policy

The Surface Operations Branch District Managers, Area Managers and Zone Supervisors have been consulted through the development of this document.

REVISION INFO

Rev	Date	By	Description
3.1	Jan 10 2007		
3.2	Oct 31 2011	D Vander Wal	• Removed 50/50 mix per Dan O'Keefe. • Removed specific references to Sodium and Calcium Chloride as new product for 2011 is a Multi-Chloride Brine. Changed liquid application rate from 46 (6%) to 39L/tonne (5%). • Removed Dry and Wet salt rates for pavement temperatures below -18C. • Updated Epoke rates to match Appendix B and added wet rates to obtain 20% reduction when pre-wetting. • Removed separate rate table for Hwy 174 Epoke spreaders since the resulting lane-km rates are the same as other bare pavement.

COUNCIL APPROVED MAINTENANCE QUALITY STANDARDS

For snow clearing, resources are to be deployed and snow clearing completed as defined in the Table below. If the depth of snow accumulation is less than the minimum for deployment, then resources may be deployed subject to road conditions resulting from previous snow accumulations or from forecasted weather conditions.

For treating icy roads, resources are to be deployed as soon as practicable after becoming aware of the icy conditions. Icy roads are to be treated within the times defined in the Table below after becoming aware of the icy conditions.

MAINTENANCE QUALITY STANDARD
SNOW AND ICE CONTROL ON ROADS

Road Maintenance Class		**Road Type**	**Minimum Depth of Snow Accumulation for Deployment of Resources** ***(Depth as per MMSMH)***	**Time to Clear Snow Accumulation From the End of Snow Accumulation or Time to Treat Icy Conditions** ***(Time as per MMSMH)***	**Treatment Standard**		
					Bare Pavement	**Centre Bare**	**Snow Packed**
1	A	High Priority Roads	As accumulation begins *(2.5-8 cm depending on class)*	2 h *(3-4 h)*	√		
	B				√		
2	A	Most Arterials		3 h *(3-6 h)*	√		
	B				√		
3	A	Most Major Collectors		4 h *(8-12 h)*	√		
	B				√		
4	A	Most Minor Collectors	5 cm *(8 cm)*	6 h *(12-16 h)*	√		
	B					√	
	C						√
5	A, C	Residential Roads and Lanes	7 cm *(10 cm)*	10 h *(16-24 h)*			√
	B		10 cm *(not defined)*	16 h *(not defined)*			√

Note - MMSMH refers to Ontario Regulation 239/02, Minimum Maintenance Standards for Municipal Highways shown for comparison purposes.

- **Bare Pavement:** requires that snow and ice be controlled, cleared and/or prevented for the full traveled road pavement width, including flush medians of 2 m width or less, paved shoulders and/or adjacent cycling lanes. It does not include parking lanes.
- **Centre-Bare:** requires that snow and ice be controlled, cleared and/or prevented in a strip down the middle of the road pavement width for a minimum width of 2.5 m on each side of centre-line.
- **Snow-Packed:** requires that snow and ice be cleared and that ruts and/or potholes that may cause poor vehicle control be leveled off. Abrasive or deicing materials are applied at intersections, hills and sharp curves.

LIQUID CHEMICALS

Application Rates and Reductions

USE OF LIQUID CHEMICALS					
Chemical	**Use**	**Application Ratio**	**Chemical Concentration**	**Application Rate**	**Dry Salt Reduction**
CaCl, MgCl, or Multi-Chloride	Pre-Wetting	5% by weight	Varies (28%-35%)	39L / t	20%[1]
CaCl, MgCl, or Multi-Chloride	Straight Liquid Application	N/A	Varies	60 to 100L/ lane-km	-

[1] The Epoke controller does not support setting a separate reduction percentage – the rate will only be reduced by the set liquid application ratio (5%).

Pre-Wetted Salt

- Pre-wetting salt is a recommended practice to enhance the performance of the road salt.
- When salt is pre-wet, the brine solution is formed quicker than dry salt and more material is retained on the road surface. It is the brine solution that prevents or breaks the bond between the road surface and snow/ice.
- The enhanced performance of the salt as well as the retention of salt on the road surface facilitates achieving a bare road more quickly and maintains bare pavement longer. As a result, a reduction in salt application rates can achieve the same effectiveness as dry salt application at traditional rates.

Practical temperature ranges for Pre-Wetted Salt (WET SALT)

- Sodium Chloride Brine ($NaCl$):
    - From 0 to –9ºC (0 to -12ºC as per pre-wetting practices in urban areas)
- Calcium Chloride ($CaCl_2$), Magnesium Chloride ($MgCl$), and Multi-Chloride Brines with a minimum eutectic temperature of -30°C:
    - From 0 to –15ºC (0 to -18ºC as per pre-wetting practices in urban areas)

Direct Liquid Applications (DLA)

- Anti-icing by Direct Liquid Application is a recommended practice to treat frost and black ice conditions in the shoulder seasons at pavement temperatures between 0 and –10ºC.
- Liquid should be applied to treat forecasted conditions at the following rates:

Winter Event	**Litres / LaneKm**	**mL / m^2 (at 3m width)**
Frost	60	20
Light Snow	60 to 80	25
Moderate to Heavy Snow, Freezing Rain	80 to 100	30

- DLA should be applied:
    - As close to the beginning of the winter event as possible
    - When the air and pavement temperatures are both below +5°C currently and forecasted to remain below +5°C within the next 12 hours.
    - When the air and pavement temperatures are a minimum of 10°C above the eutectic temperature of the DLA liquid and forecasted to do so for the next 24 hours.
- DLA should NOT be applied:

  - When the relative humidity is below 60% and the air and pavement temperatures are between 0°C and +5°C.
  - More than once in a three-day period unless a Winter Event (frost, snow, freezing rain or rain) has removed the product from the pavement. Note that DLA liquid can remain on the pavement up to several days after the initial application.

GENERAL INFORMATION

When the Pavement Temperature is below –18ºC

- Below –18ºC, the salt melting action is close to none.
- Below –18ºC, the use of salt shall be discontinued and replaced by an abrasive.
- Multiple factors can affect the performance of de-icing chemicals and abrasives below pavement temperature of –18ºC. Under such conditions, supervisors shall select the most appropriate material based on the current and expected traffic volume, current and forecasted weather and road conditions.

Abrasives

- Accepted abrasives are Sand and Grit
- Straight abrasive does contain salt to prevent the stockpile from freezing. The goal is to minimize the amount of salt mixed with the abrasives. The objective is to use an engineered abrasive of 5% salt / 95% sand or grit by volume. The following interim abrasive ratios are accepted (where the engineered ratio cannot be achieved due to equipment and material storage constraints)
  - 10% salt / 90% sand or grit by volume

Rush Hours and Forecasted Conditions

- Supervisors are responsible for making timely material application calls based on forecasted conditions and expected traffic peak hours.

Freezing Rain

- When Freezing Rain occurs, abrasive materials (sand or grit) will be applied on snow packed roads on a continuous basis (to the full Road Width).
- Snow Packed Roads – where available, graders with ice blades shall drag the roads to aid traction.

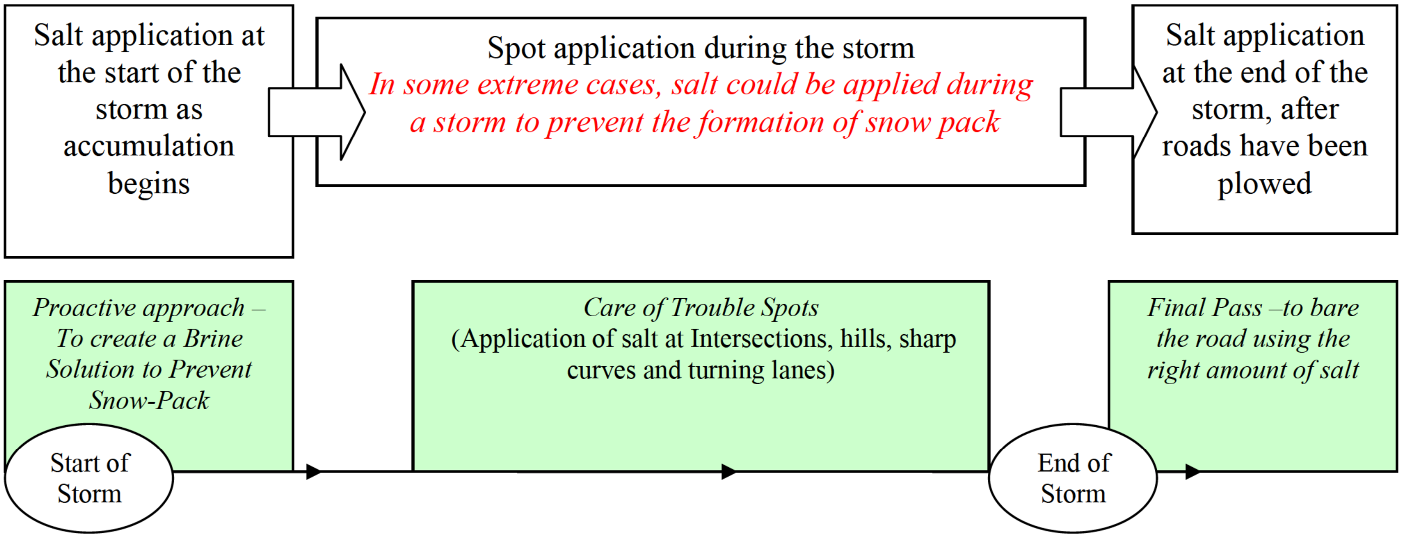
MATERIAL APPLICATION POLICY
BARE PAVEMENT

Pavement Temperature °C	**Material**	**Frost and Black Ice** ***Kg/2-lane km***	**Light Snow <1cm/hr** ***Kg/2-lane km***	**Heavy Snow >1cm/hr** ***Kg/2-lane km***	**Freezing Rain** ***Kg/2-lane km***
0 to –5°C	DRY SALT	70	100	140	230
	WET SALT	55	80	110	185
-5 to –10°C	DRY SALT	85	140	180	230
	WET SALT	70	110	145	185
-10 to –18°C	DRY SALT	85	180	230	230
	WET SALT	70	145	185	185
< -18°C*	ABRASIVE	350	350	350	-

* Refer to the General Information Section for additional information when the Pavement Temperature is below –18ºC. When forecasted warming conditions are expected, dry/wet rates of 180/145, and 230/185 may provide some baring-off benefit.

* Note: Use wet rates where pre-wetting capable spreaders and liquid supply is available.

Timing of Application – BARE PAVEMENT ROADS



Start of the Storm
Salt shall be spread just at the beginning of the icy precipitation.

End of Storm
Salt shall not be spread once bare pavement is achieved and when no further precipitation is forecasted.

MATERIAL APPLICATION POLICY
BARE PAVEMENT
(EPOKE SPREADERS)

Pavement Temperature °C	Material	Frost and Black Ice		Light Snow <1cm/hr		Heavy Snow >1cm/hr		Freezing Rain	
		g/m2	*Width*	*g/m2*	*Width*	*g/m2*	*Width*	*g/m2*	*Width*
0 to –5°C	DRY Salt (WET Salt)*	70kg/2ln-km		100kg/2ln-km		140kg/2ln-km		230kg/2ln-km	
		35 (30)	2m	50 (43)	2m	70 (60)	2m	115 (98)	2m
		23 (20)	3m	35 (30)	3m	45 (38)	3m	77 (65)	3m
		17 (14)	4m	23 (20)	4m	35 (30)	4m	58 (49)	4m
		17 (14)	5m	20 (17)	5m	28 (24)	5m	45 (38)	5m
-5 to –10°C	DRY Salt (WET Salt)*	85kg/2ln-km		140kg/2ln-km		180kg/2ln-km		230kg/2ln-km	
		45 (38)	2m	70 (60)	2m	90 (77)	2m	115 (98)	2m
		28 (24)	3m	45 (38)	3m	58 (49)	3m	77 (65)	3m
		20 (17)	4m	35 (30)	4m	45 (38)	4m	58 (49)	4m
		17 (14)	5m	28 (24)	5m	35 (30)	5m	45 (38)	5m
-10 to –18°C	DRY Salt (WET Salt)*	85kg/2ln-km		180kg/2ln-km		230kg/2ln-km		230kg/2ln-km	
		45 (38)	2m	90 (77)	2m	115 (98)	2m	115 (98)	2m
		28 (24)	3m	58 (49)	3m	77 (65)	3m	77 (65)	3m
		20 (17)	4m	45 (38)	4m	58 (49)	4m	58 (49)	4m
		17 (14)	5m	35 (30)	5m	45 (38)	5m	45 (38)	5m
< -18°C†	ABRASIVE	350kg/2ln-km		350kg/2ln-km		350kg/2ln-km		-	
		175	2m	175	2m	175	2m	-	-
		115	3m	115	3m	115	3m		
		88	4m	88	4m	88	4m		
		70	5m	70	5m	70	5m		

* When the pre-wetting system is engaged, the dry material output is reduced. The Epoke controller does not support setting a separate reduction percentage – the rate is only reduced by the set liquid application ratio (5%). Material 2 was therefore configured with rates reduced by 15%.

* Use wet rates where pre-wetting capable spreaders and liquid supply is available.

† Refer to the General Information Section for additional information when the Pavement Temperature is below –18°C. When forecasted warming conditions are expected, dry/wet rates of 180/145, and 230/185 may provide some baring-off benefit.

Notes

There are 2 variables affecting the material output on an Epoke salt spreader:

-Material Application Rate in **g/m^2** AND Application Width in **m**.

Examples:

1- For a rate of 100kg/2ln-km, the Epoke Setup would be 25g/m^2 at a Width of 4m. **OR** a rate of 50g/m^2 at a Width of 2m.

2- For a rate of 170kg/2ln-km, the Epoke Setup would be 42g/m^2 at a Width of 4m. **OR** a rate of 57g/m^2 at a Width of 3m.

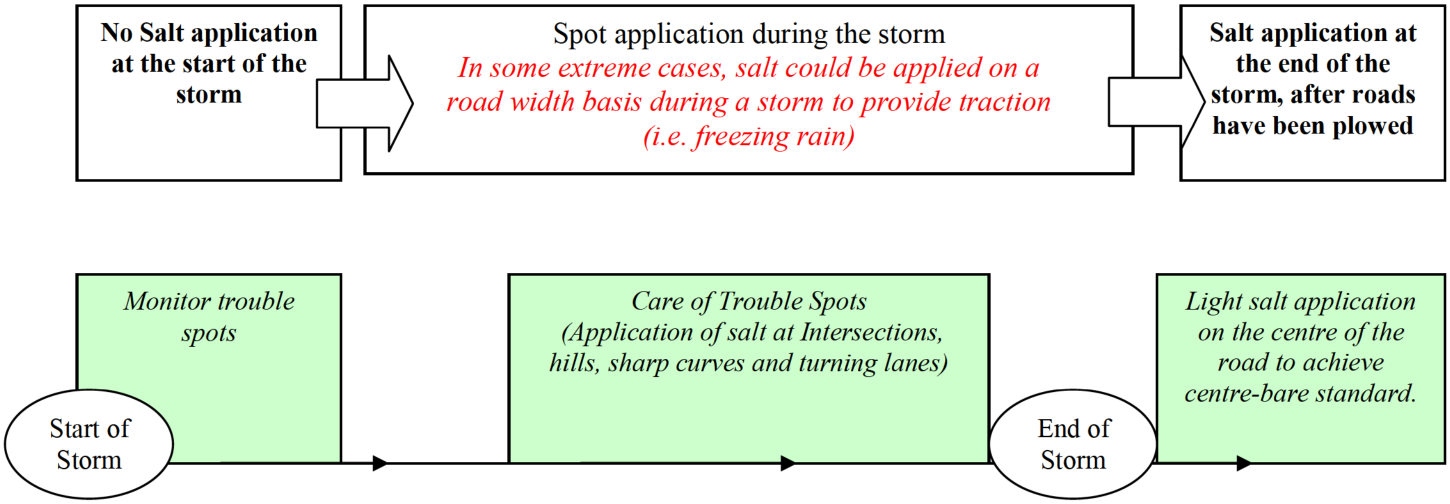
MATERIAL APPLICATION POLICY

CENTRE-BARE PAVEMENT

Pavement Temperature °C	Material	Frost and Black Ice *Kg/2-lane km*	Snow *Kg/2-lane km*	Freezing Rain *Kg/2-lane km*
0 to –5°C	DRY SALT	70	100	230
	WET SALT	55	80	185
-5 to –18°C	DRY SALT	85	140	230
	WET SALT	70	110	185
< -18°C	ABRASIVE	350	350	-

Note: Use wet rates where pre-wetting capable spreaders and liquid supply is available.

Timing of Application – CENTRE-BARE PAVEMENT ROADS



No Salt application at the start of the storm

Spot application during the storm

In some extreme cases, salt could be applied on a road width basis during a storm to provide traction (i.e. freezing rain)

Salt application at the end of the storm, after roads have been plowed

Monitor trouble spots

Care of Trouble Spots (Application of salt at Intersections, hills, sharp curves and turning lanes)

Light salt application on the centre of the road to achieve centre-bare standard.

Start of the Storm

No Salt application at the start of the storm. Monitor trouble spots.

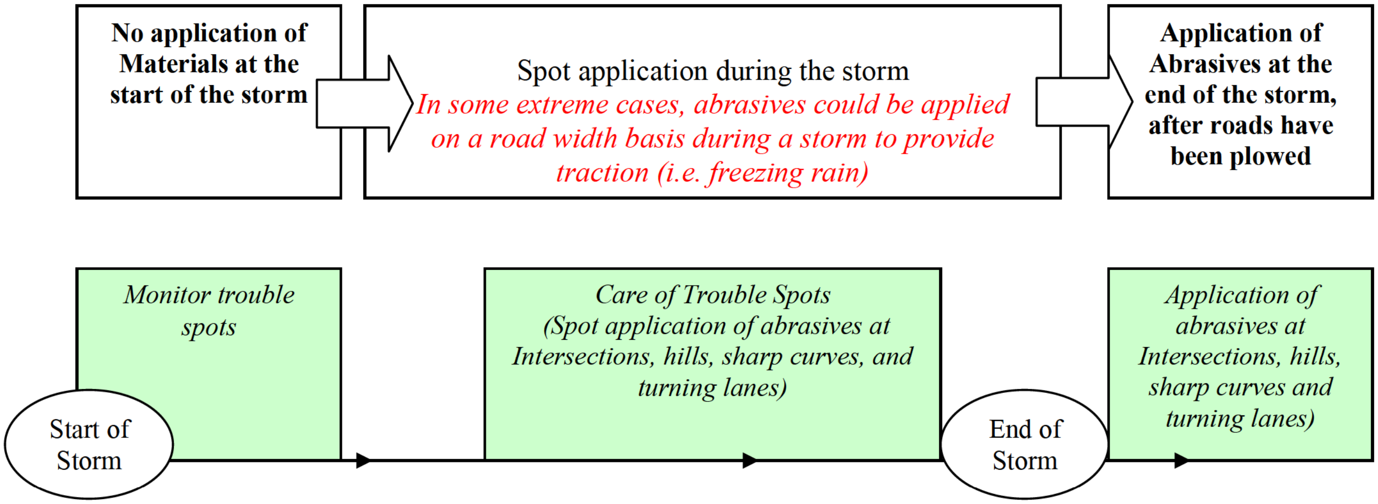
End of Storm

Salt shall not be spread once centre-bare pavement is achieved and when no further precipitation is forecasted.

MATERIAL APPLICATION POLICY
(Intersections, Hills and Sharp Curves)
SNOW PACKED

Pavement Temperature ºC	Material	Frost and Black Ice *Kg/2-lane km*	Snow *Kg/2-lane km*	Freezing Rain *Kg/2-lane km*
0 to –30ºC and below	ABRASIVE	350	350	500

Timing of Application – SNOW PACKED ROADS



Start of the Storm
No application of abrasives at the start of the storm. Monitor trouble spots.

End of Storm
Abrasives shall not be spread once traction is provided.

BLASTING POLICY

The On-Board Electronic Controller's Blast function is an important tool for roadway de-icing operations. It allows operational staff to timely increase the amount of spread material at trouble locations such as steep hills and sharp curves. Although the blast function is indispensable, it should be used with care as it its liberal use can lead to significant increases in salt consumption and environmental impacts.

- Supervisory staff shall work toward minimizing the amount of salt being spread using the Blast function to achieve the required maintenance quality standard.
- Many variables come into play during a winter weather event. As such, the call to allow the use of the Blast Function during a winter event is left to the judgment of the supervisory staff, as the first priority is the safety of the traveling public.

The Blast function shall only be used at the following locations:

- Steep Hills
- Elevated Curves
- Intersections (within 30m of the stop line on the approach side only)
- Shade areas
- Right and Left Turning Lanes
- Bus Bays
- Railways (within 30m of the railway crossing on the approach side only)
- Bridge Decks

  Caution: when blasting salt on a bridge deck. Rock salt needs heat to dissolve. Spreading salt on a bridge deck could lower its surface temperature to a point where the brine solution will refreeze.

Application:

- The Blast function shall only be used under severe winter conditions
- The Blast function shall not be used during light winter weather events such as light snow, frost, etc.
- The blast function shall not be used while clearing the roads (stripping) at the end of a storm.

On-Board Electronic Controller's Blast function

- The Epoke controllers will blast at the highest material calibration setting.
- The CS-230 controller will blast to its maximum hydraulic power (which can be adjusted if too high)
- The CS-440 controller can be calibrated at a defined Blast rate for each material.
  - The Blast Rate for Salt is to be set at 300kg/2 lane-km
  - The Blast Rate for Abrasive is to be set at 500kg/2 lane-km. Note: Suburban/Rural District has a requirement to Blast Abrasives on gravel roads at a rate of 700kg/2 lane-km. To achieve this rate, the spreaders need to be calibrated using two gate settings. The District will provide, every fall, a list of spreaders requiring this specific calibration.

FIGURES